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FOREWORD

Today, both durability and economic factors play a very important part in the choice of bridge abutment type. Durability considerations are very important in providing long-term performance of the bridge structure, both in the choice of structural form and in the design of construction details.

Economic considerations focus on the whole life cost and on the duration of construction. This is particularly so with replacement bridges, where there is a significant cost element for lane rentals for closures, and a more expensive design may make overall savings from an earlier completion time. In addition, where land/space is limited in urban areas, the type and method of construction are also important, because of the constraints being imposed.

Steel sheet pile embedded retaining walls can fulfil all the criteria for economic bridge abutments.

It is hoped that this guide will encourage designers and constructors to consider a steel substructure option more frequently during the conceptual and preliminary design phases of projects and thereby to take advantage of the potential to build more efficiently.

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SUMMARY

This document provides guidance for the design and construction of steel sheet pile embedded retaining walls for bridge abutments. The reader is introduced to steel sheet pile embedded retaining walls and to the benefits of their use as bridge abutments.

A design basis is presumed based on limit state design principles, and is written with reference to UK construction legislation, DETR Highways Agency Specifications, UK Codes of Practice, and European Standards.

Awareness of the type of soil data that is required to perform an accurate design is very important, therefore the document focuses on the importance of high quality site investigations and the selection and evaluation of soil parameters for use by the structural designer. Both limit equilibrium methods based on classical theory of soil mechanics, and soil-structure interaction methods are presented to determine the earth pressure forces acting on the sheet piles.

Bridge abutments also have to resist vertical loads from the bridge superstructure, therefore methods to determine the axial capacity of steel sheet piles in different soil types are covered by reference to the SCI publication Steel bearing piles guide.

As construction and installation are important aspects of design, there are sections on pile driving installation methods and equipment, driveability, construction tolerances, and noise and vibration during sheet pile driving.

The long-term performance of a bridge abutment has to be ensured, therefore design for durability is a major requirement. This necessitates that realistic rates of corrosion are used, and that protection to the sheet piles is provided where necessary.

A comprehensive list of references is included, together with a worked example.

Guide de dimensionnement pour les culées de ponts en pieux en acier

Résumé

Ce document est réalisé sous forme d’un guide de conception et de réalisation de rideaux de pieux en acier servant de murs de retenue des terres dans les culées de ponts. Le lecteur est familiarisé avec la technique des rideaux de pieux en acier servant de mur de retenues des terres et sur les avantages de cette technologie lors de son application à la réalisation de culées de ponts.

Le dimensionnement est basé sur les principes des états limites et est basé sur les codes anglais, DETR Highways Agency règles, UK code de pratique, et sur les codes européens.

Une bonne connaissance des caractéristiques du sol est un facteur fondamental pour le dimensionnement. En conséquence, le guide insiste sur l’importance des investigations sur site et sur l’évaluation précise des paramètres du sol qui seront utilisés par le concepteur. Des méthodes basées sur l’équilibre limite du sol et sur
l'interaction sol-structure sont présentées afin de déterminer la pression agissant sur le rideau de pieux.

Les culées de ponts doivent aussi reprendre les efforts verticaux provenant de la superstructure du pont. Des méthodes permettant de calculer la capacité portante axiale des pieux, pour différents types de sols, sont données dans le guide SCI Steel bearing piles guide.

Les problèmes liés à la réalisation de ce type de culée sont très importants et sont passés en revue tels que installation de la machine de fonçage, équipements, tolérances de construction, bruit et vibrations.

Le comportement à long terme de ce type de culée doit être garanti; ce point constitue un critère majeur du dimensionnement. Ceci nécessite qu'un taux de corrosion réaliste soit pris en compte et qu'une protection des pieux soit utilisée si nécessaire.

Une liste de références bibliographiques est également fournie ainsi qu'un exemple de dimensionnement.

Leitfaden für Brückenwiderlager aus Stahlspundwänden

Zusammenfassung


Die angenommene Berechnungsgrundlage basiert auf den Prinzipien des Grenzzustands der Tragfähigkeit, entspricht der englischen Gesetzgebung, den DETR-Richtlinien (Highways Agency Specifications), den englischen Normen und europäischen Standards.


Brückenwiderlager müssen auch vertikale Lasten aus dem Brückenüberbau aufnehmen. Daher wird für die axiale Tragfähigkeit der Stahlspundwände in verschiedenen Bodenarten Bezug genommen auf den SCI - Leitfaden Stahlpfähle.

Da Konstruktion und Einbau wichtige Aspekte der Berechnung sind, müssen die verschiedenen Verfahren und Ausrüstungen, die Rammbarkeit, Bautileranzen, Lärm und Vibrationen während des Eintreibens berücksichtigt werden.

Das Langzeitverhalten eines Brückenwiderlagers muß sichergestellt sein, daher ist die Dauerhaftigkeit eine wichtige Forderung. Die Annahme einer realistischen Korrosionsrate ist nötig und, falls erforderlich, ein Korrosionsschutz für die Spundwände.

Eine umfassende Referenzliste und ein Rechenbeispiel sind enthalten.
Guida progettuale per le spalle da ponte su palancole in acciaio

Sommario

Questa pubblicazione fornisce una guida alla progettazione e realizzazione di spalle da ponte con muri di contenimento a palancole. Per il lettore viene proposta un'introduzione alle palancole in acciaio per muri di contenimento e sono sottolineati i benefici associati al loro utilizzo nelle spalle da ponte.

Viene presentato un approccio progettuale basato sulla filosofia degli stati limite ed è fatto riferimento alla legislazione in vigore nel Regno Unito relativamente alle costruzioni, alle raccomandazioni DETR società Autostrade, ai codici progettuali del Regno Unito e alle vigenti normative europee.

Nonostante la perfetta conoscenza del tipo di terreno sia un'esigenza primaria per effettuare una progettazione accurata, il documento è focalizzato sull'importanza di indagini in situ di elevata qualità e sui criteri di scelta e valutazione di quei parametri del terreno utili ai fini della progettazione strutturale. Per la determinazione della pressione del terreno agente sulle palancole sono introdotti i metodi dell’equilibrio limite, basati sulla teoria classica della meccanica del suolo, e quelli per la valutazione dell’interazione terreno struttura.

Dovendo le spalle essere progettate per sopportare le azioni verticali dell’impalcato, vengono richiamati i metodi per determinarne la capacità portante in relazione a diversi tipi di terreno e in accordo alla pubblicazione SCI Guida alle pile in acciaio.

In aggiunta, essendo la costruzione e l’installazione importanti aspetti progettuali, viene fatto riferimento ai metodi e all’attrezzatura per l’installazione con battipalo, alla manovrabilità, alle tolleranze di costruzione e ai problemi associati a rumori e vibrazioni legate alla messa in opera.

Le prestazioni a lungo termine delle spalle devono essere garantite e pertanto risulta di primaria importanza anche la durabilità dell’opera. Ciò implica che si faccia riferimento, in termini realistici, al problema della corrosione e che sia prevista, se necessaria, una specifica protezione.

Viene inclusa un’esauriente lista di riferimenti bibliografici ed è riportato anche un esempio applicativo.

Guía para proyecto de tablestacas metalicas en estribos de puente

Resumen

Este documento contiene información para el proyecto y construcción de tablestacas de acero para contención de tierras en estribos de puentes. Se trata de mostrar al lector las ventajas de los muros de contención con tablestacas de acero embebidas, en su uso como estribos de puente.

Se presupone que el proyecto está basado en la filosofía de estados, limite y que se ajusta a la legislación de construcción en UK, normas de la Agencia de Autopistas DETR, normas de buena práctica y Recomendaciones europeas.
Como es muy importante el conocimiento del tipo de datos del suelo necesarios para llevar a cabo un buen proyecto, el documento incide en la importancia de la calidad de los datos, obtenidos in situ así como en la selección y evaluación de los parámetros de suelo a usar en el proyecto de la estructura. Para el cálculo de los empujes que actúan sobre las tablestacas se presentan los métodos de equilibrio límite basados en las teorías clásicas de mecánica de suelos así como los métodos de interacción terreno-estructura.

Los estribos de puente también tienen que resistir las cargas verticales provenientes de la superestructura, por lo que se usan los métodos descritos en la monografía Steel Bearing Piles Guide del SCI para determinar la capacidad axial de las tablestacas.

Puesto que, en el proyecto, tanto la construcción como la colocación son temas importantes, deben considerarse los métodos de hinca, las tolerancias de construcción, colocación y equipamiento, así como los problemas de ruido y vibración.

Debe asegurarse el funcionamiento a largo plazo del estribo, por lo que la durabilidad es un requisito importante; ello implica el uso en el cálculo de velocidades de corrosión reales y que se establezca la protección de las tablestacas cuando sea necesario.

Junto con un ejemplo completamente desarrollado se incluye una lista de referencias.
1 INTRODUCTION

Steel piling for abutments of new and replacement bridges can be aesthetically attractive and cost effective. Use of embedded steel sheet piling brings savings in dead load, provides a compliant retaining wall, and permits speedier construction. In addition, for replacement bridge projects, traffic interruption can be minimised.

It is hoped that this guide will encourage designers and constructors to consider a steel substructure option more frequently during the conceptual and preliminary design phases of projects and thereby to take advantage of the potential to construct more efficiently.

1.1 Scope

The reader is introduced to the two types of steel sheet pile section for embedded retaining walls that are commonly used. Benefits of their use are presented, as is the consideration of aesthetics. Examples of recently constructed steel sheet pile bridge abutments complete this introductory Section.

A design basis is used based on limit state design principles, which is written with reference to UK construction legislation, Department of Transport Specifications, UK Codes of Practice, and the new Eurocodes. A flowchart is given, showing how a typical design procedure can be implemented.

Awareness of the soil data that are required to perform an accurate design of a sheet pile bridge abutment is very important. The document therefore advises on site investigations and the selection and evaluation of soil parameters. Wherever practicable, soil parameters are to be measured, however generic default values are also presented. Both total stress and effective stress principles are defined, and their application to short- and long-term design situations is explained.

The limit equilibrium analysis methods based on classical theory of soil mechanics are used to determine the overall stability of embedded walls. For determination of bending moments and forces acting on the bridge abutment due to lateral soil loading, however, both limit equilibrium methods and the methods that address soil-structure interaction are presented. Although limit equilibrium methods have been used predominantly in the past to analyse embedded retaining walls, the soil-structure interaction methods are now becoming used more widely for more economic design. This is because the soil-structure interaction methods are more theoretically correct and can be applied to more complex design situations.

Bridge abutments have to resist vertical loads from the bridge superstructure, therefore the axial capacity of steel sheet piles must be known. Comprehensive information on axial capacity of sheet pile retaining walls is presented based on the load transfer mechanisms from sheet pile to soil.

As construction and installation are important aspects of design, a comprehensive treatise is given that addresses pile driving installation methods and equipment, driveability, construction tolerances, and the environmental implications of noise and vibration during installation.
Consideration of long-term performance of the bridge abutment requires that design for durability is addressed. Both corrosion and the protection of steel sheet piles in natural ground and industrial soils are therefore discussed.

A comprehensive list of references has been included together with a worked example to illustrate use of design procedures.
2 BRIDGE ABUTMENTS AND SHEET PILE SECTIONS

This Section introduces the reader to the forms of steel sheet embedded retaining walls that are used for the abutments of simply supported bridges (see Figure 2.1). Steel sheet piles have many inherent advantages over the other forms of material and these advantages are highlighted. In addition it is shown that steel sheet piling can be made aesthetically pleasing with minimum additional effort. The use of steel sheet piles for bridge abutments is not a new concept, and many such bridges have been built. Examples of these bridges are described.

![Figure 2.1 A typical sheet pile bridge abutment](image)

2.1 Embedded steel sheet pile abutments

There are two principal ways in which sheet piles can be used in bridge abutments: as cantilever walls and as anchored walls.

2.1.1 Cantilever walls

Cantilever walls rely on their embedment depth to provide support for the soil and to carry the weight of the bridge superstructure (see Figure 2.2). These walls are generally restricted to abutments where the maximum retained height does not exceed about 4.5 m. The retained height is restricted to prevent lateral deflections in the in-service state becoming too large, and to guard against the use of large wall section sizes (there is a rapid increase in required section size as the retained height increases).

Cantilever steel sheet pile retaining walls usually have an in-situ reinforced concrete pile cap, which provides an effective transfer of forces from the bridge deck to the abutment, without the need to consider tolerances in pile installation.
2.1.2 Anchored retaining walls

Anchored retaining walls are similar to cantilever walls except that they are provided with an additional support near the top of the wall (see Figure 2.3). For these types of walls it is possible to increase the retained height of the abutment significantly (up to about 15 m), keep lateral deflection magnitudes to within in-service limits, and reduce structural forces in the wall. As an additional support is provided by the anchor, the required embedded depth is reduced noticeably, which reduces piling costs, however extra cost and construction work are required to install the anchor system.
2.2 Steel pile section profiles

Steel piles for bridge abutment walls are available in various profiles. 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 an abutment depends on the size and configuration of the bridge structure and the magnitude of the axial and lateral loads.

2.2.1 Sheet piles

For bridges of small span and/or retained height, adequate abutments can be formed from steel sheet piles. Two profiles, designated as U and Z, are available. In the UK, Larssen (U profile) and Frodingham (Z profile) sections are commonly used. There are no real preferences for the use of Larssen or Frodingham sections, although each type of section does have its own characteristics, which in certain situations can influence the choice.

Larssen sections

Larssen sections (U profile) are proprietary products manufactured by British Steel (see Figure 2.4). As Larssen profiles are interlocked along the centre line of the combined section (at the position of maximum shear in the section), it is necessary that the total shear force is transferred across the interlock for the full modulus of the combined section to be developed. If this shear force is not transferred across the interlock, increased lateral deflections of the pile wall may be observed.

Figure 2.4 A Larssen sheet pile

When sheet piles are fully driven into reasonably good soils, the shear force between the outer and inner piles in a pair of Larssen type piles is resisted by

- friction at the interlocks
- the capping beam
- the embedment of the piles below excavation level
- friction at the soil/steel interfaces
- loads transmitted from the walings where appropriate.

Interlocks can however be crimped or welded:

- where cantilever walls are to be piled
- where pile penetration is obstructed by underlying rock
where the piling passes through soft clay or water
when lubrication is applied to the interlocks.

Recently British steel’s range of Larssen steel sheet pile sections has also been increased to include wider sections. These new sections are called LX sections and have been introduced to provide a superior strength-to-weight sheet pile that has comparable driving capabilities and saves driving time.

Larssen sections have a greater range of section moment of inertia than Frodingham sections. The range of section modulus for LX varies from 830 cm$^3$/m for the LX8 section to 3201 cm$^3$/m for the LX32 section, and for the Larssen section from 610 cm$^3$/m for the 6W section to 5066 cm$^3$/m for the 6 section. The complete range of Larssen sections and their properties are listed in the British Steel *Piling handbook*.

**Frodingham sections**

Frodingham sections (Z profile) are also proprietary products that are manufactured by British Steel (see Figure 2.5). They are usually supplied interlocked in pairs, which saves time in handling and pitching. Crimping and welding of Frodingham piles are not required to ensure the full strength of the piled wall (interlocks are located at the extremity of the wall where shear force in section is zero), but they can be of benefit in handling and driving and ensuring watertightness of the wall.

![Figure 2.5 A Frodingham sheet pile](image)

Frodingham sheet piles have advantages in a marine environment, as the interlocks are more watertight than Larssen piles.

The range of section modulus for Frodingham profiles varies from 688 cm$^3$/m for the 1BXN section to 3168 cm$^3$/m for the 5 section. The complete range of Frodingham sections and their properties are listed in the British Steel *Piling handbook*.

### 2.2.2 High Modulus Piles

A High Modulus Pile is a proprietary product manufactured by British Steel Piling. A typical section is shown in Figure 2.6 for Frodingham sheet piles welded to the outer surface of the flange of a Universal Beam. Each pile is driven clutched to an adjacent pile to form an interlocking sheet pile wall acting compositely with the Universal Beams. The resulting profile provides a wall of identical repeating units. High Modulus Piles are produced in a range of sizes...
such that the structural capacity can be very closely matched, thus providing maximum economy.

Currently British Steel offers High Modulus Piles formed from pairs of Frodingham profile section welded to a wide variety of Universal Beams, which are spaced at 850 or 966 mm centres, depending on the section selected. Two Frodingham sections (4N and 5) combined with eleven weights of Universal Beam are available.

As the Universal Beam increases the structural capacity of the section significantly, it has a greater stiffness that is capable of supporting both horizontal and axial loads with acceptable deflections. This type of section is most appropriate for medium to large span bridges where the magnitude of loading and/or retained height rule out Larssen or Frodingham sheet piles. The spacings of the Universal Beams are ideal for the construction of shallow bridge decks formed of beams at close centres, but sufficiently far apart for inspection and maintenance.

The complete range of High Modulus Piles, their dimensions and their properties can be obtained from the British Steel Piling handbook.

2.2.3 Box piles

Box piles are formed of two or more sheet pile sections welded together. 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. They are used to support vertical and horizontal forces and can be positioned in the pile abutment so that its appearance is unaffected.

Frodingham plated piles are formed by continuously welding a plate to a pair of interlocked and intermittently welded sheet piles, while Larssen box piles are formed by welding together two sheet pile sections with either continuous or intermittent welds (see Figure 2.7).

Special box piles can be formed using certain combinations of sheet piles. Further information can be obtained from British Steel Piling Technical Services, Scunthorpe.
2.3 Applications and benefits of sheet piles in bridge abutments

The use of steel sheet and High Modulus Piles provides a prefabricated, high quality foundation of known structural integrity that fulfils the requirement for minimum construction time. Not only can piles be driven rapidly in the vast majority of soil types but they are capable of being loaded immediately, which is a distinct advantage in fast-track construction projects.

By adopting sheet piling or High Modulus Piles for abutments, construction space is minimised and major excavation avoided. Piles can be driven at the minimum distance from an existing carriageway or existing concrete foundations of previous bridges; this means that where a single span bridge is chosen, the span of the new bridge can be minimised. As there is no initial requirement for anything more than minor excavation or filling around the piles, the bridge deck may be installed at a very early stage, for example prior to the main excavation works when widening roads. This will again minimise the disruption to traffic and reduce overall costs.

These advantages and others can be summarised as follows:

- Construction is significantly quicker than that for rigid reinforced concrete foundations.
- There is no requirement to excavate for foundations.
- There is no disturbance of the existing ground during piling.
- The steel components have shop quality as opposed to site quality.
- They can easily be made aesthetically pleasing.
- They can be placed in advance of other works.
- They have immediate load-carrying capacity.
- They can be used as curtain walling to contain the working site.
- The wall can be installed in stages for rail bridge weekend possessions.
2.4 Aesthetics

Bridge abutments can be very dominant features on the urban and rural landscape. Careful design can use the appearance of sheet piles to good effect without needing the significant increase in cost of cladding.

Apart from having to satisfy the functional requirements, a steel sheet pile should be made to blend in with its surrounding environment as far as possible and to be aesthetically pleasing. The aspects that are important are:

- height of abutment/pier and inclination of its front face
- curvature of the abutment/pier in plan (poor design can give an appearance of a kink in the longitudinal elevation of the wall)
- gradient and surface treatment of the adjacent ground
- surface textures of the front facing
- the coping of the abutment.

The appearance of a sheet pile can be altered by choice of colour or cladding if required.

2.5 Examples using sheet pile abutments

Many bridges have been built using steel sheet pile abutments. These bridges include small bridges with spans of the order of 5 m (subways or bridges over small rivers/brooks), small to medium bridges with spans in the range 8 to 20 m (bridges over canals, rivers, and single carriageway roads), and large span bridges that span between 20 and 35 m. Where the bridge span and the retained height is small, Larssen or Frodingham steel sheet piles are most appropriate; otherwise High Modulus Piles may be used. Examples of typical sheet pile abutment bridges are described below.

2.5.1 Humber Road bridge, Immingham

The Humber Road bridge is located within Immingham docks on Humberside and was constructed as part of the realignment of the Humber Road near the West gate into the docks. Although situated in a heavy industrial area, it was important that the bridge should be both functional and attractive, while creating an easily maintained structure that would represent good value for money in both initial and whole life costs (see Figure 2.8).

The bridge structure is simply supported, spans 36 m, and bridges over a railway on a skew of 25½°, onto abutments that are approximately 8 m in height. For ease of erection, with minimal possession of the rail track, a superstructure of reinforced concrete deck slab acting compositely with five steel plate girders was adopted. For the bridge abutments, a sheet piled box structure was constructed using a combination of Larssen 32W and 20W. As differential settlement is a problem at this site, a reinforced concrete bankseat abutment was constructed at high level on granular infill, to provide the greatest possible distribution of bridge deck loading. The steel was protected by surface treatment using high-build, isocyanate cured, epoxy pitch coating.
2.5.2 Canal bridge, Stoke-on-Trent

This bridge replaced a Victorian wrought iron beam and brick jack bridge in Stoke-on-Trent. As the bridge was located on a busy route it was necessary to maintain two-way traffic flows during construction of the new bridge. The choice of steel sheet piling proved the most practical solution. The main advantage of employing steel sheet piling was that the piles would act as both foundation and abutment and could be driven in a single operation to reduce construction time considerably. Frodingham 3N and 4N sections were used, driven by a 1-tonne hydraulic hammer.

The bridge spans the Trent and Mersey Canal and a towpath in the heart of a conservation area (see Figure 2.9). Planning consent dictated that the piled elevations of the bridge be clad aesthetically to blend with surrounding buildings. Wing walls were clad in brickwork and the abutment faces underneath the bridge deck were painted. A vinyl ether coating was applied on site to give a blue semi-gloss finish (British Steel PC3 protective coating).

2.5.3 Holes Bay bridge, Poole

The Holes Bay bridge spans across the main Waterloo to Weymouth railway line (approximately 16 m). Owing to the presence of artesian pressures and the requirement for low displacement piles (because of the proximity of the railway and a drainage channel), it was decided to use permanent sheet pile abutments.

High Modulus Piles (of size Frodingham 3N) were used for the abutments with $610 \times 305 \times 149$ kg/m Universal Beams (18 m long) welded on the back. The abutments were anchored back to a continuous concrete anchorage.
2.5.4 Stockman's Lane flyover, Belfast

The Stockman's Lane flyover comprises a 20-m span M-beam bridge deck on permanent sheet pile abutments and wing walls. The flyover runs alongside an existing railway bridge and embankments. The choice of permanent sheet piling for the road bridge abutments avoided hazardous excavation and the use of extensive temporary sheet piling. In addition, this form of construction caused minimum disruption to traffic and no disturbance at all to service pipes and cables nearby.

The abutment piles are Larssen 4/20 piles in Grade S355 steel, boxed for part of their length and tied to concrete anchor blocks. A reinforced concrete capping beam was designed to provide bearing seatings and curtain walls.

2.5.5 Simwhite bridge, Grimsby

The Simwhite bridge is a twin deck bridge allowing a dual carriageway to span across a river. The bridge comprises steel sheet piling supporting a simple, reinforced concrete bankseat. The piles used were 16 m lengths of 25W Larssen, driven using conventional panel driving techniques. Support to the heads of the piles is provided by tie rods connected to concrete anchorages.

2.5.6 Capel St Mary A12 underpass bridge, near Ipswich

The A12 spans across a single carriageway underpass road (see Figure 2.10). The span of the bridge is approximately 10 m, with a retained height of approximately 5 m. The abutments consist of High Modulus Piles that are anchored using inclined ground anchors. At the top of the abutment a reinforced concrete pile cap beam is provided, onto which the reinforced concrete deck beams are seated. The steel sheet piles are exposed at the abutment and at the wing walls, with the exposed surface shotblasted and painted to a coloured finish to a DOT Specification.
Figure 2.10 *Capel St Mary bridge, near Ipswich*

2.5.7 **Railway underline bridge, Shrivenham**

This bridge forms part of the improvements undertaken for British Rail’s high speed service from London to the South-West. The structure bridges a new road and was built within an existing embankment. It replaces a level crossing on the line between Shrivenham and Knighton near Swindon. The main abutment piles comprise sheet pile and Universal Beam elements. The wing walls are Larssen sheet piles. As it was not feasible to anchor the heads of the piles in the embankment, British Rail engineers used the bridge parapet beams to restrain the ends of prestressed transom beams. These in turn support the heads of the piles. When the deck was complete, excavation was carried out and the new road constructed. The exposed faces of the piles were shotblasted and painted.
3 DESIGN BASIS

This Section presents the design basis for sheet pile retaining wall bridge abutments where the bridge deck rests on bearings at the top of the wall.

The design basis for fully integral bridge construction, involving moment/rotation continuity between the bridge beams and the wall, and lateral displacements of the top of the wall due to bridge thermal expansion and contraction, is not covered in this publication. Design guidance for such integral construction is given in the SCI Publication *Integral steel bridges: Design guidance* (2).

3.1 General principles

The design guidance given in this document relates to the use of steel sheet piling acting as a retained wall at the end of a bridge. The sheet piling resists lateral pressures by bending action, either as a vertical cantilever or as an anchored or propped cantilever. Along part or all of its width, where it supports the bridge deck, the piling is also required to carry vertical loads.

Limit state design is generally presumed throughout this publication for the whole bridge.

3.2 Design standards

3.2.1 National and European standards

For bridges, the applicable National Standard is BS 5400(3). This covers the design of steel, concrete, and composite structural elements of bridges. Composite bridge deck design is normally in accordance with Parts 3, 4, and 5 of BS 5400.

For the design of *earth retaining structures*, reference may be made to BS 8002(4), or the earlier CP2(5), and for foundations to BS 8004(6).

**BS 8002**

BS 8002 is primarily applicable to small- to medium-sized earth-retaining structures with a retained height of up to approximately 8 m, although it is stated that many of the recommendations are more generally applicable. It is a code of practice based on the use of the simplistic and traditional limit equilibrium method for retaining wall design that has been used to ensure the stability of walls. This method is based on the use of theoretical limiting earth pressures without practical proof that they can co-exist at the ultimate limit state of wall overturning.

As BS 8002 is based on this simplistic approach, it cannot define uniquely the limit states that are to be used for design. This is seen in its definition for limit state design in that “the safety and stability of the retaining wall may be achieved, whether by overall factors of safety, or partial factors of safety, or by other measures”. Owing to this lack of precision in definition, BS 8002 refers both to partial factor based limit state codes of practice such as BS 8110(7), BS 5400, and BS 5950(8), and also to BS 449: Part 2(9), which is a *working stress code of practice* based on the use of lumped factors of safety.
BS 8002 uses an approach based on worst credible soil and ground parameters to develop an adequate margin of safety\(^{(10)}\). A mobilised soil strength is advocated for use in design at the serviceability limit state only (so as to limit retaining wall movements), because BS 8002 states that “the most severe earth pressures that can credibly occur on a retaining wall, occur at that limit state”. The mobilised soil strength is obtained by dividing the representative strength by the mobilisation factor.

No analysis at the ultimate limit state is required by BS 8002 because it assumes that the forces acting on the retaining wall at the serviceability limit state are greater than those at the ultimate limit state. This is a practical resolution but does not comply strictly with limit state design philosophy.

According to the research by Potts and Fourie\(^{(11)}\), which is quoted in the ISE publication *Soil-structure interaction - the real behaviour of structures*\(^{(12)}\), the earth pressures generated are related to wall movement (see Figure 3.1), and much more precision is required in the definition of serviceability limit state before the ultimate limit state can be dismissed, as is stated in BS 8002.

Further discussion is given in Section 6.1.

![Figure 3.1](image)

*Figure 3.1  Development of earth pressure coefficients with increasing wall displacement (rough wall) - Potts and Fourie\(^{(11)}\)*

**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 design where it is assumed that *total stresses* act in the soil. *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 encountered in temporary works design where undrained soil behaviour and soil properties are relevant.

**Eurocodes**

Recently two CEN documents have become available. Eurocode 7 *Geotechnical design* has been issued by BSI as DD-ENV 1997-1(13), and the draft prestandard Eurocode 3: Part 5 *Design of steel structures - Piling* has been circulated in industry as prENV 1993-5(14). These are fundamentally more rigorous documents that apply limit state principles and use a partial factor approach.

Eurocode 7 can be of some help in the design of bridge abutments since it uses limit state design with partial factors that are compatible with the design approach of BS 5400.

In Eurocode 7, the partial factors that are applied to the characteristic value of the soil parameter are presented in Clause 2.4.2, table 2.1, which is reproduced below for information. Annex B of DD ENV 1997-1 provides additional recommendations relating to these partial factors.

**Table 3.1**  *Partial factors to be applied to actions and ground properties, according to DD ENV 1997-1*

<table>
<thead>
<tr>
<th>Case</th>
<th>Actions</th>
<th>Factor on ground properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Permanent</td>
<td>Variable</td>
</tr>
<tr>
<td></td>
<td>Unfavourable</td>
<td>Favourable</td>
</tr>
<tr>
<td>Case A</td>
<td>1</td>
<td>0.95</td>
</tr>
<tr>
<td>Case B</td>
<td>1.35</td>
<td></td>
</tr>
<tr>
<td>Case C</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

Notes: Case A Loss of static equilibrium; strength of structural material or ground insignificant. Case B Failure of structure or structural elements, including those of the footing, piles, basement walls, etc., governed by strength of structural material. Case C Failure in the ground.

$\phi$ Angle of shearing resistance

$c_u$ Undrained shear strength

### 3.2.2 Highways Agency standards

The Highways Agency sets out its requirements and advice for the design and construction of structures in Standards (BDs) and Advice Notes (BAs). The BD documents contain mandatory requirements that are effectively the *building regulations for bridges in the UK*. Some of the BD documents are used to invoke BSI codes of practice as mandatory design rules, subject to a few amendments to suit the Highways Agency’s particular requirements. The BA documents give background information, guidance on the application of the related BD, and
generally recognised rules that satisfy the requirements of the BD. The BAs are not intended to be mandatory.

**Durability**

Durability considerations are very important to ensure long-term performance of the bridge structure, both in the choice of structural form and/or in the design of construction details. The Highways Agency has published documents BD 57 and BA 57 *Design for durability*\(^{(15)}\), which present requirements for durability. The basic requirement is a design life of 120 years, but this is proving difficult to achieve with modern reinforced and prestressed concrete construction. The cost of maintaining their structural strength and integrity is high. Concrete piling also has durability problems due to reinforcement corrosion. No attention is currently given to this aspect, but it is as relevant as the consideration of corrosion losses on steel piling. Driven concrete piling suffers damage due to cracking caused by tension waves and this creates a ready path for oxygenated ground water to cause corrosion that destroys the integrity of the pile.

**Wall design**

For embedded retaining walls that are part of bridges and other highway structures, requirements are given in the Highways Agency’s BD 42 *Design of embedded retaining walls and bridge abutments (unpropped or propped at the top)*\(^{(16)}\). This document specifies that limit state design principles are to be applied both for ultimate and serviceability states, and that BS 8002 should only be used for particular aspects of design. For steel embedded retaining wall bridge abutments, design is to be in accordance with BS 5400: Parts 3, 4, and 5, using the limit state partial factor approach and not the working stress approach.

The purpose of BD 42 is stated as “not to provide comprehensive design rules, but to set out the significant requirements of design and construction with a view to achieving greater consistency in approach between designers”. Reference is made to CIRIA Report 104 *Design of retaining walls embedded in stiff clay*\(^{(17)}\), where the principles of limiting equilibrium stability are used to determine earth pressures and forces acting on the structure. CIRIA Report 104 was written primarily to address the design of singly propped or cantilever walls embedded in stiff over-consolidated clays where the diaphragm wall technique or the driving of sheet piles is to be used. CIRIA Report 104 says that “it is also applicable, in principle, to firm clays and granular soils”.

Although the text in BD 42 relates to the simple limit equilibrium methods used in CIRIA Report 104, the use of numerical analytical methods where soil-structure interaction is considered is equally permissible.

Along part of their length, retaining walls for bridge abutments may need to resist vertical loads from the bridge superstructure. Appropriate rules for evaluating the vertical resistance of retaining walls are given in BD 32 *Piled foundations*\(^{(18)}\). BA 25\(^{(19)}\) provides guidance on the use and interpretation of BD 32.

For the structural design of steel bridges, BD 13 *Design of steel bridges*\(^{(20)}\) invokes the use of BS 5400: Part 3, and design load effects from the bridge deck are specified in document BD 37 *Loads for highway bridges*\(^{(21)}\).

For the assessment of the overall stability of the soil/abutment, BD 42 requires that the *recommended factors of safety* in CIRIA Report 104 (table 5, page 73) are
used. No partial factors are applied to the relevant highway loadings, and the partial load factors for earth pressure, as defined in BD 37, are not applied.

For the design of the structural steel piling elements that comprise the bridge abutments, BD 42 permits either a limit equilibrium approach or an approach based on soil-structure interaction. Where the limit equilibrium approach is to be used, BD 42 requires that "bending moments and forces in the wall are determined in accordance with CIRIA Report 104". Load effects due to earth pressures (bending moments and shear forces in the wall), however, are multiplied by partial factors $\gamma_f$, as stated in Section 3.5 to 3.12 of BD 42. This requirement supersedes the requirements presented in document BD 37. Relevant highway loadings applied to the bridge abutment and the corresponding partial factors are in accordance with those stated in BD 37.

Where a numerical analytical method is used for design of a bridge abutment, i.e. a soil-structure interaction method, the stability requirements, using the limit equilibrium approach, as stated in BD 42, are to be followed to determine the depth of embedment of the wall. For the design of structural elements, the requirements of Sections 3.5 to 3.12 of BD 42 are to be used. Reference to CIRIA 104 is not required as forces and bending moments due to the soil and highway loading are obtained directly from the soil-structure interaction analysis.

**Accidental loading**

Where bridge supports are located within 4.5 m of the edge of a carriageway, BD 60\(^{(22)}\) requires them to be designed to withstand vehicle collision loads. The magnitude and application of this loading are given in that document.

### 3.2.3 Material properties for steel piles

Steel sheet piling, including sections for box piles, is produced in accordance with BS EN 10248\(^{(23)}\); grades S270GP and S355GP are commonly used. Universal Beams (for High Modulus Piles) and other plates and sections are produced in accordance with BS EN 10025\(^{(24)}\); grades S275 and S355 are commonly used. 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). For steelwork designed to BS 5400: Part 3, however, notch toughness is required, although the service temperature for steel in contact with the ground may not be as low as the minimum effective temperature of the bridge deck. In the absence of specific guidance from the Highways Agency, 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 the 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.
3.3 Limit state design

The principal limit states generally considered are:

Ultimate limit state - at which collapse or other form of failure occurs.
Serviceability limit state - the state prior to collapse, beyond which specified serviceability criteria are no longer met.

For retaining walls, there are several possible ultimate limit states that may be reached including:

- failure by forward rotation
- penetration failure
- toe failure
- soil failure
- rupture of anchor
- yielding of sheet pile.

These failure modes are shown in Figure 3.2.

![Failure modes diagram](image-url)

**Figure 3.2** Modes of failure at the ultimate limit state
The serviceability limit state may be reached when there is:

- movement of the retaining structure that may cause collapse or affect the appearance or efficient use of the structure, nearby structures, or services that rely on it
- unacceptable leakage through or beneath the wall
- unacceptable transport of soil grains through or beneath the wall
- unacceptable change to the flow of ground water.

Both the ultimate and serviceability limit states are considered by BD 42 and procedures are defined for implementation in design. These procedures are discussed in Section 13 and individually in the preceding sections.

Limit state design, as set out in BS 5400, requires that the design resistance $R^*$ is at least equal to the design load effects $S^*$. The partial factors to be used are $\gamma_{f1}$ on loads, $\gamma_m$ on material strengths, and $\gamma_{f3}$ to take account of inaccurate assessment, etc. The partial factors are applied in two different ways, depending on the Part of BS 5400 concerned. In Part 3 the design values are expressed as

$$R^* = \frac{\text{function (characteristic strength)}}{\gamma_{f3}\gamma_m}$$

and $S^* = (\text{effects of } \gamma_{f1} \times \text{design loads})$

whereas in Part 4 they are expressed as

$$R^* = \frac{\text{function (characteristic strength)}}{\gamma_m}$$

and $S^* = \gamma_{f3} (\text{effects of } \gamma_{f1} \times \text{design loads})$

Consequently, care has to be exercised in the application of the partial factor $\gamma_{f3}$ when dealing with a mixture of steel and concrete elements. This situation also arises in the design of steel retaining structures, since BD 42 also requires that $\gamma_{f3}$ is applied to the load effects; when that is the case, the factor should be omitted from the calculation of design resistance, even for steel elements.

Design resistances are determined in accordance with BS 5400: Parts 3, 4, and 5 for the steel, concrete, and composite elements respectively. The bridge deck and steel piles are designed to BS 5400: Parts 3 and 5, while the capping beam is designed to BS 5400: Parts 4 and 5.

Design of sheet piling to date has used the elastic section properties of the wall, but there is a move towards using plastic section properties for the ultimate limit state design of sheet pile walls. This is detailed in the new Eurocode 3: Part 5. Development of fully plastic structural capacity of U-section sheet piles (Larssens) is only assured if pairs of sections are crimped or welded at the interlocks, although this has not been UK practice to date. The root problem is whether the ultimate limit state can ever be defined in the soil as well as in the steel; current testing is as yet inconclusive on this. Z profiles such as Frodingham sections do not suffer from this problem. Guidance to determine the appropriate capacity of U sections is given in British Steel's *Piling handbook.*
In driveability assessments, the calculated dynamic stresses from a stress wave analysis program like GRLWEAP are compared with $0.9 f_y$ for the steel grade being used.

### 3.4 Loading and design parameters

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

- soil weight
- earth pressures
- ground water and free water pressures
- seepage forces
- surcharge loads
- bridge structure self weight
- traffic loads including braking loads
- temperature.

Geometric parameters consist of the following:

- 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 designed directly and not to assume that these level variations are included within any factor(s) of safety. For limit states with severe consequences, geometric parameters should represent the most unfavourable or worst credible values that could occur.

### 3.5 Prescriptive measures

In situations where calculation models are not available or not necessary, prescriptive measures can be used. These measures involve conventional or conservative details or the inclusion of new information based on recent research and developments, combined with attention to specification, workmanship, and material control. For example, when durability or corrosion is considered, definition of a limit state can be omitted by choosing prescriptive measures.

The use of prescriptive measures is stated in ENV-1997-1 Eurocode 7.

### 3.6 Observational Method

The Observational Method involves making the best estimate of geotechnical behaviour, in conjunction with formulation of contingency plans for additional measures to be taken if the actual behaviour exceeds predictions by an unacceptable margin. In the construction industry, increasing emphasis is being placed on the value of the Observational Method (see Peck\(^ {22}\)) 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. Among the reasons for this is the difficulty in predicting the soil response to structural loading from a limited number of tests on soil samples, and the general lack of monitoring of real highway structures for correlation.

Where a serviceability limit state deflection is judged by the designer to be really necessary and is specified for a retaining wall, for example where protecting an adjacent building from the effects of the bridge works, then the Observational Method may be the only reliable method to permit control of the works.

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 actual behaviour lies within the acceptable limits.
- A monitoring plan is set up that shows that 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 in design by adopting conservative values of soil properties (see Section 4.3). For larger and more complex structures, however, any over-conservatism may lead to unacceptably high costs.

The Highways Agency is considering the implications of the use of the Observational Method for steel and concrete piled retaining structures and is funding research in this area. The Transport Research Laboratory has started this process for concrete retaining walls but not yet for sheet pile walls. Various studies on the use of the Observational Method for highway structures have been or are in the process of being published by the Transport Research Laboratory and others.

Numerous highway structures have been built using this method, for example the A406 underpass at Neasden and the Limehouse Link tunnel. Operational experience of the use of instrumentation data to verify temporary works design has been documented on the Aldershot Road underpass, Guildford.

Monitoring the behaviour of structures as part of the Observational Method provides a reliable means of evaluating the validity of current design methods, from comparisons of the measured to predicted behaviour.

### 3.7 Design report

Although not a requirement of BD 42 (but a requirement of ENV 1997-1 Eurocode 7), the assumptions, data, calculations, and results of the verification of safety and serviceability of sheet pile retaining wall abutments are to be collated and presented clearly in a design report.
The report should include but not be limited to the following:

- An examination and description of the site.
- Interpretation of the results of ground investigation.
- Analysis of the results of site monitoring of ground water conditions.
- A list of the geotechnical design assumptions.
- Any anticipated geotechnical problems and statements on the contingency actions to be taken.
- Statements on geotechnical requirements for the design and construction of the sheet pile retaining wall, including testing, inspection, and maintenance requirements.
- Stability analyses of the site and calculations for the structural design of the steel sheet pile wall.
4 SOIL DATA FOR DESIGN

This Section provides the designer with an awareness of the soil data that are required to perform an accurate analysis of a steel sheet pile bridge abutment. An appreciation of the methods that are commonly used will also enable appropriate geotechnical surveys to be specified. In some cases, soil properties may not be available initially (concept design prior to site investigation being carried out), or previously determined values may be sufficient for design. Typical values are therefore also presented. BS 8002 is particularly useful for reference on good practice in determining soils data for design and provides guidance values of soil parameters.

4.1 Site investigation

Accurate soil parameters both at and adjacent to the construction site are required for design. These soil parameters must not be determined in isolation, however, but need to be presented with information relating to the physical conditions in the vicinity of the structure. This includes the topography of the site, details of adjacent foundations and services, and the nature of the ground water conditions.

Site investigations are performed to ascertain the character and variability of the strata underlying the site of the proposed retaining wall and adjacent to it. In particular, those properties that could affect the performance of the retaining wall and the choice of the method of construction should be assessed thoroughly. Hence a site investigation should, where possible, be carried out prior to a method of construction being proposed and a design commenced.

Loads on sheet pile walls and their stability are influenced significantly by ground water conditions. Therefore, when appropriate, it is essential that the regimes of the ground water are determined. For waterfront structures this includes the seasonal and tidal variations of water in front of the wall. Ground water exists in many forms including hydrostatic, artesian, perched water conditions, and under drainage.

Although site investigations are important in enabling an accurate design to be produced, the extent and detail of investigation have to reflect the complexity of the soil and water conditions and the type of structure that is proposed. ENV 1997-1 provides classification categories for geotechnical design requirements, which can be adopted in assessing the extent and detail that are required for a particular structure. These categories are defined as Categories 1, 2, and 3 (Section 2.1 of ENV 1997-1).

For some complex cases, site investigations may be required to be performed during construction (see the Observational Method described in Section 3.6 of this publication). Periodic ground inspections during construction enable actual conditions prevailing to be monitored and soil parameters modified as the design is advanced.

Where site investigations are required, they are to be carried out in accordance with BS 5930: 1981 Site investigations and methods of in-situ and laboratory...
testing to BS 1377: 1990 *Methods of tests for soils for civil engineering purposes*: Parts 1 to 92.

4.2 Soil data for design

The soil properties and their parameters that are required for design depend on the type of approach that is to be used for the analysis of the soil and the design of the retaining wall. The fundamental soil properties that are required for any design approach include:

- saturated and unsaturated bulk densities (unit weight) and moisture content
- undrained and drained shear strength including angle of shearing resistance and cohesivity
- Standard Penetration resistance data for end bearing parameters
- soil classification properties.

Deformation and stiffness soil properties include:

- Young’s Modulus
- Poisson’s ratio
- coefficient of horizontal subgrade reaction
- wall friction and adhesion
- over-consolidation ratio (OCR)
- initial coefficient of earth pressure at rest $k_c$.

4.3 Selection and evaluation of soil parameters

The determination of selected values of soil parameters for design needs to be based on the careful assessment of a range of values of each parameter that might govern the performance of the retaining wall during its design life, with account taken of the conditions representative of the ground and the nature of the environment.

The assessment of appropriate parameters is often dependent on the mechanism or mode of deformation being considered. For sheet pile abutments, strain levels and compatibility need to be considered in the assessment of strengths in materials through which a presumed failure surface can occur. Ranges of values may also be required, particularly if the soil parameter magnitudes are likely to change during the lifetime of the bridge abutment. Typically for sheet pile abutments embedded in clays it is necessary to obtain soil parameters both for short- and long-term conditions. This requires that soil parameters for drained and/or undrained conditions are obtained.

BD 42 makes specific reference to CIRIA Report 104 for definitions relating to soil properties to be used in design, but data are presented only for stiff clays. Reference is also made to BS 8002 but only in a general manner. In BS 8002, however, data relating to soil parameters are presented for both granular and cohesive soils. Matters are complicated further in that CIRIA Report 104 and BS 8002 each have distinct definitions for soil properties. Sections 4.3.1 and
4.3.2 present these definitions to emphasise how important it is to define precisely what is to be measured during the site investigation.

Where soil parameters are obtained from sources that quote typical values, the designer must be confident that the values used are compatible with the design methods or procedures being implemented. One of the principal areas of inaccuracy in over-consolidated clays and compacted engineered fills lies in the determination of the initial earth pressure at rest coefficient $k_o$. Guidance on this is now given in BA42 and in the SCI publication *Integral steel bridges: Design guidance*. The value of the earth pressure coefficients will change as wall displacement occurs (see Figure 3.1) and this relationship should be established for the design of steel sheet pile walls of significant height ($> 4.5$ m).

### 4.3.1 Soil parameters based on CIRIA Report 104

CIRIA Report 104 takes into account the uncertainty in the selection of soil strength parameters for stiff clays 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 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".

Although the *moderately conservative* soil parameter is used from the basis of experience to date, 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 4.1.

![Figure 4.1](image_url)  
*Figure 4.1 Representation of moderately conservative and worst credible soil parameters*
4.3.2 Soil parameters based on BS 8002

BS 8002 defines how the properties for both cohesive and cohesionless soils should be obtained. Soil properties are based on representative values and are defined as “conservative estimates of the properties of the soil as it exists insitu”. In this case conservative is defined 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.” The definition given in BS 8002 could be assumed to be similar to that defined as a worst credible value in CIRIA Report 104. This is substantiated by Akroyd(10).

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, however, then the representative value is to be a cautious assessment of the lower or upper bound of the acceptable data.

4.4 Typical soil unit weights

The design of permanent earth retaining structures generally involves an effective stress analysis (see Section 4.5), where soil unit weights are required in the calculation. Wherever possible, unit weights are obtained from site investigations, borehole logs, in-situ tests, and laboratory tests. As unit weight can be determined with confidence, in most cases it is taken to be a mean value. Where the parameter has not been measured, however, it is necessary to refer to typical values. This is common when conceptual or preliminary designs are undertaken before soil investigations have been carried out. Table 4.1 gives mean values of bulk unit weight for soils and fills, as given in CIRIA SP95(33) and BS 8002.

4.5 Soil strength parameters

4.5.1 Effective stress and total stress principles

Effective stress and total stress principles need to be understood such that appropriate soil parameters can be measured and used in geotechnical design. The appreciation of these principles is most important in cohesive soils with low permeability where pore water pressures are dissipated slowly. For cohesionless soils where permeability is high and pore water pressures are dissipated quickly, measurement of soil properties is more straightforward.

Cohesive, very fine-grained, relatively impermeable soils adjust slowly to changes in loads and differences in water levels. In the short term, a cohesive soil exhibits a shear strength that is dependent on the locked-in effective stress. The effective stress cannot change without an increase or decrease in volume occurring (a change in water content). This strength in the short term is independent of any recently applied loads or changes in hydraulic conditions, because of the relatively impermeable nature of cohesive soils. The immediate strength is called the apparent cohesion or undrained shear strength \( c_u \) of the soil. Over a period of time, water is squeezed out of, or is drawn into, a cohesive soil and, as the ground adjusts to a new set of conditions, this undrained shear strength changes. Thus where cohesive soils are present, only total stress undrained shear strength
### Table 4.1  Typical unit weight of soils and fills

<table>
<thead>
<tr>
<th>Type of material</th>
<th>Dry (kN/m³)</th>
<th>Saturated (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Loose</td>
<td>Dense</td>
</tr>
<tr>
<td>Gravel</td>
<td>16</td>
<td>18</td>
</tr>
<tr>
<td>Well graded sand and gravel</td>
<td>19</td>
<td>21</td>
</tr>
<tr>
<td>Coarse or medium sand</td>
<td>16.5</td>
<td>18.5</td>
</tr>
<tr>
<td>Well graded sand</td>
<td>18</td>
<td>21</td>
</tr>
<tr>
<td>Fine or silty sand</td>
<td>17</td>
<td>19</td>
</tr>
<tr>
<td>Rock fill and quarry waste</td>
<td>15</td>
<td>17.5</td>
</tr>
<tr>
<td>Brick hardcore</td>
<td>13</td>
<td>17.5</td>
</tr>
<tr>
<td>Slag fill</td>
<td>12</td>
<td>15</td>
</tr>
<tr>
<td>Ash fill</td>
<td>6.5</td>
<td>10</td>
</tr>
<tr>
<td>Topsoil</td>
<td>16</td>
<td>19</td>
</tr>
<tr>
<td>River mud</td>
<td>14.5</td>
<td>17.5</td>
</tr>
<tr>
<td>Silt</td>
<td></td>
<td>18</td>
</tr>
<tr>
<td>Very soft clay</td>
<td>16</td>
<td>16</td>
</tr>
<tr>
<td>Soft clay</td>
<td>17</td>
<td>17</td>
</tr>
<tr>
<td>Firm clay</td>
<td>18</td>
<td>18</td>
</tr>
<tr>
<td>Stiff clay</td>
<td>19</td>
<td>19</td>
</tr>
<tr>
<td>Very stiff clay or hard clay</td>
<td>20.0-21.0</td>
<td>20.0-21.0</td>
</tr>
</tbody>
</table>

Soil properties are measured and used in design for short-term conditions. This is most relevant in temporary works design.

In the long term, when the shear stresses and total stresses due to construction have been applied, all volume changes due to consolidation or swelling have occurred, and the ground water in the soil has reached an equilibrium level (the pore water pressures have dissipated), effective stress strength parameters apply.

For clays, the definitions of long term or short term depend very much on the permeability of the clay strata. The valid period for an undrained, total stress condition can vary from a few days to several months. Often, clay strata are laminated with bands of sand or silt that can greatly shorten the draining time that would apply if the whole soil mass were pure clay. In such circumstances, drained effective stress soil strength parameters would be more appropriate than total stress undrained soil strength parameters. For stiff clays, long-term effective soil strength parameters would be applicable, because they would be more critical for design than total stress soil strength parameters. For soft clays, the reverse often applies. If any doubt exists about which will prove critical then both total stress and effective stress soil strength parameters should be determined and the most critical used in design.

For cohesionless soils, pore water pressures dissipate very quickly in the short term, hence soil properties are based on effective stress conditions being relevant for all stages of construction and service (i.e. in the short term and long term both for temporary and permanent works).
4.5.2 Effective stress soil strength properties

Coulomb was responsible for the development of earth-pressure theory in the 18th Century. He divided the strength of a soil into two components, namely cohesion, where the strength is independent of applied forces and a function only of the area of rupture, and friction, which is proportional to the compressive force on the rupture plane. This concept, although developed by Coulomb in terms of total stress, was modified in terms of effective stress by Terzaghi, and remains the basis of soil-strength theory to this day.

Terzaghi's principle of effective stress is central to an understanding of soil strength. In all soils, whether they are sands, gravels, silts, or clays, shear strength is derived mainly from internal friction characterised by the effective angle of shearing resistance $\phi'$.

Soil strength is described by the modified Coulomb equation, which presents the behaviour of the soil in terms of effective stress parameters $c'$ and $\phi'$ as given by the relationship:

$$\tau = c' + \sigma' \tan \phi'$$

where $\tau$ is the shear strength of the soil
$c'$ is the effective stress cohesion strength of the soil
$\phi'$ is the effective stress angle of shearing resistance of the soil
$\sigma'$ is the vertical effective stress where $\sigma'$ is given by:

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

and $\sigma$ is the total stress
$u$ is the pore water pressure.

The failure envelope, as presented by the relationship given by Terzaghi above, is shown diagrammatically in Figure 4.2.
Cohesive soils

Soil parameters for drained conditions can be determined in accordance with BS 1377: Part 7 and 8[34]. Effective strength parameters are obtained by carrying out a number of consolidated undrained triaxial compression tests with pore water pressure measurement. The tests are carried out sufficiently slowly to ensure equalisation of pore water pressures during the test. The effective stress angle of shearing resistance friction $\phi'$ of cohesive soils does not vary widely, but the effective stress cohesion strength intercept $c'$ (which has a relatively large effect on the calculated long-term earth pressures) may be very different from one soil to another. For normally consolidated (soft) clays, $c'$ is taken to be zero. As the soil becomes more heavily consolidated, $c'$ is found to increase.

Typically for clays

$$0 < c' < 100 \text{ kN/m}^2$$

and

$$13^\circ < \phi' < 25^\circ$$

It is found that the failure envelope for over-consolidated clays is not a straight line but is curved, due to dilatancy effects at low stress levels (see Figure 4.3). To represent the behaviour and the corresponding parameters of the soil adequately, it is therefore necessary to define a peak angle of shearing resistance $\phi'_p$ and a critical-state angle of shearing resistance $\phi'_{\text{crit}}$. See Section 2.2.3 of BS 8002 for a detailed description of these definitions.

![Figure 4.3](image)

**Figure 4.3** Failure envelope for over-consolidated clays

BS 8002 requires that representative (worst credible) values of both the peak and critical soil strength parameters $\phi'_p$ and $\phi'_{\text{crit}}$ 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 CIRIA Report 104.
Where no test data are available, a preliminary value of the angle of shearing resistance \( \phi' \) can be estimated from the Plasticity Index of the soil (see CIRIA Report 104 and BS 8002); see Table 4.2, which is reproduced from BS 8002. It is unwise to assume an effective cohesion intercept of more than 10 kN/m\(^2\), even if the soil is heavily consolidated, and it is usual to assume \( c' = 0 \) in most cases.

### Table 4.2 Angle of shearing resistance derived from Plasticity Index (CIRIA Report 104)

<table>
<thead>
<tr>
<th>Plasticity Index (%)</th>
<th>15</th>
<th>20</th>
<th>25</th>
<th>30</th>
<th>40</th>
<th>50</th>
<th>80</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \phi' ) (degrees)</td>
<td>30</td>
<td>28</td>
<td>27</td>
<td>25</td>
<td>22</td>
<td>20</td>
<td>15</td>
</tr>
</tbody>
</table>

The measurements that are carried out to obtain soil strength properties (as described above) are almost always based on triaxial compression tests, however in reality the earth pressure problem is one of plane strain. The difference in soil strength properties that are obtained, though, is slight (approximately 10%), hence the properties measured from triaxial tests can be used with confidence since they are on the safe side.

Valuable laboratory data for stiff clays encountered in the UK (e.g. London Clay, Lower Oxford Clay) can also be obtained from reliable sources. A useful source of general references is given by Cripps and Taylor\(^{(35)}\). Again caution should be exercised in applying soil strength parameters given in the references to a design in the absence of a proper investigation and testing programme, but the references nevertheless provide a useful background.

**Cohesionless soils**

The effective stress soil strength parameters for cohesionless soils are a function of particle size distribution, soil density, imposed stress level, angularity, and cementation. They are obtained indirectly from laboratory testing (drained shear box or drained triaxial tests) of recompacted disturbed samples obtained from site investigation boreholes or by interpretation of in-situ static or dynamic penetration tests (Standard Penetration or static cone test).

Alternatively, \( \phi' \) for granular soils can be estimated from previous work or from generalised equations related to classification tests. For example, investigation by Peck \textit{et al.}\(^{(36)}\) enables the angle of shearing resistance \( \phi' \) to be estimated from Standard Penetration Tests (see Figure 4.4).

The strength of cohesionless soils is based on effective stress conditions being present (both in the short term and long term) and given by the effective stress angle of shearing resistance \( \phi' \). The cohesive intercept \( c' \) is considered to be zero for cohesionless soils.
Where measured soil strength parameters are not available, typical soil parameters can be used as a preliminary guide. BS 8002 presents peak and effective angles of shearing resistance $\phi'_p$ and $\phi'_{	ext{crit}}$ (degrees), which are based on generalised equations and are given respectively by:

$$\phi'_p = 30 + A + B + C$$

$$\phi'_{	ext{crit}} = 30 + A + B$$

$A$, $B$, and $C$ are factors that take into account the angularity of the particles, the grading of the sand/gravel, and results of Standard Penetration Tests (for further details see BS 8002, table 3).

CIRIA Report SP95 provides typical values for soil strength properties as a preliminary guide for a range of granular soils and fills, presented here as Table 4.3.

**Table 4.3 Effective stress angle of shearing resistance for granular soils and fill (CIRIA Report SP95)**

<table>
<thead>
<tr>
<th>Type of material</th>
<th>Effective stress $\phi'$ (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Loose</td>
</tr>
<tr>
<td>Gravels</td>
<td>35</td>
</tr>
<tr>
<td>Sands</td>
<td>30</td>
</tr>
<tr>
<td>Silts</td>
<td>24</td>
</tr>
<tr>
<td>Clayey silts</td>
<td>21</td>
</tr>
<tr>
<td>Silty clays</td>
<td>15</td>
</tr>
<tr>
<td>Rock fill and quarry waste</td>
<td>40</td>
</tr>
<tr>
<td>Brick hardcore</td>
<td>40</td>
</tr>
<tr>
<td>Slag fill</td>
<td>30</td>
</tr>
<tr>
<td>Ash fill</td>
<td>35</td>
</tr>
</tbody>
</table>
Rock

The engineering properties of rock relevant in design are controlled by the extent and orientation of the bedding planes and joints within the rock mass together with the water pressures on the discontinuity planes. The following are indicative values of the effective stress angle of friction of rocks, which can conservatively be treated as composed of granular fragments. Values are taken from BS 8002 and are presented in Table 4.4.

Table 4.4 Angle of effective shearing resistance \( \phi' \) for rocks

<table>
<thead>
<tr>
<th>Stratum</th>
<th>( \phi' ) (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chalk</td>
<td>35</td>
</tr>
<tr>
<td>Clayey marl</td>
<td>28</td>
</tr>
<tr>
<td>Sandy marl</td>
<td>33</td>
</tr>
<tr>
<td>Weak sandstone</td>
<td>42</td>
</tr>
<tr>
<td>Weak siltstone</td>
<td>35</td>
</tr>
<tr>
<td>Weak mudstone</td>
<td>28</td>
</tr>
</tbody>
</table>

4.5.3 Total stress soil strength properties

Total stress soil strength parameters for undrained conditions are obtained from triaxial compression tests on undisturbed samples obtained from boreholes. These parameters can be determined in accordance with BS 1377: Parts 7 and 8. The undrained shear strength \( c_u \) of a cohesive material is obtained from unconsolidated (quick) undrained triaxial tests.

Commonly, the undrained strength of a soft clay with a small over-consolidation ratio (less than 3) is required because its minimum strength occurs in the short-term undrained state. Where for temporary works construction the undrained strength of a stiff over-consolidated clay is to be relied upon, it is necessary to consider very carefully the vulnerability of the soil due to drainage paths present in the soil. This is particularly so for stiff clays with sand layers or pre-existing fissures, load-induced shear ruptures, or tensile cracks that can initiate failure patterns that are more critical than the theoretical homogeneous assumptions.

For a clay in the undrained state, its angle of shearing resistance \( \phi \) is taken to be zero and the cohesion intercept \( c \) is equal to the undrained shear strength \( c_u \) (see Figure 4.5).

![Figure 4.5 Failure envelope for undrained conditions](image)

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In the absence of test data, and for small structures only, BS 8004 provides approximate values of the undrained shear strength $c_u$ obtained from sample description (see Table 4.5).

**Table 4.5 Undrained shear strength of clays (BS 8004)**

<table>
<thead>
<tr>
<th>Consistency of clay</th>
<th>Field indications</th>
<th>Undrained shear strength $c_u$ (kN/m$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS 5930 Widely used</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Very stiff</td>
<td>Very stiff or hard</td>
<td>$&gt;150$</td>
</tr>
<tr>
<td></td>
<td>Brittle or very tough</td>
<td></td>
</tr>
<tr>
<td>Stiff</td>
<td>Stiff</td>
<td>100-150</td>
</tr>
<tr>
<td></td>
<td>Cannot be moulded in the fingers</td>
<td></td>
</tr>
<tr>
<td>Firm to stiff</td>
<td></td>
<td>75-100</td>
</tr>
<tr>
<td>Firm</td>
<td>Firm</td>
<td>50-75</td>
</tr>
<tr>
<td></td>
<td>Can be moulded in the fingers by strong pressure</td>
<td></td>
</tr>
<tr>
<td>Soft to firm</td>
<td></td>
<td>40-50</td>
</tr>
<tr>
<td>Soft</td>
<td>Soft</td>
<td>20-40</td>
</tr>
<tr>
<td></td>
<td>Easily moulded in the fingers</td>
<td></td>
</tr>
<tr>
<td>Very soft</td>
<td>Very soft</td>
<td>$&lt;20$</td>
</tr>
<tr>
<td></td>
<td>Exudes between the fingers when squeezed in the fist</td>
<td></td>
</tr>
</tbody>
</table>

4.6 Soil deformation parameters

Soil deformation parameters are required for analyses of wall and ground displacements.

Reliable measurements of soil deformation parameters can be difficult to obtain from field or laboratory tests. Hence, where possible, soil deformation parameters should be assessed from back analysis of the behaviour of previous work in similar ground conditions.

The stress-strain behaviour of soil is generally nonlinear (see Figure 4.6), however it is often convenient in design to assume a linear (or log-linear) relationship between stress and strain for the behaviour of the soil within the range of strain being modelled. In practice, Young’s Modulus for a soil is defined as a line passing through the origin and the stress-strain curve at the maximum strain being considered. It is therefore important where laboratory tests have been performed, that a stress-strain curve is obtained. This curve enables a design engineer to make the most of the information that is available, and choose a value for Young’s Modulus that is relevant for the range of strain under consideration.

Where strata of different soils are present, the designer must know the basis for the Young’s Modulus value quoted. This is important because the ultimate condition for each individual soil can occur at significantly different strain levels. For example, the strain level at failure for a non-fissured clay may be 4%, while for a fissured clay it can reach 10%. For a sand or gravel, the strain level at failure can be even higher, typically approaching 20%.
4.6.1 Typical values of Young’s Modulus

Several methods are available for determining Young’s Modulus of a soil. These include laboratory tests (unconfined compression tests and triaxial compression tests) and in-situ tests [Standard Penetration Tests (SPT), Cone Penetration Tests (CPT), pressuremeter, and plate-load tests]. As the laboratory tests are expensive and the confidence in the results questionable, the in-situ SPT and CPT tests have been much used to obtain Young’s Modulus of soils. As a preliminary guide, the information given by Bowles\(^{(37)}\), in which equations for Young’s Modulus are presented, should be used. This information is summarised in Table 4.6.

Borin\(^{(38)}\) also gives values for Young’s Modulus of soils, and these are presented in Tables 4.7 and 4.8. Table 4.7 presents modulus values for normally consolidated cohesionless soils that vary linearly with depth. This is based on the assumption that Young’s Modulus of normally consolidated cohesionless soil is proportional to the vertical effective stress, which is directly proportional to depth. Table 4.8 is specific to cohesive soils, and is based on undrained shear strength. Both these tables are commonly used to obtain a value of Young’s Modulus, which is used in computer software analysis tools.

4.6.2 Poisson’s ratio

Borin\(^{(38)}\) provides values of Poisson’s ratio for soils that may be used for design. These values are presented in Table 4.9.

4.6.3 Deformation parameters for stiffness models
(subgrade reaction)

In the subgrade reaction model, the stiffness of the soil is characterised by a coefficient of horizontal subgrade reaction \(k_h\), which is usually expressed in terms of force/area/displacement, or pressure per unit displacement, which is a spring constant.
Table 4.6  *Equations for Young’s Modulus (kPa) based on in-situ test methods*

<table>
<thead>
<tr>
<th>Soil</th>
<th>SPT</th>
<th>CPT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand (normally consolidated)</td>
<td>$E_s = 500(N+15)$</td>
<td>$E_s = 2$ to $4q_c$</td>
</tr>
<tr>
<td></td>
<td>$E_s = (15000$ to $22000)ln N$</td>
<td>$E_s = (1+D_i)q_c$</td>
</tr>
<tr>
<td></td>
<td>$E_s = (35000$ to $50000)log N$</td>
<td></td>
</tr>
<tr>
<td>Sand (saturated)</td>
<td>$E_s = 250(N+15)$</td>
<td></td>
</tr>
<tr>
<td>Sand (over-consolidated)</td>
<td>$E_s = 18000+750N$</td>
<td>$E_s = 6$ to $30q_c$</td>
</tr>
<tr>
<td>Gravelly sand and gravel</td>
<td>$E_s = 1200(N+6)$</td>
<td></td>
</tr>
<tr>
<td>Clay</td>
<td>$E_s = 320(N+15)$</td>
<td></td>
</tr>
<tr>
<td>Silty sand</td>
<td>$E_s = 300(N+6)$</td>
<td></td>
</tr>
<tr>
<td>Soft clay</td>
<td>-</td>
<td>$E_s = 3$ to $6q_c$</td>
</tr>
<tr>
<td></td>
<td>Using the undrained shear strength $c_u$ in units of $c_u$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$I_p &gt; 30$ or organic</td>
<td>$E_s = 100$ to $500c_u$</td>
</tr>
<tr>
<td></td>
<td>$I_p &lt; 30$ or stiff</td>
<td>$E_s = 500$ to $1500c_u$</td>
</tr>
<tr>
<td></td>
<td>$E_{(OCR)} = E_{(OCR)^t}$</td>
<td></td>
</tr>
</tbody>
</table>

$N$ is the SPT blow count  
$q_c$ is the value measured in a Cone Penetration test  
$OCR$ is the over-consolidation ratio  
$I_p$ is the Plasticity Index  
nc means normally consolidated

Table 4.7  *Young’s Modulus for normally consolidated cohesionless soils*\(^{(39)}\)

<table>
<thead>
<tr>
<th>Relative density of soil</th>
<th>Loose</th>
<th>Medium</th>
<th>Dense</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry or moist sand</td>
<td>880</td>
<td>2800</td>
<td>7100</td>
</tr>
<tr>
<td>Submerged sands</td>
<td>560</td>
<td>1800</td>
<td>4600</td>
</tr>
</tbody>
</table>

Table 4.8  *Young’s Modulus for cohesive soils*

<table>
<thead>
<tr>
<th>Consistency</th>
<th>Undrained shear strength $c_u$ (kN/m$^2$)</th>
<th>Young’s Modulus $E_s$ (MN/m$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft clay</td>
<td>20-40</td>
<td>1.6-6.0</td>
</tr>
<tr>
<td>Firm clay</td>
<td>40-75</td>
<td>6.0-20.0</td>
</tr>
<tr>
<td>Stiff clay</td>
<td>75-100</td>
<td>20.0-75.0</td>
</tr>
</tbody>
</table>

Table 4.9  *Poisson’s ratio for soils*

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Poisson’s ratio $\nu_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cohesionless soils</td>
<td>0.2-0.3</td>
</tr>
<tr>
<td>Drained over-consolidated clays and unloading of normally consolidated clays</td>
<td>0.1-0.2</td>
</tr>
<tr>
<td>Undrained clays</td>
<td>0.5$^*$</td>
</tr>
</tbody>
</table>

* 0.5 is the theoretical value, however for analysis a value of 0.49 is used.*
Coefficients of horizontal subgrade reaction for typical soils for use in design are recommended by Terzaghi (1955)\(^\text{39}\).

### 4.7 Wall friction and adhesion properties

Wall friction and wall adhesion are components of shear strength between the back of the wall and the soil it supports. The value for the angle of wall friction \(\delta'\) is often expressed as a function of the effective angle of shearing resistance of the soil \(\phi'\), or of the angle of skin friction between the soil and the wall material \(\delta_s\). The wall adhesion \(c_w'\) is expressed as a function of the effective cohesivity of the soil \(c'\). \(\delta'\) and \(c_w'\) are not material properties, however, but depend on the relative movement between the soil and the retaining wall at that limit state, which in turn is a function of the limiting loading and foundation conditions. They are mobilised properties that have a value between zero and the limiting values given by the material properties.

In design, a constant value of \(\delta'\) is assumed, although wall friction is not mobilised uniformly throughout the length of a retaining wall. Also different values of \(\delta'\) are required for active and passive soil movements.

BD 42 requires that CIRIA Report 104 is used to determine wall friction where the maximum value of \(\delta'\) for the active and passive zone should not exceed \(\%\phi'\) and \(\%\phi'\) respectively. The range of allowable angles of wall friction \(\delta'\), however, varies noticeably based on the Standard or code of practice used. For instance, BS 8002 states that a design value of \(\delta'\) for active and passive zones should be \(\%\phi'\) for effective stress analyses, while the British Steel Piling handbook recommends that \(\delta'\) is to be taken as zero for the active zone and \(\%\phi'\) for the passive zone.

It is recommended that the maximum values for wall friction to be used for the design of sheet pile bridge abutments are those stated in CIRIA 104 (see Table 4.10).

**Table 4.10 Maximum mobilised angle of wall friction**

<table>
<thead>
<tr>
<th></th>
<th>Active</th>
<th>Passive</th>
</tr>
</thead>
<tbody>
<tr>
<td>(%\phi')</td>
<td>(%\phi')</td>
<td></td>
</tr>
</tbody>
</table>

CIRIA Report 104 also recommends maximum values of wall adhesion that should not be exceeded in design. These values are presented in Table 4.11.

**Table 4.11 Maximum mobilised values of wall adhesion**

<table>
<thead>
<tr>
<th></th>
<th>Active</th>
<th>Passive</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective stress</td>
<td>Total stress</td>
<td>Effective stress</td>
</tr>
<tr>
<td>0</td>
<td>(0.5c_w) but (\leq 50 \text{kN/m}^2)</td>
<td>0</td>
</tr>
</tbody>
</table>
5 LIMIT EQUILIBRIUM METHOD

This Section describes the manner in which earth pressures adjacent to retaining walls are determined, based on the simplistic limit equilibrium theory. It presents commonly used methods that can be applied to analyse and design embedded retaining wall bridge abutments.

The limit equilibrium analysis is performed assuming 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.

With the development of analysis techniques that have striven to satisfy the fundamental theoretical requirements to simulate soil-structure behaviour, newer soil-structure interaction methods have been introduced, which are becoming more widely used by designers. The soil-structure interaction methods are discussed in Section 6.

5.1 Representation of earth pressures

Earth pressures and the manner in which they are mobilised are fundamental to the analysis of embedded steel sheet pile retaining walls. The lateral pressures that act on embedded sheet pile abutments are a function of:

- the materials and surcharges that the abutment must support
- the ground water and foundation conditions
- the mode and magnitude of movement that the wall undergoes as a result of soil-structure-foundation interaction.

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 the ratio of the effective horizontal stress to the effective vertical stress at any depth below the soil surface (see Figure 5.1). The coefficient may be expressed as:

\[
k = \frac{\sigma_h}{\sigma_v}
\]

Earth pressures in the ground are in an initial or in-situ state, commonly referred to as the at-rest state. As an embedded sheet pile wall moves towards a collapse or limiting state, zones are formed in the adjoining ground within which the soil is in a state of plastic equilibrium. A simplistic representation of these zones is shown in Figure 5.2. Within these zones, the direction of shear is such that the soil thrust on the wall is a minimum within the active zones (where movement of the wall is away from the soil), and a maximum within the passive zones (where movement of the wall is towards the soil).
The stresses acting on an element of soil within a soil mass (shown in Figure 5.1) may be represented graphically by the Mohr coordinate system in terms of the shear stress \( \tau \) and the effective normal stress \( \sigma' \) (see Figure 5.3). In this system, the state of stress is indicated by a circle, known as the Mohr circle, constructed for the soil element.

There are critical states of stress at which the shear stress reaches the shear strength of a soil. The envelope of such states is known as the Mohr envelope of failure. If the Mohr circle lies within the failure envelope, the shear stresses in all directions are less than the shear strength of the soil. If the circle touches the failure envelope, the shear strength is fully mobilised along a plane within the soil element and a state of plastic equilibrium is considered to have been reached.

Figure 5.3 shows three states of stress for a soil, the at-rest, and the limiting conditions of active and passive states. The earth pressure coefficients are defined as \( k_o \), \( k_a \), and \( k_p \).
Failure, envelope h, Passive state = k, d, -v

Effective stress u'

Active - Passive state

Figure 5.3 At-rest, active, and passive states of stress

5.1.1 At-rest earth pressure profiles

The at-rest state is assumed to act where the soil has not undergone any lateral strain, either because it has not been disturbed or because it has been prevented from expanding or contracting laterally with changes in vertical stresses. At-rest conditions normally exist in the ground behind a retaining structure when the movement of the structure is less than 0.05% of the height of the abutment (H) for a normally consolidated soil.

The earth pressure at rest is a function of the shear strength of the soil, its stress-strain history, and weathering history, and is of importance in assessing the amount of deformation required for the soil mass to reach the active or passive state (see Figure 3.1).

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

\[ \sigma_h = k_o \sigma_v' \]

where \( k_o \) is the at-rest pressure coefficient

\( \sigma_v' \) is the effective vertical stress.

For normally consolidated soils that have not been subjected to removal of overburden or to activities that have resulted in lateral straining of the ground, \( k_o \) can be obtained from an approximate expression developed by Jaky, where

\[ k_o = 1 - \sin \phi' \]

and \( \phi' \) is the effective angle of shearing resistance of the soil.

This expression is applicable for normally consolidated clays and sands and can also be applied to fill materials (\( c' = 0 \)).

For a cohesionless soil with \( \phi' = 30^\circ \), \( k_o = 0.5 \), which is noticeably higher than the active value (\( k_a = 0.33 \)), i.e. by 50%.
The at-rest earth pressure coefficient for a lightly over-consolidated clay $k_{o,oc}$ can be obtained from the relationship given by the Canadian Geotechnical Society\(^{[41]}\), where

$$k_{o,oc} = (1 - \sin \phi') \, OCR^{0.5}$$

and $OCR$ is the over-consolidation ratio of the soil, which is given by

$$OCR = \frac{\sigma_{v,oc}'}{\sigma_v'}$$

where $\sigma_{v,oc}'$ is the effective vertical stress in the over-consolidated state.

For soils with complex stress histories, the distribution of $k_o$ with depth should be investigated carefully (Burland \textit{et al.}\(^{[42]}\)). 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.

### 5.1.2 Active and passive earth pressure coefficients

Simplistic mathematical representations of earth pressure coefficients are those determined originally by Rankine and Coulomb. The Rankine theory is based on the assumption that the wall introduces no changes in the shearing stresses at the surface of contact between the wall and the soil, i.e. the wall is smooth and no wall friction is present. For a horizontal retained surface and no wall friction, the active and passive earth pressure coefficients $k_a$ and $k_p$ are given by:

$$k_a = \frac{1 - \sin \phi'}{1 + \sin \phi'} \quad \text{and} \quad k_p = \frac{1 + \sin \phi'}{1 - \sin \phi'}$$

Coulomb’s theory considers the limiting equilibrium of a wedge of soil failing on planar surfaces. Hence a force on the wall is calculated rather than the pressure distribution. The angle of wall friction $\delta$ mobilised at limit state has to be assumed for this theory. The expressions for the active and passive earth pressure coefficients $k_a$ and $k_p$ for this method are given by:

$$k_a = \left[ \frac{\cos \phi'}{1 + \sqrt{\frac{\sin (\phi' + \delta) \sin \phi'}{\cos \delta}}} \right]^2$$

and

$$k_p = \left[ \frac{\cos \phi'}{1 - \sqrt{\frac{\sin (\phi' + \delta) \sin \phi'}{\cos \delta}}} \right]^2$$

The Rankine approach underestimates passive earth pressure magnitudes, while the use of the Coulomb theory can result in significant overestimation.

Following on from these original theories by Rankine and Coulomb, further analytical solutions have been developed and earth pressure coefficients taking into account wall friction etc. are also available. These refined earth pressure...
coefficients are presented in graphical or tabular form. Commonly quoted earth pressure coefficients in codes of practice and design manuals include those from Kerisel and Absi\cite{43}, Caquot and Kerisel\cite{44}, and Sokolovsky\cite{45}. Caquot and Kerisel curves for $k_a$ and $k_p$ for a horizontal ground level are shown in Figure 5.4.

![Figure 5.4 Caquot and Kerisel curves for $k_a$ and $k_p$](image)

5.1.3 Effect of wall friction and adhesion

In order to mobilise active and passive states of stress, the retaining wall and the adjacent soil must move. In doing so, the soil in the active zone will be subject to lateral extension and there will be a tendency for the soil to settle relative to the retaining wall. On the passive soil, the soil will compress laterally and will tend to move upwards relative to the retaining wall. As the surfaces of most retaining walls are rough, this relative movement between the soil and the structure will mobilise shear stresses (see Figure 5.5).
The movement also has an effect on the magnitude of the minimum and maximum horizontal earth pressures. For the minimum or active limit state, wall friction or adhesion will decrease the horizontal earth pressure slightly. On the other hand, for the maximum or passive limit state, wall friction or adhesion may increase the horizontal earth pressure significantly depending on its magnitude. Wall friction values that are to be used for design are discussed further in Section 4.7.

5.2 Determining lateral earth pressures

Analytical expressions using the limit equilibrium approach estimate the stresses under limiting active and passive states assuming that the pressure profiles increase linearly with depth. Figure 5.6 shows the lateral earth pressure distribution for a cantilever and a propped embedded wall.

Limiting earth pressures for cantilever and anchored retaining walls can be presented by generalised equations. Equations presented below are for long-term, drained effective stress analysis and for short-term, undrained total stress analysis.

5.2.1 Long-term, drained effective stress analysis case

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

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

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

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 5.6  *Active and passive pressure distributions for cantilever and anchored embedded retaining wall abutments*

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

- \( c_w \) 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 the following relationships:

\[
k_{ac} = 2 \sqrt{k_a \left( 1 + \frac{c_w}{c'} \right)}
\]

\[
k_{pc} = 2 \sqrt{k_p \left( 1 + \frac{c_w}{c'} \right)}
\]
5.2.2 Short-term, undrained total stress analysis

If a short-term, undrained total stress analysis is undertaken, the generalised horizontal active and passive earth pressures are given by:

\[ \sigma_a = (q + \gamma z) - c_u k_{ac} \]
\[ \sigma_p = (q + \gamma z) - c_u k_{pc} \]

and

\[ k_{ac} = k_{pc} = \frac{1}{2} \sqrt{1 + \frac{c_w}{c_u}} \]

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.

5.2.3 Representation of layered soils

The lateral earth pressure equations presented above assume that the lateral pressures increase linearly with depth. In nearly all cases, however, layered soils are present in the ground. To model the lateral pressure distribution for layered soils, the soil pressure at the interface of each stratum is calculated by modifying the common overburden pressure at the interface by the soil pressure coefficient relative to the soil immediately above and below the interface. This produces a change in the soil pressures on the wall at the interface of each stratum (see Figure 5.7).
Figure 5.8). Such sudden changes in lateral pressures are unlikely to occur in practice.

Figure 5.8  Earth pressure distribution for layered soils

5.2.4 Trial wedge method

The trial wedge method is used when the methods described above are not appropriate to the situation. These include for example:

- where an irregular ground surface is present
- where there is ground water seepage
- where there are concentrated loads and surcharges present
- where non-horizontal soil interfaces exist in multi-layer material.

As this is a graphical method, it has the advantage that the significance of the various factors involved can be appreciated readily in the calculation process, however the method still has limitations in determining passive earth pressures. Usually this method is performed using a computer program such as ReWaRD (see Section 5.5).

5.3 Earth pressure due to surcharges

Surcharges applied to the retained ground surface increase the earth pressures acting on an abutment. Surcharges can be permanent or temporary and can be classified into uniformly-distributed loads and point loads. The behaviour of uniform vertical surcharges is presented below. For the other less common types of vertical surcharges such as line loads and point loads, reference can be made to Foundation design by Teng\(^{46}\) and CIRIA Report 104. More recent information on the effects of surcharges can be obtained from the Institution of Structural Engineers report Soil structure interaction - the real behaviour of structures\(^{12}\).

For the design of bridge abutments, only uniformly-distributed surcharges acting on the retained side of the retaining wall are considered as they comprise the main form of loading from the road above.

In the case of a uniformly-distributed surcharge \(q\) acting on the retained soil, the horizontal stress induced is given by \(k_q\) (see Figure 5.9).
Surcharge pressures are included in the generalised active and passive pressure equations presented in Section 5.2.

In the majority of cases, surcharge loads consist of the vertical superimposed loads acting on the active soil wedge. These include:

- the movement of road and rail vehicles including cranes
- the storage of transported goods.

For road and rail traffic loading see document BD 37.

Where it is assumed there are no traffic or storage loads acting on the retained side, there is a requirement in BS 8002 that a minimum surcharge load of 10 kN/m² is applied.

### 5.4 Earth pressure due to compaction

Where the bridge deck level is to be above existing ground level, it is necessary to backfill sheet pile abutments with compacted soil (see Figure 5.10). Compaction to a minimum density is often specified to ensure that it has adequate strength and stiffness such that future self settlement of the fill and consequential surface settlement are prevented. The compaction process involves the use of heavy compaction plant near the abutment where typically rollers weighing 5-10 tonnes are used. This can have a detrimental effect on the wall as the compaction process induces large horizontal earth pressures that are subsequently locked into the soil mass. Such pressures can vary considerably both in magnitude and distribution and are often much greater than the earth pressures predicted by the classical theories described previously.
5.4.1 Compaction of granular fill

Compaction-induced earth pressures against retaining walls for granular soils are determined using the simplified approach proposed by Ingold\(^{48}\). This method is summarised in Figure 5.11 and the corresponding equations are given.

\[
\sigma'_{hm} = \sqrt{\frac{2Q_i \gamma}{\pi}} \\
\sigma'_h = k \gamma z \\
\frac{z_c}{h_c} = k \sqrt{\frac{2Q_i}{\gamma \pi}} \\
h_c = \frac{1}{k} \sqrt{\frac{2Q_i}{\gamma \pi}}
\]

where

- \(k\) is the earth pressure coefficient. For a retaining wall that can move sufficiently to mobilise active conditions in the fill, \(k = k_a\), otherwise \(k = k_o\).
$Q_i$ is the intensity of the effective line load imposed by the compaction plant. For dead weight rollers, the effective line load is the weight of the roller divided by its width, and for vibratory rollers it should be calculated using an equivalent weight equal to the dead weight of the roller plus the centrifugal force generated by the roller’s vibrating mechanism. The latter may be taken to be equal to the dead weight of the roller in the absence of trade data.

$z_c, h_c$ are the critical depths as shown.

$\sigma_{hm}'$ is the maximum residual horizontal earth pressure induced by compaction.

$\sigma_h'$ is the horizontal earth pressure induced by overburden stress.

The compaction induced pressure can sometimes be a sufficient part of the total pressure against a retaining wall. Consequently it is often worth limiting the loading due to compaction plant within a certain distance behind the wall. Further information can be obtained from Broms(49), Ingold(48)(49), and TRRL publication LR766(50).

### 5.4.2 Compaction of cohesive soils

The compaction of cohesive soils can result in even greater earth pressures than those obtained using granular soils. Compaction in granular materials occurs under drained conditions where subsequent volumetric strains are minimal. This is not true for cohesive materials, however, and the effective stress equations for granular fill are not applicable.

Due to the complex behaviour of cohesive soils, the Department of Transport has published a specification that limits their use; see Symons and Murray(51), Clayton and Symons(52), and TRRL publications LR946(53) and RR192(54).

### 5.5 Available computer software packages for analysis and design

There are numerous commercially available analysis and design software products that are commonly in use by design practices. ReWaRD is one such product.

ReWaRD is a comprehensive software package that enables the analysis of the retaining walls to be performed. Wall types include sheet pile, bored pile (secant and contiguous), and diaphragm. Outputs include soil pressures each side of the wall, structural forces (bending moments and shear forces), together with estimates of ground movements behind the wall. The design options available include design to one of seven Design Standards (BS 8002, Eurocode 7, CIRIA Report 104, Hong Kong Geoguide(55), CP2(65), British Steel’s Piling handbook, and Highways Agency’s BD 42). ReWaRD is marketed by British Steel and Geotechnical Consulting Group and was written by Dr A J Bond and Dr D M Potts of Imperial College London.
5.6 Water pressures

The presence of water behind a bridge abutment has a marked effect on the forces applied to the wall. Many recorded retaining wall failures can be attributed to the incorrect allowance for water pressures. It is very important therefore that either provision is made for adequate drainage behind a retaining wall or that proper account is taken of all possible water pressures in design.

Where the water table is present at a shallow depth below the soil surface, a large proportion of the load on the active side of the retaining wall will be due to this. On the passive side of the wall, however, the effect of water pressure forms a much smaller proportion of the overall passive resistance. Additionally, where water is flowing through the ground, this will affect the value of pore water pressures and may reduce significantly the value of the passive resistance of the soil in front of the toe of the wall and increase the active soil load.

Notwithstanding the difficulties due to soil variability, simple methods of calculation can be used to form a basis for judgements.

5.6.1 Hydrostatic pressures

Where the tip of a steel sheet pile wall penetrates a virtually impermeable stratum, e.g. clay, but above this level the soil is considered to be permeable, pore water pressures on each side of the wall above that stratum will be equal to simple hydrostatic pressures (see Figure 5.12).

\[ u = \gamma_w x \]

where \( \gamma_w \) is the density of water (9.81 kN/m² for fresh water and 10.00 kN/m² for salt water)

\( x \) is the depth below water table.

![Figure 5.12](hydrostatic_pressure_on_an_abutment.png)

**Figure 5.12** Hydrostatic pressure on an abutment

In this case the maximum net pore water pressure \( u_c \) is given by:

\[ u_c = \gamma_w (h + i - j) \]
5.6.2 Water seepage

The situation where there is flow or seepage in the ground near the sheet piles is more complicated. Usually in these cases flow net techniques are used to determine water pressures and to model the flow regime in the ground.

In the long term, steady-state seepage develops when there is a difference in water levels behind and in front of the wall (see flow net in Figure 5.13). It is assumed that a sufficient volume is available above the clay, that the wall is impermeable, and that the soil has homogeneous permeability characteristics.

Figure 5.13 Flow net diagram for steady-state long-term seepage around abutment

Various simple methods are available for estimating water pressures without having to perform flow net studies. BS 8002 details an approach based on the assumption that the differential head of water is dissipated uniformly along the length of the flow path adjacent to the wall (see Figure 5.14).

Figure 5.14 Seepage pressures on abutment in the long term
In this case, the head difference is given by:

\[ h + i - j \]

and the flow path length by:

\[ 2d + h - i - j \]

The pore water pressure \( u_t \) at the bottom of the wall is:

\[ u_t = \frac{2 (d + h - j) (d - i)}{2d + h - i - j} \gamma_w \]

and the largest value of net water pressure \( u_c \) by:

\[ u_c = (h + i - j) \frac{2 (d - i)}{(2d + h - i - j)} \gamma_w \]

It is observed that the water pressure at the wall toe on the active side is equal to that on the passive side, however this is equivalent to reducing the bulk density of water \( \gamma_w \) on the active side of the wall and increasing it on the passive side. Therefore because of the upward seepage pressure, the passive soil pressure is reduced, and because of downward seepage, the active soil pressure is increased. Although this is not a significant problem for cantilever walls, it could be so for anchored walls where the retained height is usually greater.

### 5.6.3 Tension cracks in cohesive soils

On many sites there is overburden of fill or granular material on top of clay on the active side of a retaining wall, or alternatively the area behind the wall will have a roadway built over it. These features should prevent the formation of shrinkage or tension cracks in the clay behind the wall in periods of dry weather. The possibility of tension cracks opening in the short term, however, can lead to the likelihood of water penetrating to significant depths in the retained clay very quickly. For a wall moving as shown in Figure 5.15, it is reasonable to assume that tension cracks behind the wall do not extend below the excavation level in front of the wall.

![Figure 5.15](image)

**Figure 5.15** Cracks in clays for temporary works
The possibility of such cracks would seem to be confined mainly to cantilever walls, rather than to anchored walls where the position of the anchoring force is usually located at some distance below the wall head. The deflected form of the anchored wall will tend to close any cracks formed. Based on the above considerations, CIRIA 104 recommends that a minimum equivalent fluid pressure on the active side of a retaining wall of 5 kN/m² per metre depth measured below the wall head is used.

There may be instances where boreholes reveal that the fabric of the clay is such that there are substantial numbers of silt or sand partings. In such instances, even when considering the design of temporary works under short-term conditions, the designer should evaluate both soil and water pressures over the whole wall on the basis of long-term conditions and therefore in terms of effective stresses. Because it is very unlikely that full porewater pressure equilibrium prevails in the temporary works condition, however, it is usually acceptable to adopt a very low factor of safety when soil and water pressures are calculated on this basis.

5.7 Wall displacements and ground movements

Ground movements occur when soils are excavated adjacent to sheet pile retaining wall abutments. These movements can either be local to the abutment or in the vicinity of adjacent structures. As the magnitude and extent of these movements depend on soil properties, support stiffness, excavation technique, the phasing of excavation stages, and ground water, the ground movements cannot be predicted accurately.

BD 42 requires that wall displacements and ground movements are estimated to check that the serviceability limit states of adjacent and supported structures are not exceeded. Wall displacements and ground movements adjacent to the wall cannot, however, be calculated using limit equilibrium methods. Approximations of ground movements are therefore obtained from relevant field data and from experience of similar structures in similar ground conditions. Where the data indicate that adjacent structures or services may be affected, it will be necessary to perform a deformation analysis to determine ground movements. Differential movements are usually more important than total movements, and horizontal movements are often more damaging than vertical movements.

5.7.1 Estimating ground movements

Various methods are available to calculate ground movements. These include elastic analysis and elastic-plastic analysis methods, which in most cases require numerical finite element methods of analysis. These finite element methods require complex soil-structure models to be developed that are very sensitive to the parameters that are entered into the models. Specialist geotechnical expertise is often required to select sensible parameters, and where soil data are scant, ground movements are estimated based on case histories. These case histories can provide useful information for comparison with theoretical predictions or as a direct guide to likely movements around similar new constructions.

The patterns of movement associated with a free and rigidly propped flexible cantilever wall are illustrated in Figure 5.16, however these simple patterns of movement are altered by more global movements of the ground that occur around the structure.
Field observations

Observations of vertical ground movements around a number of excavations in various soil types have been summarised in graphical form by Peck (see Figure 5.17). Where more comprehensive information is required on ground movements, reference should be made to case histories that were used by Clough and O'Rourke and St John et al.

Figure 5.16 Settlement adjacent to bridge abutments

The data collated by Peck, Clough and O'Rourke, and St John et al. are used to indicate displacements of retaining walls in the limit equilibrium software analysis package ReWaRD.
6 SOIL-STRUCTURE INTERACTION

This Section provides guidance for the analysis of retaining walls using soil-structure interaction to determine mobilisation of earth pressures and to predict wall movement.

The simplistic limit equilibrium principles described in Section 5 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. In addition, many of the solutions only furnish values of limiting pressures on the wall assuming that the shearing resistance is fully mobilised and that the wall adhesion and angle of wall friction are constant and can be specified. No account is taken of the mode of wall displacement on the resultant earth pressure and no indication of the distribution or magnitude of earth pressure prior to ultimate failure is given. The estimation of structural forces in the embedded retaining wall under working load conditions is therefore extremely difficult.

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.

Commercially-available computer-aided soil-structure interaction analysis tools are described that are capable of analysing the behaviour of walls in the in-service condition.

6.1 The effects of wall deformation on soil pressures

Potts and Fourie\textsuperscript{(11)} used a finite element method of analysis to study the effects of wall deformation on soil pressures for a rigid wall embedded in an initially horizontal, uniform soil deposit. Three modes of wall deformation, namely horizontal translation, rotation about the wall top, and rotation about the toe, provided an analogy to commonly-occurring cases. The mode of deformation of a real retaining wall is complicated and depends in part on both flexibility and the type of wall. For embedded cantilever walls, the mode of displacement is essentially one of rotation about the toe. Alternatively, an anchor located near the top of the wall relates to the case of rotation about the top. The development of earth pressure coefficients with increasing wall displacement as determined by Potts and Fourie is shown in Figure 6.1.

Rotation about the base requires significantly more displacement to obtain failure conditions than do the other modes of displacement. For higher values of the at-rest earth pressure coefficient $k_o$, active and passive conditions are mobilised at similar displacements, however for low $k_o$ values, active conditions occur before passive conditions. Clearly displacements necessary to mobilise active and passive conditions are dependent on the value of $k_o$ as well as upon the mode of deformation. The limiting values of $k$ ($k_a$ active and $k_p$ passive) are less sensitive to the mode of displacement and the $k_o$ value.
Figure 6.1 Development of earth pressure coefficients with increasing wall displacement (rough wall) (Potts and Fourie)

The effect of the mode of wall displacement on the distribution of earth pressure is summarised in Figure 6.2. Numerical prediction was in agreement with experimental observations made by Brod(59), who carried out laboratory model retaining wall tests in which three modes of displacement were simulated. The lines labelled $k_a$ and $k_p$ shown on these figures correspond to the Caquot and Kerisel values and the labels 1, 2, and 3 refer to the sequence of movement. The pressure coefficient $k_m$ noted for each curve is given for passive pressures by

$$k_m = \frac{k - k_o}{k_p - k_o}$$

and for active pressures by

$$k_m = \frac{k - k_o}{k_a - k_o}$$

For a wall rotating about its top or bottom, the distributions are far from the linear distributions commonly assumed in traditional limit equilibrium design and given by the simplistic equations presented previously.

Potts and Fourie noted that for a wall rotating about its top, the soil on the passive side at failure is in an active condition near the top, but exceeds the classical passive value lower down. The pressure distributions also change during the mobilisation of the limiting condition. Such vastly different pressure distributions imply that wall bending moments will also be highly dependent on the mode of wall deformation for sheet pile retaining walls.
Figure 6.2  *Earth pressure distributions (rough wall) (Potts and Fourie)*

Clearly the development of limiting earth pressures is complex. While the pressures in the limiting condition for a wall rotating about its toe are similar to those given by the simple limit equilibrium method, for modes of deformation that involve mainly rotation about the top, and for the pressures prior to reaching a limiting condition for all modes of deformation, the stress distributions are nonlinear with depth.

Contours of normalised incremental displacement at failure ($\Delta/\Delta_{\text{max}}$, where $\Delta$ is the incremental displacement and $\Delta_{\text{max}}$ is the maximum displacement) for a smooth wall subject to the three different modes of wall deformation are shown in Figure 6.3.
Inspection of the figures for the wall rotating about the top and translating horizontally indicate that there are zones where the soil has a normalised displacement greater than 85% (zone A) or less than 15% (zone B). Soils in these regions have similar displacements and are therefore moving as blocks. Contours close together show high gradients of displacements and therefore indicate potential failure surfaces. For the wall that translates, the failure surfaces on both the active and passive sides of the wall are planar and in agreement with the Rankine planes of maximum stress obliquity (inclined at \(45 + \phi/2\) to the horizontal) that pass through the toe of the wall. For the wall rotating about its top the failure surfaces are curved.

For the wall rotating about its toe, a different pattern of displacement is indicated. There are no zones of block movement adjacent to the wall and no indication of failure surfaces. Instead the contours are equally spaced, indicating that the soil in front and behind is subjected to either uniform compression or extension respectively. Clearly, larger absolute wall movements would be required to mobilise plastic failure than in the narrow failure zones indicated for a wall rotating about its top or translating. This difference in mode of failure explains the differences in displacements between the different modes of wall movement. It also helps explain the large differences in displacement associated with embedded cantilever walls without an anchor (rotation about a point below formation level) and those with an anchor (rotation about anchor position).
Although the study presented above was for a rigid wall, it does show clearly actual behaviour rather than the assumed simplistic behaviour of the limiting equilibrium methods.

### 6.2 Soil-structure interaction analysis methods

Soil-structure interaction analysis methods predict the earth pressure distribution that acts on the design configuration of the wall. As the relative stiffnesses of the wall and the soil are modelled, the earth pressure profiles predicted using these methods are much more realistic and compare favourably with actual earth pressures. Figure 6.4 shows a typical earth pressure profile for an anchored wall obtained from a soil-structure interaction analysis.

![Actual measured pressure vs. Pressure profile from classical theory](image)

**Figure 6.4** *Actual horizontal earth pressure distribution for a flexible sheet pile abutment*

The simplest of the soil-structure interaction methods is the Winkler spring model (beam on elastic foundations) where the soil is modelled as a spring. There are other methods of increasing complexity, where the soil is modelled by boundary element, finite difference, and finite element numerical approximations.

#### 6.2.1 Winkler spring model

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. Normally the coefficients of horizontal subgrade reaction recommended by Terzaghi(39) are used.

The subgrade reaction approach is commonly used for soil-structure interaction because of the ease with which it can be applied, and 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 6.5). In addition, the in-situ horizontal soil stresses are input together with the active and passive earth pressure coefficients to provide the limiting values of the horizontal effective stress. Where an anchor is to be simulated, additional springs at the required stiffness and at the appropriate level are input.
In most situations the bending moments and shear forces obtained from the Winkler method are insensitive to the values of the spring stiffness chosen and used in the analysis. This is not the case, however, for the prediction of deformations of the wall. Deformations obtained from these analyses can therefore only be regarded as rough estimates and need to be checked by field measurements.

The analysis can be carried out with the wall being backfilled towards its top or excavation from the top of the wall downwards. If progressive softening that takes place in the long term as a result of the swelling of clays is to be modelled, then it is necessary to change the spring stiffnesses.

Limitations of the beam on elastic foundation approach are:

- difficulty in determining appropriate spring stiffnesses for analysis
- the method cannot simulate unusual initial soil stresses directly
- inability to model the development of wall friction and construction sequence
- inability to model surface movements of the retained soil.

### 6.2.2 Boundary element and finite element methods

More sophisticated techniques are available that model general soil-structure interaction problems. These methods are more rigorous in the formulation of the problem and overcome the shortcomings of the beam on elastic foundations. A commonly used boundary element method is the one proposed by Pappin et al.\(^{(61)}\)

Boundary element methods still have shortcomings, so if a sophisticated analysis is required, a finite element analysis should be performed. Finite element methods have the ability to predict both earth pressures and deformations with a minimum of simplifying assumptions; they are not discussed because they are seldom necessary or justified for routine design of bridge abutments. Where it is necessary to use finite element analysis techniques, specialist advice should be sought.
6.3 Available soil-structure interaction software packages for analysis

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

6.3.1 WALLAP

WALLAP (Wall Analysis Program) is a widely used commercial package from Geosolve and described by Borin\(^1\), designed specifically for routine retaining wall design. Finite elements are used to model the sheet pile wall and a finite element model or Winkler spring model can be chosen for the soil. The springs in the Winkler spring model can either be interconnected or independent.

WALLAP can be used to model cantilever walls, anchored walls, and strutted excavations. Initial or in-situ soil pressures can be defined and the pressures determined during wall/soil movement are constrained to lie between active and passive limits. Wall excavation can be modelled together with dewatering, placing of surcharge, and the introduction of struts and anchors. In addition, complex water profiles can be included to model steady seepage, submergence, and perched water tables.

The deliverables from the program include a stability analysis, wall displacement versus depth profile, bending moment, shear force, earth pressure distributions, and strut loads.

Users need to be aware that the Winkler spring approximations do not yield the same wall displacements as those given by more sophisticated models. Brooks and Spence\(^2\), for example, compared the results from WALLAP and FLAC, and concluded that lower stiffnesses must be used in the Winkler spring models to obtain similar displacements to the finite element models.

6.3.2 FREW

FREW (Flexible Retaining Wall analysis) is part of the OASYS suite from Ove Arup and Partners, London, which was also developed specifically for steel sheet pile and retaining wall design. The wall is represented as a line of nodal points, and three stiffness matrices relating nodal forces to displacements are developed. One matrix represents the wall in bending; the other two matrices represent the soil on each side of the wall. The soil can be modelled in three different ways:

- **Subgrade reaction method using Winkler springs** - where the soil is represented as a series of non-interactive springs. This method is considered to be unrealistic in many circumstances, but is appropriate when modelling laterally-loaded sheet piles.
- **SAFE flexibility method** - where the stiffness of the soil is represented as an elastic solid with the soil stiffness matrices being developed from pre-stored stiffness matrices calculated using the SAFE finite element program\(^3\). The elastic material representing the soil can have any specified variation of stiffness with depth.
- **Mindlin method** where the soil is represented as an elastic solid with the soil stiffness based on the integrated form of the Mindlin equations. This method
can model a wall of limited length in plan but is limited to constant stiffness with depth.

The FREW program can be used to analyse the behaviour for each stage of the construction sequence. At each stage it calculates the force imbalance at each node imposed by that stage and calculates displacement and soil stresses using the stiffness matrices. If the soil stresses are outside the active or passive limiting pressures, correction forces are applied and the problem solved iteratively until the stresses are acceptable. Allowance can be made for arching within the soil body when calculating the active and passive limiting pressures.

FREW determines earth pressures, shear and bending moments in the wall, strut forces, and displacements. Full details of the assumptions and analysis methods are given in Pappin et al.\textsuperscript{(64)}

6.3.3 FLAC

FLAC is a program for the solution of general geomechanical problems based on a finite difference method. The program has been developed by ITASCA Consulting Group, Minneapolis, USA. It is capable of solving a range of earth retention problems, and any type of nonlinear soil stress-strain relationship can be followed. It is suitable for the solution of retaining wall, tunnel-lining, and rockbolting problems.
7 OVERALL STABILITY OF EMBEDDED RETAINING WALLS

This Section presents the methods that are used to assess the overall stability of embedded retaining walls and determine a depth of embedment.

The governing criterion for stability is one of moment equilibrium or security against overturning of the wall. 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), however, a check should be made that a deep-seated slip plane (passing behind and below the wall) does not develop.

A depth of embedment for the wall to be used for design is obtained from the equation defining moment equilibrium, where restoring moments exceed overturning moments by a prescribed safety margin that is stated in Codes of Practice and Standards.

7.1 Cantilever walls

Embedded cantilever retaining walls rely solely on the resistance of the ground below excavation level for their stability. Fixed-earth conditions are said to apply, where the bottom of the wall is assumed not to displace. In addition, it is assumed that there is enough movement of the ground above the bottom of the wall to allow active and passive soil pressures to be generated, as determined by the classical theory of limiting equilibrium (see Section 5.1).

A steel sheet pile cantilever wall is in reality flexible, and it has a typical deflected profile as shown in Figure 7.1. For overall stability calculation purposes, however, the wall is assumed to be rigid compared with the soil, and to rotate about a point near to the toe of the wall (see point O, Figure 7.2).

![Figure 7.1 Actual displacement profile of a cantilever wall](image-url)
Figure 7.2 Assumed fixed-earth condition for a cantilever wall

For the minimum depth of embedment that will achieve equilibrium, the pressure distribution and corresponding active and passive soil pressure force resultants $P_a$ and $P_p$ are as shown in Figure 7.3.

Figure 7.3 Assumed pressure distribution for a cantilever wall

The pressure distribution is highly idealised, particularly in the vicinity of the point of rotation $O$, where it is assumed that there is an instantaneous change from full passive pressure on the front of the wall to full passive pressure distribution behind the wall. Determination of the required embedment depth using this distribution of earth pressures can be particularly tedious, and a further simplification is made. The difference between the active and passive pressure forces below point $O$ ($P'_a$ and $P'_p$) is replaced by an equivalent force $R$ at the level of point $O$. The resultant force diagram is as shown in Figure 7.4.

The value of the required depth to point $O$, $d_o$, is obtained by taking moments about point $O$. The minimum required depth of embedment is then determined by applying an empirical correction factor $C_c$, as proposed by Teng\(^{(49)}\). The minimum required embedment depth $d$ is given by:

$$d = d_o (1 + C_c)$$

where $C_c$ is taken to be 0.2.
It is recommended that a simple check is made to ensure that the additional embedment \((d - d_o)\) is sufficient to potentially mobilise a force at least as large as the force \(R\) implicit in the calculation of \(d_o\).

### 7.2 Anchored walls

An anchored retaining wall is formed when a cantilever wall is restrained by some form of tie near the top of the wall. For this configuration there are two methods of design - *fixed-earth* and *free-earth*.

#### 7.2.1 Fixed-earth method

In the *fixed-earth* method, the wall is assumed not to displace both at the bottom of the wall (as assumed for the cantilever wall) and also at the anchor position. Between these positions the wall has to bend, although displacements will be quite small. Nevertheless, active and passive pressures can be assumed to develop (see Fig. 7.5).

![Figure 7.5 Actual displacement profile of an anchored wall under fixed-earth conditions](image)

Although the *fixed-earth* method is used in Europe, both CIRIA 104 and BS 8002 recommend that the *free-earth* method should be used, because of its simplicity and economy in design.
7.2.2 Free-earth method

In the free-earth method, it is assumed that there is insufficient embedment of the wall to prevent horizontal movement at the toe. Consequently, the wall is assumed to rotate as a rigid body about the anchor point. The anchor point is assumed to move only sufficiently to develop active pressure in the retained soil (see Figure 7.6).

![Figure 7.6](image)

**Figure 7.6** *Free-earth boundary condition for a cantilever abutment*

The required depth of embedment is determined by equating moments about the anchor point, assuming fully mobilised active and passive earth pressures (expressed as resultant forces $P_a$ and $P_p$), as shown in Figure 7.7.

![Figure 7.7](image)

**Figure 7.7** *Free-earth boundary condition for an anchored abutment*

(Note that no correction to the depth of embedment is necessary for the anchored wall assuming free-earth conditions.) Consideration of horizontal equilibrium allows the necessary anchor force to be calculated.

7.3 Design methods

For design, the moment equilibrium condition is used directly or indirectly to ensure that restoring moments exceed overturning moment by a prescribed safety margin. This is achieved either by the use of partial factors in a limit state design method (often called the factor on strength method) or by use of a single factor of safety (often called the factor on moment method).

The two methods are applicable to both cantilever and anchored walls.
The methods are used to determine a depth of embedment based on horizontal loads only. The design of the wall to resist vertical loads is dealt with in Section 8, while the determination of the wall cross-section to resist the horizontal loads is dealt with in Section 9.

### 7.3.1 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 factors can be partial factors for an ultimate limit state or factors that represent the soil strength that is to be considered for a serviceability limit state.

In an effective stress analysis, the soil strength parameters include $c'$, the effective cohesion, and $\phi'$, the effective angle of shearing resistance. To allow for the uncertainties associated with $c'$ and $\phi'$, two factors of safety are defined: $F_c$ and $F_{\phi}$.

The factored parameters for effective stress analysis are termed mobilised values and are given by:

$$ c_{m}' = \frac{c'}{F_c} \quad \text{and} \quad \phi_{m}' = \tan^{-1}\left(\frac{\tan \phi'}{F_{\phi}}\right) $$

where $c_{m}'$ and $\phi_{m}'$ are the mobilised values of the respective strength parameters $c'$ and $\phi'$.

To maintain overall consistency

$$ \left(\frac{\delta}{\phi}\right)_m = \frac{\delta}{\phi'} \quad \text{and} \quad \left(\frac{c_w}{c'}\right)_m = \frac{c_w}{c'} $$

where $\delta_m$ and $c_{wm}$ are the mobilised values of wall friction $\delta$ and wall adhesion $c_w$.

For total stress analysis:

$$ c_{um} = \frac{c_u}{F_c} $$

where $c_{um}$ is the mobilised value of the undrained shear strength $c_u$.

It is common when performing an effective stress analysis to assume that $F_c = F_{\phi}$, although with the introduction of ENV 1997 individual factors for $c'$ and $\phi'$ are now quoted.

The mobilised strengths are used to calculate the earth pressure coefficients and the distribution of earth pressure on the wall. The factored strength parameters increase active earth pressures and reduce passive earth pressures and modify the relative distribution of these pressures.

The resultant forces on the back and front of the wall are then expressed as a function of the unknown depth of embedment. Equating moments to zero enables the embedment depth to be calculated. When the forces are expressed as functions
of embedment depth, this equation reduces to a cubic expression in terms of embedment depth.

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, in choosing the safety factor as it is sensitive in the calculation of embedment depth.

The factor of safety on soil strength is given in Table 7.1.

Table 7.1  Soil strength factors $F_c$ and $F_p$

<table>
<thead>
<tr>
<th>Reference</th>
<th>Design conditions</th>
<th>Effective stress</th>
<th>Total stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>CIRIA Report 104</td>
<td>Moderately conservative design</td>
<td>1.2-1.5*</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Permanent works in stiff clays</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Worst credible design</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BS 8002†</td>
<td>Serviceability limit state</td>
<td>1.2</td>
<td>1.5†</td>
</tr>
</tbody>
</table>

* Usually 1.5 except for $\phi' > 30^\circ$ when lower value may be used.
† These are mobilisation factors appropriate for a serviceability limit state and are not strength factors as such. They have been included because they follow a similar procedure in stability design when using this method.
‡ Larger than 1.5 for clays that require large strains to mobilise peak strength.

7.3.2 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 methods are available to determine an embedment depth, although each gives different answers and each behaves differently as parameters are varied. These are empirical methods that have been shown to behave successfully.

The three methods are:
- Burland-Potts method
- Gross pressure method
- Net total pressure method.

Burland-Potts method

This method was developed by Burland et al. It is a consistent method providing satisfactory results using a lumped factor $F_r$ for the practical range of soils and wall geometries.

The resultant earth pressure forces acting on the wall are split into net activating and net resisting components. The net activating forces are those forces that arise from the retained height of soil, while the net resisting forces are those forces from the soil below excavation level (see Figure 7.8). These net forces are
Earth Net water

Figure 7.8  Burland-Potts method

expressed as a function of the unknown depth of embedment. The net resisting moment \( M_{mp}' \) is divided by \( F_r \). The factored net resisting moment is equated to the net activating moment to obtain the depth of embedment. This method involves the solution of a cubic equation.

The active earth pressure diagram is modified by altering the pressure below excavation level (in front of the wall) to be equal to the pressure at excavation level. The passive earth pressure diagram is also modified, by deducting the difference between the initial and modified active pressures; these modifications are shown by the unshaded areas in Figure 7.8.

Taking moments either about the toe for a cantilever wall (fixed-earth method) or the anchor point for an anchored wall (free-earth method), the design embedment is that for which the following dependent relationship is satisfied:

\[
\frac{M_{mp}}{F_r} = M_a + M_w
\]

where
- \( M_{mp} \) is the moment of modified passive earth pressure
- \( M_a \) is the moment of modified active earth pressure
- \( M_w \) is the moment of net active water pressure
- \( F_r \) is the lumped factor of safety.

Care must be exercised where a value of \( c' \) is used and for total stress conditions, and reference should be made to the paper by Burland et al. \(^{(65)}\)

Factors of safety for the Burland-Potts method are presented in Table 7.2.

**Gross pressure method**

In this method, the active and passive pressure diagrams are unmodified. The gross passive pressure is factored by a lumped factor \( F_p \), leaving water pressures unfactored (see Figure 7.9).
Table 7.2  

<table>
<thead>
<tr>
<th>Reference</th>
<th>Design conditions</th>
<th>$F_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Burland, Potts and Walsh</td>
<td>Ultimate limit state</td>
<td>1.5-2.0</td>
</tr>
<tr>
<td>CI/RRA Report 104</td>
<td>Moderately conservative design,</td>
<td>1.5-2.0*</td>
</tr>
<tr>
<td></td>
<td>permanent works in stiff clays</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Worst credible design,</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>permanent works in stiff clays</td>
<td></td>
</tr>
</tbody>
</table>

* Usually the higher value.

Figure 7.9  

Gross pressure method

The design embedment is that which satisfies the relationship:

\[
\frac{M_p}{F_p} = M_a + M_w
\]

where $M_p$ is the moment of passive earth pressure  
$M_a$ is the moment of active earth pressure  
$M_w$ is the moment of net active water pressure  
$F_p$ is the lumped factor of safety.

This method is described in Code of Practice No. 2 Earth retaining structures (commonly abbreviated to CP2). CP2 only considered the design of walls in clays in terms of the gross pressure method and total stress conditions acting (undrained shear strength). For total stress analysis, this method can give rise to the value of $F_p$ rising to a peak and then decreasing as the penetration increases. This is illogical, and if the method is used it may become uneconomic.

When the method is used with effective stress analysis, it is conservative for permanent works design in the case of most clays if an appropriate value of $F_p$ is used (see Section 7.4).

Factors of safety for the gross pressure method are presented in Table 7.3.
Table 7.3  Gross pressure method - lumped factor

<table>
<thead>
<tr>
<th>Reference</th>
<th>Design conditions</th>
<th>$F_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>BSI CP2</td>
<td>Ultimate limit state</td>
<td>2.0</td>
</tr>
<tr>
<td>CIRIA Report 104</td>
<td>Moderately conservative design, permanent works in stiff clays</td>
<td>1.5-2.0*</td>
</tr>
<tr>
<td></td>
<td>Worst credible design, permanent works in stiff clays</td>
<td>1.2-1.5*</td>
</tr>
</tbody>
</table>

* The lower values may be used if $\phi < 20^\circ$; the higher values should be used if $\phi > 30^\circ$.

**Net total pressure method**

This approach is presented in the British Steel Piling handbook. In this approach, the active and passive pressure diagrams are modified to produce net pressures as shown in Figure 7.10.

![Passive and Active Pressures](image)

**Figure 7.10  Net pressure method**

The design embedment is that which satisfies the relationship

$$\frac{M_{np}}{F_{np}} = M_{na}$$

where $M_{np}$ is the moment of net passive (earth and water) pressures, $M_{na}$ is the moment of net active (earth and water) pressures, and $F_{np}$ is the factor of safety.

Diagrammatically this can be represented by the moment of the unshaded area on the passive side of the wall and the moment of the unshaded area on the active side of the wall.

Although this method is not recommended in CIRIA Report 104 for use in clays, it is used for granular soils. BD 42 permits the use of this method, provided that worst credible soil parameters are used and a factor of safety $F_{np}$ of not less than 2.0 is used.
7.4 Choice of analysis method

A comparison of the factors of safety defined by the methods above has been carried out by Potts and Burland\(^{66}\), Day and Potts\(^{67}\), 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 becoming adopted increasingly 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\(^{68}\).

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.

7.5 Requirements of the Highways Agency's BD 42

BD 42 requires that, for the ultimate limit state of overall stability, the required depth of embedment is determined, using one of the recommended limit equilibrium methods and factors of safety. No recommendations are given as to which method should be used.

Both moderately conservative and worst credible soil properties are to be used with at least two of the three stability methods and their corresponding lumped factor or partial factors.

Unfactored highway loadings are to be applied at the relevant points and at the appropriate levels, in combination with unfactored permanent and any other loads. Partial load factors for earth pressure are not to be applied, because a lumped factor approach is to be adopted. Live load surcharge is applied only to the retained side of the wall.

Where forces from the bridge superstructure act on the retaining wall, the vertical capacity of the pile for the depth of embedment determined from the overall stability analysis has to be shown to be adequate. Comprehensive information regarding vertical capacity is given in Section 8.
8 DESIGN FOR AXIAL LOADING

This Section presents methods for predicting the axial capacity of sheet piles and bearing piles for bridge abutments or piers that resist vertical loads from the bridge superstructure. BD 42 requires that BS 8004 and BD 32 are used to determine vertical load capacity. As BS 8004 is based on the lumped factor of safety approach and minimal information is given on steel sheet piles, however, it is recommended that the SCI publication Steel bearing piles guide\(^{(69)}\) is used for design.

Axial loads acting on the top of the steel abutment are loads directly from the bridge deck superstructure (see Figure 8.1). The vertical loads from the bridge deck superstructure comprise dead loads, traffic loads, and environmental loads, and are transferred from the bridge deck to the abutment via bridge bearings. These bridge bearings are commonly seated on a reinforced concrete pile capping beam (which forms the top of the sheet pile abutment) and are positioned on or as near as possible to the vertical axis of the steel sheet pile. This arrangement at the top of the sheet pile therefore enables an effective direct transfer of load to take place from the bearing via the capping beam and thereafter axially through the pile. Finally the axial load in the pile is transferred to and resisted by the surrounding soil.

![Figure 8.1 Vertical loading from bridge deck bearings](image)

8.1 Ultimate axial capacity and load transfer

A sheet pile bridge abutment 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 skin 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 8.2).
Figure 8.2  Wall friction and end bearing resistance against vertical loads

The basic relationship is given in BS 8004 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*

$A_s$ is the surface area of the pile in contact with the soil†

$q_b$ is the unit base resistance value

$A_b$ is the steel cross-sectional area of the base of the pile or plug cross-sectional area.

* The average value of $q_s$ over the length of the pile is taken for a soil profile with more than one soil type.

† See Section 8.7 for determination of surface area.

The relative magnitudes of the ultimate wall friction and ultimate end bearing resistances depend on the geometry of the pile and the soil profile. Where a pile is embedded in a relatively soft layer of soil, but bears on a firmer stratum, this type of pile is referred to as an end bearing pile. It derives most of its capacity from the base resistance capacity $R_b$. On the other hand, where no firmer stratum is available on which to found the pile, the pile is known as a friction pile. In cohesive soils, the wall friction capacity $R_s$ predominates, while in non-cohesive soils, the overall axial capacity is more evenly divided between shaft friction and base resistance capacity.

Numerous computer programs are available commercially to calculate the vertical capacity of piles. One is PILE, which is part of the OASYS suite of geotechnical programs\(^{70}\).
8.1.1 Mobilisation of shaft friction and base resistance

The equation presented above only considers the ultimate state condition where the pile has been allowed to deform sufficiently to allow both the ultimate shaft friction and the ultimate base resistance capacities to be developed. Commonly, load transfer curves are produced, which are plots of load resistance versus axial deformation of the pile head for displacements ranging from zero to the ultimate limit or to an achievable maximum value. These plots include mobilised soil-pile shear transfer versus local pile deflection and mobilised base resistance versus axial tip deflection.

Axial load transfer curves for clays and sands are obtained from large-scale static pile tests. These pile tests have monitored both the load and strain at a number of positions along the pile. Tests that have been performed by Gibbs et al. on clays and fine granular soils show the general behaviour (see Figure 8.3).

![Graph](image)

**Figure 8.3 Axial load transfer curves for soils**

It is found that at any position along the pile, full shaft friction resistance is not developed until the pile has deflected axially (relative displacement between pile and soil) to a magnitude in the range 7-10 mm. Once this deflection is reached, no further additional wall friction resistance develops and apart from any small reduction in peak value of wall friction (due possibly to strain softening), the curve tends to a constant resistance. This constant resistance value can be assumed to be the ultimate unit shaft friction of the soil at any level.

The behaviour of a pile in clay in base resistance is also shown in Figure 8.3. It is seen from the base resistance load versus pile tip deflection (equal to pile head movement) curve, that a greater pile tip deflection is required to achieve the ultimate base resistance capacity than is required to develop the shaft friction capacity, i.e. in excess of 40 mm.
The mobilisation of the total axial resistance with increasing displacement for a pile is obtained from the summation of the mobilised shaft friction resistance and the mobilised base resistance (see Figure 8.3).

8.2 Vertical settlement and serviceability

The design ultimate capacity of a steel pile $R_{cd}$ is given by:

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

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

$\zeta$ 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, while $\zeta$ is a material partial factor. These factors are not provided by BS 8002 or BS 8004 but are given in DD-ENV 1997-1 Eurocode 7. In Eurocode 7 for driven piles:

$$\gamma_s = 1.3$$
$$\gamma_b = 1.3$$
$$\zeta = 1.5$$

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 from BD 37.

8.3 Ultimate Capacity in cohesive soils

8.3.1 Wall friction

Most piles in clay develop a high proportion of their overall capacity in shaft friction, hence more effort has been devoted to developing reliable data for estimating values of shaft friction in clays than in sands. Previous work in this field for both clays (and sands) is presented in the SCI publication Steel bearing piles guide and various background references are given. The unit shaft friction $q_s$ for clays can be estimated in terms of the undrained shear strength of the soil and is given by the relationship

$$q_s = \alpha c_u$$

where $\alpha$ is a dimensionless factor

$c_u$ is the undrained triaxial shear strength of the soil.
BS 8004 does not offer any specific advice on the value of $\alpha$, however, SCI studies show that $\alpha$ can be taken conservatively to be 0.25 for the ultimate skin friction value. [From pile load tests it has been found that the value of $\alpha$ increases with time. In the short term (typically when site pile tests are performed) $\alpha$ is approximately equal to 0.25, but with time (months/year later) $\alpha$ can reach a long-term value of 0.5.]

### 8.3.2 Base resistance

The long-term drained base resistance capacity of a pile in clay is significantly greater than its undrained capacity. The settlements required to mobilise the drained capacity, however, are far too large to be acceptable. Also, the immediate load carrying capacity of a steel pile must be sufficient to support all loads during construction. For these reasons it is common to calculate the base capacity of piles in clay in terms of the undrained shear strength $c_u$.

The magnitude of the base resistance capacity $q_b$ generated in cohesive soils is given in BS 8004 as:

$$ q_b = 9c_u $$

where $c_u$ is the undrained triaxial shear strength of the soil beneath the toe.

As an indication, the undrained shear strength of cohesive soils is generally in the range from 20 kN/m$^2$ for soft clay to 400 kN/m$^2$ for very stiff or hard clays. For this range of $c_u$, the base resistance values are in the range 0.2 to 3.6 MPa.

### 8.4 Ultimate Capacity in cohesionless soils

#### 8.4.1 Shaft friction

The unit shaft friction $q_s$ for a granular soil is given by:

$$ q_s = 2N_b $$

where $N_b$ is the Standard Penetration Test value.

#### 8.4.2 Base resistance

Methods of estimating values of the base resistance can be based either on fundamental soil properties or soil properties determined directly from in-situ measurements. For cohesionless soils, the most reliable method of predicting base resistance is to use the static cone penetrometer (Dutch cone) in the site investigation. The base resistance is calculated from the relationship:

$$ q_b = \bar{q}_c $$

where $\bar{q}_c$ is the average cone resistance within the zone influenced by stresses imposed under the pile wall tip.

Extensive experience with pile predictions based on the cone penetrometer in Holland has produced a set of design rules that have been summarised by Meigh(72).
The magnitude of the base resistance capacity $q_b$ generated in cohesionless soils can also be given by the relationship:

$$q_b = 400N_b$$

where $N_b$ is the Standard Penetration Test value.

For sands, the base resistance values are an order of magnitude greater than cohesive soils and range in value up to 40 MPa. Although this value may seem high in relation to the 10 MPa quoted in BS 8004, it is nevertheless realistic, and even higher values (up to 70 MPa) have been measured in sands offshore.

### 8.5 Ultimate Capacity in rock

Where piles are driven through clay/sand strata but are terminated at depth into a relatively incompressible rock stratum, the main component providing resistance is the base resistance. In these cases low axial movements of the pile will occur, owing to the compressive strength of the rock, and it may not be possible to generate appreciable shaft friction resistance in the clay/sand layers. In many cases, the maximum design load for such a pile will be governed by the stresses in the pile material itself, rather than the base resistance in the rock.

Base resistance capacities for intact rocks are of a magnitude greater than base resistance capacities for even cohesionless soils, and base resistance has been measured in the range 100-400 MPa. In the case of weathered or highly jointed rock, base resistance values reduce significantly and can be of the order 10 to 100 MPa. Driven steel piles provide the means to use dynamic testing during driving, using the stresswave method. This can be used to demonstrate the magnitude of end bearing that is available in rocks.

### 8.6 Mobilisation of shaft friction on a retaining wall

For a bridge abutment, adequate resistance must be provided by the steel pile to accommodate the vertical loads from the bridge deck superstructure. The vertical loads are applied as axial loads acting at the top of the pile.

For design to resist the axial load acting at the top of the sheet pile, it is important that the overall behaviour of the pile abutment is considered. Although the design of the abutment to resist axial load is undertaken independently of the lateral loading case, the behaviour of the soil adjacent to the wall needs to be considered as the wall displaces laterally. The soil on the active or retained side of the wall moves down relative to the wall in order to mobilise friction in the beneficial direction and, on the passive side, the displaced soil has to move upward (see Figure 8.4). If the abutment 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 an axially loaded pile, it may conservatively be assumed that shaft friction resistance is mobilised along the wall bounded between excavation level and the
Figure 8.4  *Generation of wall-soil friction by pile movement*

pile tip (see Figure 8.5). Only the side of the wall in contact with the passive soil zone is then considered.

Figure 8.5  *Length of sheet pile contributing to wall friction*

### 8.7 Determination of shaft friction surface area

#### 8.7.1 Retaining wall abutment

The surface area of sheet piles and High Modulus Piles can be obtained from British Steel's *Piling handbook* in the section that lists the coating areas for piles. For sheet piles, the surface area is normally taken to be 80% of the coated area. 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, i.e. 40% of the coated area. This area is multiplied by the depth of embedment of the pile for which shaft friction is mobilised.

Where it is found that the depth of embedment based on stability is insufficient to provide the required vertical resistance capacity, it can be assumed that any extra length of pile will have friction acting on both faces of the pile.
8.7.2 Closed section and H piles

The surface area of sheet piles that comprise box and/or H piles depends on whether or not a soil plug is formed at the tip. If no plug is formed at the tip of the closed section, the surface area is given by the summation of outside and inside surface areas. If a plug is formed, the surface area is based on the outside surface only.

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 closed box section of a size based on the external dimensions of the H pile.

8.8 Determination of base resistance area

8.8.1 Retaining wall abutment

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

8.8.2 Closed section and H section sheet piles

For closed section sheet piles, the area to be used in the valuation 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 plus the pile wall tip end bearing and the lesser is taken.

8.9 Buckling aspects of fully and partially embedded piles

There are analytical solutions available to determine the buckling behaviour of fully and partially embedded piles but the methods are quite complex. One method is provided by Bowles (37). Bowles adopts the method of Wang (73), where the method is automated using a suitable computer analysis program. Although other methods are available (Davison and Robinson (36), Reddy and Valsangkar (75)), the method proposed by Bowles is much easier to use.

A simplistic calculation can be performed assuming that the pile is unsupported laterally by the soil along the excavated height.

8.10 Serviceability limit state

The load resistance versus pile head displacement curves are very useful in showing pile behaviour at ultimate and serviceability conditions. Figure 8.6 shows the mobilisation of resistance with deformation for a cohesive soil, including wall friction, end bearing, and their combination. For simplicity, the resistance profiles are drawn as straight lines rather than curves. To establish the point on the line that represents the serviceability condition, it is necessary to review the partial factors that are used in the procedure to design for the ultimate condition.
For a limit state design where partial factors for loads and resistance are used, the equation relating load and resistance at the ultimate limit state is given by:

\[ \gamma_1 \gamma_3 P = \frac{R_c}{\gamma_z} \]

where

- \( P \) is the unfactored axial load at the pile head
- \( \gamma_1 \) is the partial factor for loads from BD 37
- \( R_c \) is the ultimate axial capacity of the soil
- \( \gamma \) is the resistance partial factors \( \gamma_s \) and \( \gamma_p \) (see Section 8.2)
- \( \xi \) is the material factor to take into account uncertainty of soil parameters determined on site or in the laboratory.

The partial load factors are taken from BD 37 and the partial resistance factors from the draft version of Eurocode 7 (tables 7.1 and 7.2). The equation above can be rewritten by inserting the magnitudes of the partial factors, therefore:

\[ 1.4 \times 1.1 \ P = \frac{R_c}{1.5 \times 1.3} \quad \text{or} \quad P = 0.3R_c = R_{cd} \]

At the serviceability limit state the partial factors for load and resistance are all equal to 1.0, therefore the serviceability condition can be defined accurately by the intercept of the curve at a resistance value \( R_{cd} \) of 0.3\( R_c \). It is seen from Figure 8.6 that for an axial load magnitude of 0.3\( R_c \), the pile head displacement at the working condition, i.e. approximately 4 mm, is significantly smaller than the pile head displacement at the ultimate condition, i.e. approximately 30 mm. In addition, it is seen that in the case of a cohesive soil the resistance is predominantly provided by shaft friction.

### 8.11 Inclined anchors

Where an inclined anchor is to be used for an anchored bridge abutment, the sheet pile must be designed to resist the induced vertical component of the inclined anchor force. This may require a deeper sheet pile embedment since the vertical component of the tie rod force can only be transmitted to the soil by friction on
the passive face below formation level and by end resistance. No friction is assumed to be taken on the active face of the sheet pile.

### 8.12 Bending moments due to axial loading

Where vertical loads from the bridge superstructure act at the top of a retaining wall bridge abutment, additional bending moments have to be included in the design.

Vertical loads from the bridge superstructure are transferred to the soil axially by the pile. As the pile undergoes lateral displacement, additional bending moments are introduced into the abutment (see Figure 8.7).

![Figure 8.7 Eccentricity of loading for a cantilever abutment](image)

For anchored walls there is also the additional bending moment that is generated due to the eccentricity resulting from the connection detail between the wall and the anchor (see Figure 8.8).

For simplicity, the additional bending moment acting on the wall is taken as constant over the height of the wall and equal to the product of the force and the eccentricity. The value is determined as follows:

\[ M_{\text{axial}} = P (y_p \pm e_p) \]

for a cantilever wall, and

\[ M_{\text{axial}} = P (y_m + y_p \pm e_p) + T (y_m + e_a) \]

for an anchored wall.
Figure 8.8  Eccentricity of loading for an anchored abutment

where  \( P \) is the axial load from bridge superstructure
\( F \) is the anchor force from foundation loads only
\( T \) is the vertical component of the anchor force \( F \), zero if the anchor is perpendicular to the axis of the sheet pile \((\alpha = 0)\)
\( y_p \) is the lateral deflection at the point of application of \( P \)
\( e_p \) is the eccentricity of the application of \( P \) from the centroidal axis of the pile
\( y_m \) is the computed deflection at the elevation of the maximum bending moment due to earth pressure
\( e_a \) is the eccentricity of the vertical component of the anchor force from the centroidal axis of the sheet pile.

It is usual to restrict the additional bending moment due to eccentricity effects to approximately one-tenth of the bending moment due to earth pressure.
9 DESIGN FOR LATERAL LOADING

A knowledge of the structural forces acting on a steel bridge abutment 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 loads from the bridge superstructure (predominantly the self weight of the bridge deck and traffic loads) acting directly onto the top of the sheet pile.

Structural forces acting on retaining wall bridge abutments due to lateral forces are calculated using one of two methods. One method is to model soil-structure interaction effects using advanced analytical methods (as described in Section 6), while the second method is one where a simplistic limit equilibrium method is used (as described in Section 5).

9.1 Choice of appropriate method

Although BD 42 makes specific reference to CIRIA Report 104 and the limit equilibrium methods, in determining bending moments and shear forces in retaining walls, it permits the use of alternative numerical analysis methods; this will allow the use of the soil-structure interaction method.

For most cases either the limit equilibrium method or the soil-structure interaction method is adequate to determine conservative values of the structural forces in the retaining wall.

A soil-structure interaction method should be used, however, where:

- the proposed method of construction involves backfilling of soil/fill behind a wall
- initial in-situ at-rest soil stresses are known to be higher than nominal
- the bending moment distribution corresponding to the design depth of embedment is required
- it is required to compare the economics of wall type.

For backfilling operations the bending moments calculated using the soil-structure interaction method have been found to be much more realistic than those calculated by the limit equilibrium method (see Oldham trial wall\(^{(76)}\)).

With the requirement of the Highways Agency to consider the use of integral bridges (see BD 57 and BA42\(^{(77)}\)), the soil-structure interaction methods will become more widely used, because they provide the necessary strain-dependent models.

9.2 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 6.2). 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 (excavation or backfilling), and the initial stresses in the soil. *Worst credible, unfactored* soil parameters with *worst credible* ground conditions should be used in design.

Two popular commercial software packages are WALLAP and FREW. For a more comprehensive description of these products see Section 6.3.

### 9.3 Limit equilibrium method

Bending moments and shear forces in the wall and, where appropriate, forces in an anchor can be calculated either manually or by automated computer software such as ReWaRD.

Where a limit equilibrium analysis is to be applied to determine structure forces, BD 42 requires that CIRIA Report 104 is used. Two methods are presented in CIRIA Report 104 to calculate structural forces acting on cantilever and anchored retaining walls. They are termed the *ultimate conditions* method and the *working conditions* method. As CIRIA Report 104 recommends that the *ultimate conditions* method is used, only this method is considered further in this document.

#### 9.3.1 Ultimate conditions method

In this method, *worst credible, unfactored* soil parameters are used, together with a lumped factor for stability of 1.0, to determine the depth of embedment $d_{LF=1}$. Bending moments and shear forces are calculated based on using $d_{LF=1}$, where full active and passive earth pressure distributions are assumed to exist. The actual design depth of embedment, $d_{design}$, is greater than that required to achieve stability against overturning, but this additional depth is ignored when the bending moments and shear forces in the wall are calculated (see Figures 9.1 and 9.2).

![Figure 9.1 Ultimate conditions method for a cantilever abutment](image)

Commonly, the maximum bending moment is determined by locating the point of zero shear force acting on the wall. At this point the bending moment will be at its maximum. By taking moments about the point of zero shear force for all forces acting above it, the maximum bending moment is obtained.
Where water pressures are involved and seepage is considered, the seepage path for the design embedment should strictly be used in determining bending moments. Neglecting the additional seepage length, however, produces only small errors.

The limit equilibrium method does not, however:

- provide any information about the distribution and magnitude of soil movements
- analyse the effects of wall movement on bending moment and lateral earth pressure
- consider the influence of the initial soil stresses (at-rest pressure coefficient $k_d$)
- consider the effects of wall/soil flexibility
- model construction stages where backfilling takes place.

These effects can be included, however, albeit indirectly, by employing empirical techniques to modify the magnitude of structural forces, once the limiting equilibrium analysis has been completed.

The influence of the above effects is most pronounced in anchored bridge abutments. In over-consolidated cohesive soils, the underestimation of $k_n$ will result in significant underestimation of bending moment magnitude. These effects and the empirical techniques are discussed in more detail in Sections 9.3.2 to 9.3.4.

### 9.3.2 Effects of method of construction and in-situ stress state

The effects of method of construction and in-situ stress state of the soil on wall bending moment magnitude have been investigated by Potts and Fourie. An advanced numerical analysis (using finite element techniques), calibrated against actual data measured in the field, was used to determine horizontal deformations and bending moments for different soil conditions. The results from the numerical analysis were then compared with results obtained from a limit equilibrium method (Burland-Potts method).
The results for an anchored diaphragm retaining wall (which is comparable to a mid-sized High Modulus Pile wall) of retained height approximately 13 m are shown in Figures 9.3 and 9.4 and in Table 9.1. Displacements and bending moments are shown for both backfilled and excavated retaining walls, embedded in soils with $k_o$ equal to 0.5 and 2.0.

![Figure 9.3](image1)

**Figure 9.3** Comparison of wall displacement profiles for excavated and backfilled retaining walls

![Figure 9.4](image2)

**Figure 9.4** Comparison of bending moment profiles for excavated and backfilled retaining walls

Figure 9.3 shows that while the movements for both backfilled cases and for the excavated wall with $k_o = 0.5$ are similar, the movements for the excavated wall with $k_o = 2.0$ are approximately eight times larger. Also, an important difference
Table 9.1  Maximum bending moment for excavated and backfilled cases

<table>
<thead>
<tr>
<th>$k_o$</th>
<th>Excavated (FE method)</th>
<th>Backfilled (FE method)</th>
<th>LE method</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>4400</td>
<td>1220</td>
<td>1140</td>
</tr>
<tr>
<td>1.5</td>
<td>3085</td>
<td>1140</td>
<td>1360</td>
</tr>
<tr>
<td>1.0</td>
<td>1820</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5</td>
<td>1220</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Maximum bending moment (kNm/m)

is that for the excavated wall with $k_o = 2.0$, the base of the wall moves towards the excavation, whereas for the other cases little movement occurs.

The associated bending moments in the wall shown in Figure 9.4 show a similar trend to that of wall displacements, with the maximum bending moment for the excavated case with $k_o = 2.0$ being some four times greater.

Table 9.1 shows a comparison between the calibrated finite element method and the limit equilibrium method. The maximum bending moments for the excavated wall with $k_o = 0.5$ and the backfilled walls are very similar, but are smaller than those from the limit equilibrium analysis. For excavated walls with higher $k_o$ values, the bending moments are larger and exceed the limit equilibrium method values substantially.

A similar trend is observed for anchor forces.

9.3.3 Effect of wall stiffness on bending moments

The stiffness of the wall can have a large effect on wall movements and on distributions of earth pressures. This arises because flexible walls deform more readily and cause redistribution of earth pressures, which generally (but not always) leads to a reduction in bending moments and thrusts. These reductions, however, are achieved at the expense of somewhat larger movements that are of practical significance for cantilever walls.

The effects of wall stiffness, method of construction, and initial in-situ stresses have been investigated to provide a more comprehensive picture of the overall behaviour of retaining walls by Potts and Fourie. For a retained height of 13.26 m, four different wall stiffnesses were investigated corresponding to a rigid wall, a diaphragm/secant (or High Modulus Pile) wall, a sheet pile wall, and a soft wall. Figures 9.5 to 9.7 show clearly for an anchored retaining wall, the effect of wall flexibility on wall movement, earth pressures, and bending moments.

The maximum values of the bending moment are presented in Table 9.2. Values for $k_o = 0.5$ and 2.0 are given, along with values obtained from a limit equilibrium method calculation, based on a depth of embedment where the lumped factor for stability is 1.0, i.e. $d_{FE-1}$. For the low $k_o$ value, the limit equilibrium method values are higher than those given by the numerical method. For the $k_o = 2.0$ case, however, the maximum bending moments obtained from the numerical analysis are higher, and for the rigid and diaphragm wall they greatly exceed the limit equilibrium method values. The influence of $k_o$ on bending moments is larger the stiffer the wall.

A similar trend is observed for anchor loads.
Figure 9.5 *Effect of wall stiffness on lateral movement of anchored walls*

Figure 9.6 *Effect of wall stiffness on lateral earth pressures on back of anchored walls*
9.3.4 Correction factors for anchored walls

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. They define the modified parameter $\rho^*$ as:

$$\rho^* = \frac{L^4 E_s}{E_w I} = \rho E_i^{sv}$$

where $\rho$ is the original Rowe’s wall flexibility number, $L$ is the full length of the wall, $E_s$ is the average stiffness of the soil over the full length of the wall, $E_w$ is the stiffness of the wall, and $I$ is the inertia of the wall.

Summary plots obtained from this work are shown in Figure 9.8.
Figure 9.8  Variation of bending moment and anchor forces

These graphs show the variation of \( M_{\text{FE}}/M_{\text{LE}} \) and \( P_{\text{FE}}/P_{\text{LE}} \) with \( \log \rho^* \) respectively where:

\[
M_{\text{FE}}, P_{\text{FE}} \text{ are the maximum bending moment/anchor force from a finite element analysis}
\]

\[
M_{\text{LE}}, P_{\text{LE}} \text{ are the maximum bending moment/anchor force from a limit equilibrium analysis.}
\]

Both graphs refer to a wall anchored at its top with a factor of safety \( F_r = 2.0 \) (Burland-Potts method\(^{(65)}\)).

The results of this numerical parametric study are used to modify the values of maximum bending moment and anchor force obtained from simple limit equilibrium calculations. A modified form of this procedure has been implemented in the program GCG ReWaRD by Bond and Potts\(^{(83)}\).

It is important that these modification factors for moments and forces in the steel pile are only applied to anchored abutments that have been analysed using the limit equilibrium free-earth support method. These correction factors are not to be applied to cantilever retaining walls or to bending moments and forces obtained from a soil-structure interaction analysis.
9.4 Requirements of BD 42

For the design of the structural steel piling elements that comprise the bridge abutments, BD 42 permits either a limit equilibrium approach or an approach based on soil-structure interaction.

Where the limit equilibrium approach is to be used, BD 42 requires that “bending moments and forces in the wall are determined in accordance with CIRIA Report 104”. Load effects due to earth pressures (bending moments and shear forces in the wall), however, are multiplied by partial factors $\gamma_f$, as stated in Sections 3.5 to 3.12 of BD 42. This requirement supersedes the requirements presented in document BD 37. Relevant highway loadings applied to the bridge abutment and the corresponding partial factors are in accordance with those stated in BD 37.

Where a numerical analytical method is used for design of a bridge abutment, i.e. a soil-structure interaction method, the stability requirements, using the limit equilibrium approach, as stated in BD 42, are to be followed to determine the depth of embedment of the wall. For the design of structural elements, the requirements of Sections 3.5 to 3.12 of BD 42 are to be used. Reference to CIRIA 104 is not required as forces and bending moments due to the soil and highway loading are obtained directly from the soil-structure interaction analysis.
10 ANCHOR DESIGN

This Section describes typical anchor systems for sheet pile walls and the analysis and design methods that are appropriate.

10.1 Introduction

An anchored wall is required when the height of the wall exceeds the height suitable for a cantilever wall or when deflections are a consideration. Anchorage systems are used to restrain abutment walls against movement caused by earth pressure and hydrostatic pressure. The anchorages are placed in such a manner as to mobilise the gravitational and passive resistance of a body of ground between the anchor and the structure.

Anchorages for bridge abutments are of two general types: grouted ground anchorages including rock and soil anchorages, and deadman anchorages; see Figures 10.1 and 10.2 for typical configurations. Grouted ground anchors may be preferable when the retained soil carries services, shallow foundations, roads, or railways, or where the ground water level is above the tie rod level. In most other cases a passive anchorage (deadman anchors such as sheet pile or mass concrete anchor) is usually more economical, provided that there is sufficient open ground available.

Where poor soil exists above a better quality soil, or where the ground water level is too high to suit construction of a balanced anchorage, a deeper anchor wall may be installed by driving sheet piles. The anchor wall is then designed as a cantilever. In this type of solution, bending moments and deflections will be larger.

![Diagram of a ground anchorage system](Image)

Figure 10.1 *A ground anchorage system*
10.2 Ground anchorages

Ground anchors include grouted, stressed and unstressed tendons, and are usually drilled inclined to the horizontal in order to reach more competent ground. Generally, ground anchors are used where other types would not provide the necessary resistance, where there is insufficient space for tie rods, or where the ground contains services (see Figure 10.1).

The design and construction of ground anchorage systems and the materials and components employed, together with the necessary corrosion protection are covered by Section 6.5 of BD 42 and by BS 8081(84). As the design and construction of ground anchors is a specialist activity, it is usual that this aspect of design is undertaken by experienced specialists.

10.3 Deadman anchorages

Deadman anchorages usually consist of a concrete block or a steel sheet pile. The anchorage may either be continuous or a series of separate units.

For an anchor system to provide its maximum restraint capacity for the tie force, it must be located at a position where:

- the passive soil zone in front of the anchor wall does not interfere with the active soil zone behind the main retaining wall
- it is outside the zone of soil that is bounded by the angle of repose of the soil.

The minimum distance from the abutment retaining wall at which an anchor must be placed to develop its full capacity is illustrated by the shaded area in Figure 10.2.

If the anchorage is positioned such that the active soil failure zone for the retaining wall and the passive soil failure zone of the anchor intersect, a reduction in anchor capacity occurs. This reduction in anchor capacity can be determined analytically (see Terzaghi(85)).
The requirements for the design of a deadman anchor are given in BD 42, which refers to CIRIA 104. BS 8002 and BS 6349: Part 2 also provide assistance in design.

Where possible, deadman anchorages are designed to be balanced, i.e. where the differential passive and active pressures on the anchorage provide adequate resistance. The proportions and depth of embedment of the deadman are generally governed by the load acting in the tie rod and by the effective passive resistance of the soil against the face.

Active and passive pressures developed on continuous anchor walls are shown in Figure 10.3 for a homogeneous soil system, positioned at the ground surface or where $h/H$ is less than 0.5 (Terzaghi). Where the anchor is positioned such that $h/H$ is greater than 0.5, the anchor must be designed using bearing capacity theory (Smith or Krebs-Ovesens).

![Figure 10.3](image)

**Figure 10.3 Horizontal earth pressures acting on a deadman anchor**

For a cohesionless soil (the use of cohesive soil is not recommended in front of anchors) where $h/H$ is less than 0.5, the capacity of the anchor wall $C_a$ is given by:

$$C_a = P_p - P_a = (k_p - k_a) \frac{\gamma H^2}{2}$$

The tie rod connection to the anchorage should ideally be located at the point of resultant earth pressures acting on the anchorage.

The anchor tie force is calculated from the main analysis of the retaining wall using either a limit equilibrium or a soil-structure interaction method. BD 42 requires that the partial factor for load effects $\gamma$ is applied to the design axial force in the anchor.
If discontinuous deadman anchors are to be used (composed of a number of relatively short walls or blocks of concrete), analytical methods to determine anchor capacity can be obtained from the United States Steel *Steel sheet piling design manual*\(^{(89)}\).

### 10.4 Walings

The reaction from an anchored sheet pile wall is transferred to the tie rods by a flexural member known as a waling (see Figure 10.4). In most cases, a waling comprises two steel channel sections that are bolted back to back either side of the tie rod. Tie rods are commonly spaced at approximately 3-m intervals. For bridge abutments where appearance is important, the waling is located on the inside face of the wall, to provide a clear outside face.

![Figure 10.4 Tie rod and waling arrangement](image)

For sizing purposes, the response of a wale may be assumed to be somewhere between that of a continuous beam on several supports (the tie rods) and a single span on simple supports. On this basis, the maximum bending moment in the wale may be approximated to:

\[
M_{\text{max}} = \frac{T s^2}{10}
\]

where \(T\) is the tie rod force per metre of wall and \(s\) is the spacing of the tie rods.

Usually, tie rods and waling are proprietary items that are designed by their manufacturers based on tie forces provided by the bridge abutment designer.
11 DURABILITY AND MAINTENANCE

11.1 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/or the design of construction details. Failure to do so can result in maintenance problems that can require costly repair. Although designs may be satisfactory in terms of adequacy of material specification, construction, and minimising capital costs, any failure to assess and encompass whole life cost design will promote potential durability problems and increase maintenance costs. Documents BD 57 and BA 57 address these considerations and provide approval procedures and design recommendations for all aspects of durability.

11.2 Corrosion allowances

The means for countering the effect of corrosion of steel piles are well developed. Guidance is given in BD 42 and British Steel’s Piling handbook.

BS 8002 and BS 6349 both consider that the end of effective life of a steel sheet pile occurs when the loss of section, due to corrosion, causes the stress to exceed the specified minimum yield strength. A pile section chosen for the in-service condition has to be adequate at its end-of-design-life (i.e. after 120 years, when designed to BS 5400), at which time the effective pile section will have been reduced by corrosion. It is not immediately obvious whether the start of in-service life condition or the end-of-design-life condition will be critical for the structural design of the abutment, because the forces in the pile may have been modified by the change in stiffness of the pile.

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 to 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-life condition may be a critical design load case in the selection of the sheet pile section.

11.2.1 Corrosion and protection of steel piles

According to BD 42, steel piles are to be designed “with sacrificial thicknesses applied to each surface, depending on the exposure conditions, to provide a design life of 120 years”. Table 11.1 summarises the required sacrificial thicknesses for each surface for different exposure conditions, which are based on the advice given in BS 8002: Clause 4.4.4.4.3.

The reduced (corroded) section properties can either be obtained by calculation or from British Steel’s Piling handbook.

The procedure to determine corrosion allowances for a particular situation in accordance with BD 42 can be presented in the form of a flowchart (see Figure 11.1).
Table 11.1  Sacrificial thicknesses for piling according to BD 42

<table>
<thead>
<tr>
<th>Exposure zone</th>
<th>Sacrificial thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Atmospheric</td>
<td>4</td>
</tr>
<tr>
<td>Continuous immersion in water or effluent</td>
<td>4</td>
</tr>
<tr>
<td>In contact with natural soil</td>
<td>2</td>
</tr>
<tr>
<td>Splash and alternating wet/dry conditions</td>
<td>9</td>
</tr>
</tbody>
</table>

It should be noted that the corrosion allowances in BD 42 all apply to unprotected steel piles. Although it is generally cost effective to provide the sacrificial steel thickness, consideration can be given alternatively 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.
- Concrete encasement of steel piles above the water line. The inherent impact absorbing properties of the pile, however, are lost by this method.

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.

Details of these options are given in British Steel’s Piling handbook.

Where the exposed face of a retaining wall may be exposed to salty road spray (i.e. a splash zone), it will probably be more effective and aesthetically pleasing to use a non-structural cladding or a protective coating, rather than a large sacrificial thickness. No guidance on the use of cladding or coating to reduce the corrosion allowance is available in BD 42, although it is understood that this is currently being reviewed for a future revision of that standard.

British Steel has undertaken significant research and development into corrosion of steel and corrosion protection, and further advice can be obtained by reference to The corrosion and protection of steel piling in temperate climates and The prevention of corrosion on structural steelwork.

11.2.2 Corrosion in fill or industrial soils

Bridges are sometimes 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 will be 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: Part 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).
Figure 11.1  *Determination of corrosion allowance*
In a *controlled fill* (i.e. selected granular fill, as referred to in Clause 3.8 of BD 42), no special measures are required, and the same corrosion rates as in natural undisturbed soils can be assumed.

Further advice on corrosion assessment and protection can be obtained from British Steel, Swinden Technology Centre or from The Steel Construction Institute.

### 11.3 Corrosion and structural forces

It is not immediately obvious whether the start of in-service life case or the end of in-service life case will be critical for the structural design of bridge abutments. At the end of in-service life, the reduced stiffness of the corroded steel pile will in turn reduce the soil pressures acting upon it (and therefore the induced moments and shears). It is certainly unduly onerous therefore to apply effects resulting from a model using the start of in-service life (uncorroded) pile stiffness to a pile with end of in-service life (corroded) section properties.

As the corrosion loss allowance varies along the pile according to the corrosion environment, the designer should be aware that the maximum corrosion may not occur at the same level as the maximum section stresses.
12 CONSTRUCTION AND INSTALLATION

This Section considers sheet pile driving installation methods and equipment, driving analysis methods, construction tolerances, and the environmental implications of sheet pile driving. In addition, the use and construction of reinforced concrete pile capping beams for bridge abutments are considered.

12.1 Pile driving installation methods

Sheet piles can be installed by a variety of methods and equipment. Each has particular advantages and disadvantages, and the final choice is, in most cases, a compromise between speed, accuracy, and economy of installation. A further deciding element is the increasing concern for noise and vibration control.

There are two classes of pile driving installation, those based on impact, and those based on vibration.

12.1.1 Impact methods

Impact drivers using air, diesel, hydraulic, or cable are the most suitable for general work in mixed soils. If noise control is important, however, careful selection is necessary and some reduction in speed of driving should be expected.

**Drop hammer**

The drop hammer is the traditional method of pile driving and is still employed. Normally, purpose-made rigs hoist the pile into position, support it during driving, and incorporate a guide for the drop hammer (see Figure 12.1).

Drop hammers generally vary from 0.5 to 2 times the pile weight, with drops usually in the range of 0.2 to 2 m.

Variants of the simple winch-raised drop hammer include rams raised by steam or compressed air or hydraulically, free falling from the top of the stroke (single acting hammers), and hammers having pressure applied on the downstroke (double-acting hammers). Large single-acting hammers have been developed with maximum net hammer weights approaching 300 tonnes, producing rated energies of up to 2.5 MNm using steam pressure (Vulcan 6300).

**Diesel hammers**

In operation, the diesel hammer employs a ram that is raised by explosion at the base of a closed cylinder. Alternatively, a vacuum can be created in a separate annular chamber as the ram moves upward, and assists in the return of the ram, almost doubling the output of the hammer over the single-acting type. The ram weight is less than that of a drop hammer, but the more rapid action can make up for this by keeping the pile moving more quickly and thereby reducing soil resistance.

The action of a diesel hammer is best suited to driving piles that derive the greater part of their resistance at the pile tip, such as occurs when driving through granular deposits. The hammer is more versatile than a drop hammer, as it can be set on top of a pile without guide rails or a pile frame.
Figure 12.1  A steel pile driving rig

A characteristic of diesel hammers is the way in which the energy output increases with driving resistance. In a very soft soil there may be insufficient resistance to pile movement to cause satisfactory compression in the ram cylinder to produce an explosion, and a smaller hammer may be necessary in the early stages of driving.

Detailed hammer specifications can be obtained from manufacturers’ data.

Hydraulic hammers

Hydraulic hammers are suitable for all sites and soil conditions and are now the most popular because they are light and reliable.

The action of hydraulic hammers is different from that of diesel hammers. In single-acting hammers, the ram is raised by hydraulic pressure and is then allowed to free fall. In double-acting hammers, the ram has additional acceleration from hydraulic pressure through a reversing valve to assist that from gravity. The stress wave transmitted to the pile has less peak and more duration, which is more suited to driving piles in clay, where the majority of driving resistance is friction on the pile shaft.

Hydraulic hammers are more efficient and quieter in operation than other types, and are used increasingly in place of simple drop hammers and diesel hammers in many applications. They are also compact and adaptable, and may be used underwater with only slight modification.

There is a wide range of hydraulic hammers available worldwide produced by several manufacturers. This competition and their popularity has encouraged their
development in both technology and size to tackle the largest sizes of pile, with highly sophisticated instrumentation to measure energy output reliably.

12.1.2 Vibratory methods

Where the soils are essentially granular and of low to medium density, a vibrating driver will install piles very quickly. Where there are adjacent structures, however, care must be taken if the piles encounter clay strata, since ground resistance may then occur, with the possibility of structural damage due to vibration.

Vibratory hammers are usually electrically powered (but may be hydraulically powered), and consist of contra-rotating eccentric masses within a housing attached to the pile head. Most pile vibrators run at low frequencies, typically 20 to 40 Hz. At these frequencies, neither the exposed length of pile nor the soil will be in resonance. Sound propagation is low, and in cohesionless soils good rates of progress can be realised. During the driving progress, the granular soil immediately adjacent to the pile is effectively fluidized, and friction on the shaft is reduced considerably.

In cohesive soils, fluidization will not occur and vibratory pile driving methods are not generally as effective. In soils that are essentially cohesive, a hydraulic thrust driver will prove valuable in urban and city centre locations.

12.1.3 Resonance pile driving

Variable frequency vibrators can be used to good effect in some soils and can be useful in environmentally sensitive areas provided that the equipment is suitable for the soil conditions. If the frequency of vibration is increased up to perhaps 100 Hz, the pile will resonate longitudinally, and penetration rates can approach 20 metres per minute in loose to moderately dense granular soils. At these frequencies, non-cohesive soils are fluidized to the point where the frictional resistance on the pile shaft is reduced to close to zero and more driving energy is delivered to the pile toe.

This method of pile installation is potentially very effective but needs thorough investigation by the user and the manufacturer to relate hammer mass and frequency reliably to the type of soil.

12.1.4 Jacking methods of pile installation

Lengths of pile, either in short units or in continuous lengths, may be forced into the ground by jacking (usually hydraulically) against a reaction. Jacking methods are exceptionally quiet and vibration-free in use, and by monitoring the pressure in the hydraulic system a good understanding of soil resistance can be obtained during installation. The reaction system can be provided by tension piles, adjacent piling, or dead loads. Special applications are the Taywood Pilemaster system and the Giken system, in which steel sheet piles can be forced into the ground against the tensile reaction provided by adjacent previously driven piles.

The method has proved very successful for micro piling (piles less than 250 mm in diameter), since the reaction loads are then provided by the structure being underpinned.
12.2 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)\(^{92}\) 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\(^{93}\). 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 \textit{Ground vibration caused by civil engineering works}\(^{94}\), and the British Steel publication P105A \textit{Legislation and practice on noise and vibration control with particular reference to steel piling}\(^{95}\).

BS 6472\(^{96}\) deals specifically with evaluation of human exposure to noise and vibration in buildings.

12.2.1 Noise from piling operations

Pile driving is an inherently noisy operation and environmental restrictions can be imposed in the \textit{Conditions of Contract} for a project. Noise levels of 85 decibels within 10 m of the piling plant are quite common. Typical data on noise levels produced by piling operations have been published by CIRIA Report No. 64 \textit{Noise from construction and demolition sites - Measured levels and their prediction}\(^{97}\). These are discussed and interpreted in CIRIA Report PG9 \textit{Noise and vibrations from piling operations}\(^{98}\).

Investigations into the sources of the noise have shown that a large proportion of it arises from secondary effects, rebound of the hammer, rope slap, engine noise, etc. Improved design of the components of a piling rig can reduce considerably the high-frequency content of the noise emitted. To dampen the noise sufficiently to be acceptable in urban situations, it may be necessary to enclose the hammer or the guide rails in an \textit{acoustic chamber}. The use of such devices usually results in some reduction in the efficiency of the pile hammer and can create difficulties in handling and pitching piles. Alternatives to drop and diesel hammer types, such as hydraulic hammers with a \textit{skirt}, can reduce the noise because the point of impact of the ram to the top of the pile is enclosed.

Driving steel sheet piling is often exceedingly noisy since the driving cap usually involves steel to steel contact. In areas where severe restrictions are placed on noise levels, pile driving vibrators or the \textit{Taywood Pilemaster} hydraulic pile driver may be adopted. Such machines emit a different frequency and lower level of noise, which may be acceptable, but they involve the use of an auxiliary power
unit that may itself emit a high level of noise and must be enclosed by appropriate means.

12.2.2 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* (99), the publication *Dynamic ground movements - Man-made vibrations in ground movements and their effects on structures* (100), BRE Digest No. 403 *Damage to structures from ground-borne vibration* (101), and the references given in Section 12.2.1.

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. One of the most popular of these is included in the German Standard DIN 4150 (102), the recommendations of which are listed in Table 12.1.

<table>
<thead>
<tr>
<th>Class</th>
<th>Description</th>
<th>Maximum velocity (mm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Ruins and buildings of historical value</td>
<td>2</td>
</tr>
<tr>
<td>2</td>
<td>Buildings with existing defects</td>
<td>5</td>
</tr>
<tr>
<td>3</td>
<td>Undamaged buildings in technically good condition</td>
<td>10</td>
</tr>
<tr>
<td>4</td>
<td>Strong buildings and industrial buildings</td>
<td>10-40</td>
</tr>
</tbody>
</table>

These recommendations have not been drawn up specifically for ground vibrations induced by piling, and it is considered that they are overly stringent for that purpose. For structures that are not of great prestige or historical value, it is considered that the limits in Table 12.1 could well be doubled without noticeable effect.

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. Several levels of recording are possible. The simplest is manual recording of peak-to-peak signals, and the most complex is a full record of the ground vibrations to enable a frequency analysis to be carried out.

In general, human perception of vibrations occurs at levels that are low in comparison with the thresholds of risk for structural damage. BS 6472\(^{(96)}\) 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 gives 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. Since 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 and cooperation gained by prior notice and careful advice and explanation by the contractor.

### 12.3 Driving methods

The main requirement of sheet pile installation is that the location and verticality of the pile is controlled and that there are no digressions either parallel or transverse to the plane of the wall. There are two different driving methods available and they are termed:

- Panel driving
- Pitch and drive.

Panel driving is the best method to use in most cases. This technique requires a guidance system to be established so that a number of individual piles can be pitched to form a panel, or part of a wall. Generally the piles in the panel are interlocked in the vertical position before driving commences. The main advantage of this technique is that good control of the alignment of the piles can be obtained.

For the pitch and drive technique, either single piles or pairs of piles are pitched and driven before proceeding to the next. Recently, with the introduction of high speed vibrating pile drivers, the use of this technique offers very effective installation.

The above is only a brief description of the various methods that are available. Further information, showing the driving procedures for each method, can be found in the TESPA *Installation of steel sheet piles* booklet\(^{(103)}\) and in the British Steel *Piling handbook.*
12.4 Pile driveability

A sheet pile has to be of such a size that during driving to the required penetration, it is undamaged and has not deviated from its intended position. The driveability of a pile is a function of its cross-section properties, length, steel grade, the load applied and duration of this application, and the installation method. As the surface area of the piling profile increases, the greater is the required driving force. To prevent deformation of the pile head, it is necessary to ensure that the pile section chosen is appropriate to the in-situ soil conditions.

The analysis of steel sheet piles during driving cannot be performed using the methods available for individual bearing piles. This is because significant additional resistance between adjacent sheet piles is encountered during driving, due to interclutch friction. The best means for predicting dynamic sheet pile capacity is for the pile to be driven, the driving history recorded, and the pile load tested.

For the preliminary design of sheet piles during driving, published data are available to enable a minimum sheet pile section size to be chosen. The data presented below are given in the British Steel Piling handbook.

12.4.1 Driveability of sheet piles in granular soils

Minimum sheet pile section size requirements for driveability in granular soils are based on the classification of the resistance of the soil (N), which is obtained from Standard Penetration Test results. Table 12.2 provides the designer with a guide to the selection of an appropriate sheet pile section size based on the resistance of the soil. This is based on a simple relationship developed by British Steel, which is presented in BS 8002 and in the British Steel Piling handbook.

It is important to be aware that the guidelines given below are appropriate to British manufactured sheet piles, driven by the panel method for an approximate section width of 500 mm. For other section widths or for those driven by other methods, a greater section modulus than that specified below may be required.

Where the pile penetrates through differing soil strata type and thickness, it is necessary to ascertain the most adverse conditions that can be present.

The data presented above are based on the fact that for granular soils the resistance to driveability is due predominantly to resistance at the toe of the pile. Shaft friction due to the surrounding soil contributes relatively little to the overall resistance to pile penetration. The governing criteria for pile section adequacy is that damage should not occur either at the pile head by the hammer impact or at the pile toe due to soil resistance.

12.4.2 Driveability of sheet piles in cohesive soils

A guide to selecting a pile section size in cohesive soil is given in BS 8002 and in Specification for steel sheet piling published by the Federation of Piling Specialists (104). A table summarising the guidance is given in Table 12.3.
Table 12.2 Selection of sheet pile section in cohesionless soils - BS 8002 and British Steel Piling handbook

<table>
<thead>
<tr>
<th>Dominant SPT $N_6$</th>
<th>Minimum wall modulus (cm$^3$/m of wall)</th>
<th>Grade S270GP</th>
<th>Grade S355GP</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-10</td>
<td>450</td>
<td>Grade FE510A for lengths greater than 10 m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>11-20</td>
<td>450</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>21-25</td>
<td>850</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>26-30</td>
<td>850</td>
<td></td>
<td></td>
<td>Lengths greater than 15 m not advisable</td>
</tr>
<tr>
<td>31-35</td>
<td>1300</td>
<td></td>
<td></td>
<td>Penetration of such a stratum greater than 5 m not advisable</td>
</tr>
<tr>
<td>36-40</td>
<td>1300</td>
<td></td>
<td></td>
<td>Penetration of such a stratum greater than 8 m not advisable</td>
</tr>
<tr>
<td>41-45</td>
<td>2300</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>46-50</td>
<td>2300</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>51-60</td>
<td>3000</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>61-70</td>
<td>3000</td>
<td></td>
<td></td>
<td>Some declutching may occur</td>
</tr>
<tr>
<td>71-80</td>
<td>4200</td>
<td></td>
<td></td>
<td>Some declutching may occur with pile lengths greater than 15 m</td>
</tr>
<tr>
<td>81-140</td>
<td>4200+</td>
<td></td>
<td></td>
<td>Increased risk of declutching. Some piles may refuse.</td>
</tr>
</tbody>
</table>

Table 12.3 Selection of sheet piles in cohesive soils - BS 8002

<table>
<thead>
<tr>
<th>Clay description</th>
<th>Minimum wall modulus (cm$^3$/m)</th>
<th>Maximum length for driving (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Grade S270GP</td>
<td>Grade S355GP</td>
</tr>
<tr>
<td>Soft to firm</td>
<td>450</td>
<td>400</td>
</tr>
<tr>
<td>Firm</td>
<td>600-700</td>
<td>450-600</td>
</tr>
<tr>
<td>Firm to stiff</td>
<td>700-1600</td>
<td>600-1300</td>
</tr>
<tr>
<td>Stiff</td>
<td>2000-2600</td>
<td>1300-2000</td>
</tr>
<tr>
<td>Very stiff</td>
<td>2600-3000</td>
<td>2000-2500</td>
</tr>
<tr>
<td>Hard ($c_u &gt; 200$)</td>
<td>Not recommended</td>
<td>4200-5000</td>
</tr>
</tbody>
</table>

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

For cohesive soils, resistance to pile penetration results predominantly from shaft adhesion with the soil. Only minimal resistance is provided by soil tip resistance at the pile toe. The overall resistance is therefore a function of the undrained shear strength of the soil, the perimeter dimension of the pile section, and the length of the pile shaft embedded in the ground. In this case the governing criterion for adequacy of pile section is that the shaft and pile head must sustain this force without buckling.
12.4.3 Driveability of High Modulus Piles

Consideration must be given to the driveability of the High Modulus Pile section into soil specific to its proposed location. High Modulus Piles can be driven with standard pile hammers with special leg guides and inserts. As a guide to the choice of section for driveability it should be borne in mind that where very hard driving is anticipated, a Universal Beam of the heaviest weight per unit length in a serial size should be used. Conversely, for easy driving, a lighter weight may be used.

For specific guidance relating to driveability of High Modulus Piles, it is important to contact British Steel Piling Technical Services.

12.4.4 Recommended maximum driving lengths

The maximum recommended length for each pile section depends upon the type of strata encountered, penetration required, and type of construction for which the piling is to be used. Table 12.4, reproduced from the British Steel Piling handbook, is provided as a guide only.

Table 12.4 Recommended maximum driving lengths (British Steel Piling handbook)

<table>
<thead>
<tr>
<th>Larssen section</th>
<th>Maximum length (m)*</th>
<th>Frodingham section</th>
<th>Maximum length (m)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>LX8</td>
<td>14</td>
<td>1N</td>
<td>11</td>
</tr>
<tr>
<td>LX12</td>
<td>17</td>
<td>1BXN</td>
<td>14</td>
</tr>
<tr>
<td>LX16</td>
<td>20</td>
<td>2N</td>
<td>14</td>
</tr>
<tr>
<td>LX20</td>
<td>23</td>
<td>3N</td>
<td>18</td>
</tr>
<tr>
<td>LX25</td>
<td>25</td>
<td>3NA</td>
<td>18</td>
</tr>
<tr>
<td>LX32</td>
<td>28</td>
<td>4N</td>
<td>23</td>
</tr>
<tr>
<td>6W</td>
<td>9</td>
<td>5</td>
<td>24</td>
</tr>
<tr>
<td>GSP2</td>
<td>19</td>
<td></td>
<td></td>
</tr>
<tr>
<td>GSP3</td>
<td>23</td>
<td></td>
<td></td>
</tr>
<tr>
<td>GSP4</td>
<td>27</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6 (122 kg/m)</td>
<td>30</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6 (131 kg/m)</td>
<td>30+</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6 (139 kg/m)</td>
<td>30+</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Driving lengths given are only approximate and depend on steel grade and ground strata.

In hard driving conditions it may be necessary to increase section size to achieve the required penetration and/or adopt Grade S355GP steel.

12.5 Steel pile installation tolerances

The accuracy of pile installation is of the utmost importance to minimise on site connection preparatory work at the bridge deck to pile head interface. Tolerances must be accepted in the positioning of driven steel piles, however, and it will be necessary to consider lack of fit forces and local moments in the design of the connection details. 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*\(^{105}\) and specifications issued by the Federation of Piling Specialists\(^{104}\), the Eurocode draft prEN 12699\(^{106}\) *Execution of special geotechnical works*, and TESPA *Installation of steel sheet piles booklet*\(^{103}\).

Only the least onerous levels of tolerance are given in those publications. Following discussions with members of the Federation of Piling Specialists (FPS), a summary of the most accurate or best alignments that can be expected using specialist equipment, skilled personnel, and careful planning of procedures in respect of site conditions was drawn up and this is presented in Table 12.5.

**Table 12.5 Steel pile driving tolerances (TESPA)**

<table>
<thead>
<tr>
<th>Type of pile and method of driving</th>
<th>For pitch and drive method or over water</th>
<th>For panel drive method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deviation normal to the wall centre line at pile head</td>
<td>± 50 mm</td>
<td>Dependent on equipment used</td>
</tr>
<tr>
<td>Finished level deviation from a specified level</td>
<td></td>
<td></td>
</tr>
<tr>
<td>of pile head, after trim</td>
<td>± 20 mm</td>
<td></td>
</tr>
<tr>
<td>of pile toe</td>
<td>± 120 mm</td>
<td></td>
</tr>
<tr>
<td>Deviation from specified inclination measured over the top 1 m of wall</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Normal to line of piles</td>
<td>± 1%</td>
<td>± 1%</td>
</tr>
<tr>
<td>Along line of piles</td>
<td>± 1%</td>
<td>± 0.5%</td>
</tr>
</tbody>
</table>

Installation planning is essential to allow for any heterogeneity in the soils. Accuracy of alignment will also be affected by pile stiffness, the driving equipment, and the competence of the workforce. Conventional pile guide frames (formed of stiff Universal Beams with the webs horizontal) ensure good alignment for sheet piles in panel driving methods. It is also important that piles be finished to level with minimal tolerances and that any damage to the top of the pile is cut off. Experience of steel sheet pile driving shows that over-length piles should be supplied to allow for on-site changes in soil conditions between boreholes. This *overdrive* allowance is normally +10%, with spare piles being provided to replace any that are damaged severely during installation. Where dramatic changes in toe level are anticipated, e.g. a geologically eroded buried rock surface, steel piling is particularly appropriate but the advice of specialist piling contractors should be sought during conceptual design of the bridge foundation. The Federation of Piling Specialists should be consulted on such a matter. Sometimes driving causes previously driven piles to rise but this can be compensated by re-driving these piles.

It is inconceivable that an acceptable level can be achieved at the top of the pile to avoid any need for on-site cutting, however precision mechanical, burning, and hydraulic grit blast cutters are available to achieve a level cut within 1-2 mm accuracy.
12.6 Installation of piles and structural capacity

When sheet piles are fully driven into reasonably good soils, full section capacity is used for design. This is because the shear force in adjacent piles is resisted by:

- friction at the interlocks
- the capping beam
- the embedment of the piles below excavation level
- friction at the soil/steel interfaces
- loads transmitted from the walings, where appropriate.

Interlocks can however be crimped or welded where cantilever walls are to be piled, where piling is prevented by rock from penetrating to the normal depth of embedment, where the piling passes through soft clay or water, or when lubrication is applied to the interlocks.

Where effective transfer of shear force at the interlocks is not achieved and full structural capacity of the combined section is not developed (most relevant to Larssen U sections), excessive deflection of the completed sheet pile wall can occur. From experience, it has been found that a rotation of 5° can result in a 15% reduction in the section modulus. This reduction in structural capacity is not generally considered a problem where only an occasional pile has rotated, however consistent pile rotation can lead to an unacceptable situation.

Detailed information can be obtained from the Federation of Piling Specialists.

12.7 Reinforced concrete pile capping beams

The inclusion of pile cap beams to the tops of steel sheet piles and High Modulus Piles is quite common in practice and generally consists of reinforced concrete construction (see Figure 12.2). While reinforced concrete pile cap beams reduce the requirements for accuracy and quality of site-work, they increase the amount of site-work and add to the length of construction time. As the deck superstructure usually starts at ground level, pile caps are frequently constructed below existing ground level with the consequential problems of avoiding water ingress and keeping the reinforcement clean. Where finished ground level is above the existing ground, there is often a problem of shuttering a large concrete pour before the backfill has been raised to the underside of the pile cap and of ensuring properly compacted concrete.

The benefits of using a concrete pile cap beam are:

- It transfers load from a single member in the bridge deck to a group of piles, thereby distributing the load effectively.
- It allows the beam spacing of the deck structure to be different from the spacing of the beam sections within the High Modulus Piles.
- It enables the deck structure interface detail (e.g. holding down bolts for a steel superstructure or starter bars for a reinforced concrete superstructure) to be attached to the heads of the piles.
- It accommodates the difference between nominal and as-built geometry of both the piles and the superstructure. The most serious of these is generally
thought by superstructure designers to be the position of the piles, but piling contractor experience shows that this is not so. Communication in this area is essential to remove this misguidance.

- It accepts a relatively large tolerance level of pile heads, from trimming to length after driving.
- It accepts damage to pile heads from driving.

Figure 12.2  *Section through a reinforced concrete pile cap*
13 DESIGN PROCEDURE

This Section provides a methodology for the geotechnical analysis and structural design of steel sheet pile bridge abutments, following the design basis explained in Section 3. Each aspect of analysis and design is addressed and a sequence in which these activities should be undertaken to design most effectively is presented. A worked example using the design procedure presented below is shown in Appendix A.

In essence, the requirements stated in BD 42 are followed. Although BD 42 concentrates primarily on the limit equilibrium method, it allows the use of numerical analytical methods based on the principle of soil-structure interaction. In the methodology presented below, appropriate guidance is given that will assist the designer in adopting a suitable procedure for the particular case under investigation.

13.1 Design methodology

A well-structured methodology 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 globally for design. Guidance given today tends to be less prescriptive than in the past. This method allows the designer to think, rather than apply rigidly a set of prescribed instructions. As geotechnics is not defined as precisely in standards and codes of practice as structural analysis and design, it is necessary to introduce a greater measure of prescription in design.

A structured methodology that can be applied to the design of a bridge abutment is presented as a flow chart in Figure 13.1. It is a high level flowchart, showing the main activities and the order in which they need to be performed. No detailed breakdown is given of the activities, but a brief comment on each is given in Section 13.2, including reference to sections of this publication for detailed advice.

13.2 Activities in the design procedure

13.2.1 Activity 1 - Design philosophy

The philosophy that is to be used for the design of embedded steel pile bridge abutments in the UK is discussed in Section 3 Design basis. Reference is made in that section to the relevant national and European standards and the advice and requirements of the Highways Agency.

13.2.2 Activity 2 - Site investigation and soil data

The importance of a site investigation is discussed in Section 4.1, and the soil data for design and detailed explanation relating to the selection and evaluation of soil parameters are given in Sections 4.2 and 4.3. Sections 4.4 to 4.7 provide information on the physical characteristics of soils, and typical values are given.
Activity 1: Determine design philosophy

Activity 2: Perform site investigation and obtain soil properties

Activity 3: Choose abutment configuration and section profile

Activity 4: Perform stability analysis and determine depth of embedment

Activity 5: Select a preliminary section size

Activity 6: Determine vertical loads from bridge superstructure

Activity 7: Satisfy stability and vertical load capacity requirements

Activity 8: Choose an analysis approach

Activity 9a: Determine bending moments and forces on pile abutment due to earth and water pressure

Activity 9b: Determine bending moments and forces on pile abutment due to earth and water pressure

Activity 10: Determine bending moments from bridge superstructure loads

Activity 11: Assess ground movements at and near abutment

Activity 12: Determine structural adequacy of bridge abutment

Activity 13: Design pile capping beam

Figure 13.1 Flowchart showing design procedure
13.2.3 Activity 3 - Bridge abutment configuration and section profiles

The types of bridge abutment configurations are discussed in Section 2.1 and the section profiles that are commonly used are presented in Section 2.2. Benefits of the use of steel piles and aesthetic considerations are discussed in Sections 2.3 and 2.4. Actual built bridges using steel sheet piles are listed in Section 2.5.

13.2.4 Activity 4 - Lateral pressures, depth of embedment, and stability

The equations that are to be used to determine lateral earth and water pressure profiles acting on the wall are given in Sections 5.1 to 5.4. Information relating to wall friction and, where applicable, wall adhesion is given in Section 4.7.

Corresponding computer software using the limit equilibrium method is discussed in Section 5.5.

The limit equilibrium methods used to check the overall stability of the retaining wall due to overturning, and to calculate the required depth of embedment are presented in Section 7. The behaviour of cantilever and anchored walls is discussed in Sections 7.1 and 7.2 respectively. The methods of analysis that are commonly used are presented in Section 7.3 together with the corresponding lumped factor or partial factors to be applied. The choice of methods is discussed in Section 7.4 and the requirements of BD 42 are given in Section 7.5.

13.2.5 Activity 5 - Choice of preliminary section size

Pile section sizes for the pile profile chosen earlier are obtained from the British Steel Piling handbook, and information on material for steel piles is given in Section 3.2.3.

Pile driving requirements

Minimum size requirements for successful pile driving are presented in Section 12.3. Tables 12.1 and 12.2 are applicable for sheet piles in cohesionless soils and cohesive soils respectively. For High Modulus Piles and box piles, specific guidance is given in Section 12.3.3. As maximum recommended driving lengths also influence pile section size, Table 12.3 provides further guidance on the selection of a section size.

Section structural capacity

Development of full structural capacity of the sheet pile wall is important in preventing excessive lateral displacement. This aspect is discussed in Section 12.5.

Corrosion requirements

Both uncorroded and corroded section properties need to be determined. Corrosion allowances to be used in design are given in Section 11.2. Reduced section properties allowing for corrosion are either calculated or obtained directly from the British Steel Piling handbook.
13.2.6 **Activity 6 - Determination of vertical load from bridge superstructure**

Bridge superstructure loads acting at the top of the pile are obtained from the bridge deck analysis. Normally unfactored loads split into their individual components (i.e. dead, live load, etc.) will be obtained.

13.2.7 **Activity 7 - Satisfying stability and vertical capacity requirements**

The bridge abutment has to resist vertical loads from the bridge superstructure that act at the pile top. Comprehensive information regarding vertical resistance and capacity of the sheet pile is given in Section 8.

Commonly, the easiest solution to satisfy both stability and vertical capacity requirements is to increase the depth of embedment, unless there is a restriction in maximum recommended driving length. If the depth of embedment exceeds the maximum recommended driving length, then a larger size pile will be required (see Section 12.3).

13.2.8 **Activity 8 - Choice of analysis method to determine forces acting on the retaining wall**

Two distinct methods can be used to determine structural forces acting on the wall and anchor, if appropriate. The choice of method is made by the engineer taking into consideration the implications of Sections 5, 6, and 9.1.

13.2.9 **Activity 9a - Determine structural forces using the limit equilibrium method**

The limit equilibrium method procedure used to determine bending moments, shear forces, and anchor forces, where applicable, resulting from lateral loading, is discussed in Section 9.3.

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) are discussed in Sections 9.3.2 and 9.3.3 and given in Section 9.3.4.

The structural forces acting on the sheet pile have to be determined both for uncorroded and corroded section properties. The application of modification factors to account for differing wall stiffness will result in two sets of factors being obtained. This is discussed in Sections 9.3.4 and 11.2.

The design of anchors and their appurtenances is given in Section 10.

13.2.10 **Activity 9b - Determining structural forces using the soil-structure interaction method**

Forces acting on the wall due to lateral loading resulting from soil, surcharges, soil compaction, and water are determined from the soil-structure interaction methods presented in Sections 6.2, 6.3, and 9.2. The effects of construction, in-situ stress, and wall stiffness are taken into account directly by the underlying theory.
The structural forces acting on the steel pile abutment have to be determined both for uncorroded and corroded section properties. Two separate analyses need to be performed, one for the uncorroded section, the other for the corroded section (see Section 11.2).

The design of anchors and their appurtenances is given in Section 10.

### 13.2.11 Activity 10 - Bending moments due to loads from the bridge superstructure

Bending moments acting on the wall arising from vertical bridge deck loads occur because of the lateral displacement of the wall and installation tolerances. These bending moments can be obtained from the equations given in Section 8.12. Displacements can be obtained either from the soil-structure interaction analysis or from estimated lateral displacements if a limit equilibrium analysis has been performed. The estimated displacements can be obtained by accessing a database of displacements measured from field investigations (see Section 5.7). Guidance on installation tolerances is given in Section 12.4.

### 13.2.12 Activity 11 - Assessment of ground movements

Estimates of displacements are made by accessing a database of displacements and ground movements measured from field investigations (see Section 5.7). For most situations, ground movements adjacent to the wall will be of little importance and there will be no need to attempt to determine them.

### 13.2.13 Activity 12 - Structural adequacy of pile abutment and appurtenances

The structural adequacy of the wall and its appurtenances under axial loading, bending, shear loading, and their combinations has to be confirmed based on limit state philosophy presented in Section 3.2. Steel pile bridge abutments and their connections are to be designed in accordance with Highway Agency standards (see Sections 3.2.2 and 9.4) and, where appropriate, accompanying Codes of Practice, as presented in Section 3.2.

### 13.2.14 Activity 13 - Design pile capping beam

The design of the pile capping beam at the top of the bridge abutment is based on the limit state design methodology in BS 5400. Additional information on pile capping beams is presented in Section 12.7.
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APPENDIX A  WORKED EXAMPLE - CANTILEVER SHEET PILE BRIDGE ABUTMENT
A.1 Brief

This worked example comprises analysis and preliminary design of a cantilever steel sheet pile abutment for a simply supported bridge. The objective is to determine an overall configuration for the abutment including an appropriate sheet pile section size. It is not a comprehensive design as would be performed in a design office as it is only intended to show procedures that can be used.

A.2 General arrangement

The bridge carries a single lane A-road over a small river. Composite Universal Beams have been used for the bridge deck. A clearance of 2.75 m is to be provided under the bridge.

Figure A.1 Elevation of bridge

Figure A.2 Section through bridge deck
A.3 Design data

A.3.1 General

<table>
<thead>
<tr>
<th>Span</th>
<th>19.0 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Headroom</td>
<td>3.2 m</td>
</tr>
<tr>
<td>Carriageway</td>
<td>7.3 m wide (2 lanes)</td>
</tr>
<tr>
<td>Surfacing</td>
<td>100 mm thick minimum</td>
</tr>
<tr>
<td>Footways</td>
<td>2.0 m wide each side</td>
</tr>
</tbody>
</table>

Location: United Kingdom
Design life: 120 years

Loading

Live loads:
- HA: 2 lanes (full loading on each lane)
- HB: 30 units
- Footway: 5 kN/m²

Unit weights:
- Concrete: 25 kN/m²
- Surfacing: 24 kN/m²

Formwork: 0.5 kN/m² allowance for possible formwork

A.3.2 Material

The material for the sheet piles complies with BS EN 10248

(i) Yield strength

Two grades of steel are available:

- Steel S270GP: \( \sigma_y = 270 \text{ N/mm}^2 \)
- Steel S355GP: \( \sigma_y = 355 \text{ N/mm}^2 \)

(ii) Young's Modulus

\( E_y = 205,000 \text{ N/mm}^2 \)

A.3.3 Soil properties

The properties of the sand and gravel (cohesionless soil) are:

(i) Bulk weight

The bulk weights for the soil in its saturated and unsaturated state are as follows:
### Soil Properties

<table>
<thead>
<tr>
<th>Soil State</th>
<th>Unit Weight (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Saturated</td>
<td>22</td>
</tr>
<tr>
<td>Unsaturated</td>
<td>20</td>
</tr>
</tbody>
</table>

#### Angle of Effective Shear Resistance

- **Moderately conservative**: \( \phi' = 37^\circ \)
- **Worst credible**: \( \phi' = 33^\circ \)

#### Cohesion

- **Cohesion**: \( c' = 0 \)

#### Standard Penetration Test

- **N**: 35 (typical)

### Soil/Wall Friction

<table>
<thead>
<tr>
<th>Friction Type</th>
<th>Active</th>
<th>Passive</th>
</tr>
</thead>
<tbody>
<tr>
<td>2/3 ( \phi' )</td>
<td>1/2 ( \phi' )</td>
<td></td>
</tr>
</tbody>
</table>

### Analysis Method

A limit equilibrium analysis using ReWaRD was performed.

### Bridge Deck Loads

**Bridge deck loads** comprise dead loads and live loads.

#### Dead Load

An estimate of dead load is made assuming typical details and section sizes for a 19-m span.
 Calculation Sheet

a) Edge beam

![Diagram of edge beam]

Figure A.3 Cross-section in the vicinity of the edge beam

Assumes \( g = 10 \text{ m/s} \)

1. Universal beam:

From BS Steel bridge charts, a 914x305x289 UB was assumed

\[
914 \times 305 \times 289 \text{ UB} = 2.89 \text{ kN/m}
\]

2. Slab + Upstand: \((220 \times 3150 + 320 \times 500) \times 25 \times 10^4\)

\[
= 21.3 \text{ kN/m}
\]

3. Footway: \(2 \times (0.27 + 0.2)/2 \times 25\)

\[
= 11.75 \text{ kN/m}
\]

4. Parapet

\[
= 0.5 \text{ kN/m}
\]

5. Surfacing: \(0.65 \times (0.1 + 0.116)/2 \times 24\)

\[
= 1.68 \text{ kN/m}
\]

6. Permanent formwork: \((3.15 - 0.5) \times 0.5\)

\[
= 1.325 \text{ kN/m}
\]

Total unfactored dead load

\[
= 39.4 \text{ kN/m}
\]
Figure A.4  Section through deck (inner beam)

1. Girder: \[ 2.89 \text{kN/m} \]

2. Slab: \[ 3 \times 0.22 \times 25 = 16.5 \text{kN/m} \]

3. Surfacing: \[ 3 \times (0.116 + 0.191)/2 \times 24 = 11.1 \text{kN/m} \]

4. Permanent formwork: \[ (3 - 0.4) \times 0.5 = 1.3 \text{kN/m} \]

Total unfactored dead load \[ = 31.8 \text{kN/m} \]

A.4.2 Live loading

For this worked example only HA loading has been considered. In a design, both HA and HB loading would be considered.

(i) HA uniformly distributed load

Loaded length \[ = 19 \text{ m} \]

From BD 37/88, Clause 6.2.1

UDL loading \[ w = 336 \left( \frac{1}{19} \right)^{0.67} \]

\[ = 46.7 \text{kN/m of notional lane} \]

UDL/girder \[ = 46.7 \times \frac{3}{3.65} \]

\[ = 38.4 \text{kN/m} \]
(ii) HA KEL

\[ KEL = 120 \text{ kN per notional lane} \]

\[ = 120 \times \frac{3}{3.65} = 99 \text{ kN per girder} \]

A.5 Design loads from bridge deck

The design loading transmitted to the sheet pile from the bridge deck is as follows:

A.5.1 Dead load

The design vertical load acting at the top of the bridge abutment

\[ P_{design} = \gamma_{fl} P \]

where \( \gamma_{fl} \) for ultimate limit state is given in the table below

<table>
<thead>
<tr>
<th>Material</th>
<th>ULS ( \gamma_{fl} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>1.05</td>
</tr>
<tr>
<td>Concrete</td>
<td>1.15</td>
</tr>
<tr>
<td>Deck surfacing</td>
<td>1.75</td>
</tr>
<tr>
<td>Other superimposed</td>
<td>1.2</td>
</tr>
</tbody>
</table>

The most heavily loaded beam is the inner beam. At ultimate limit state the design dead load is:

\[ 1.05 \times 2.89 + 1.15 \times 16.5 + 1.75 \times 11.1 + 1.2 \times 1.3 \]

\[ = 43.0 \text{ kN/m} \]

For a bridge of span 19 m, the reaction load at the support is:

\[ \frac{1}{2} \times 43.0 \times 19 = 408.5 \text{ kN} \]

As the bridge deck beams are at 3-m spacing, the load acting per metre width of sheet pile is equal to:

\[ 1/3 \times 408.5 = 136.2 \text{ kN per m width} \]
Dead load due to the pile cap and the sheet pile is assumed to be:

- Concrete pile cap = 20 kN per m width of wall
- Steel pile = 9 kN per m width of wall

The ultimate limit state for dead load of the pile cap and pile is:

\[ 1.15 \times 20 + 1.05 \times 9 = 32.5 \text{ kN} \]

Therefore the total dead load component at ultimate limit state is:

\[ 136.2 + 32.5 = 168.7 \text{ kN per m width} \]

A.5.2 Live load

For HA loading alone

\[ \gamma_{fL} = 1.5 \]

(i) HA - UDL loading

The load per m width of sheet pile is:

\[ \frac{1}{2} \times 1.5 \times 38.4 \times 19 \times \frac{1}{3} \]

\[ = 182.4 \text{ kN per m width} \]

(ii) HA - KEL Loading

The knife edge load = \[ 1.5 \times 99 \times \frac{1}{3} \]

\[ = 49.5 \text{ kN per m width} \]

Therefore total HA live loading is:

\[ 182.4 + 49.5 = 231.9 \text{ kN per m width} \]

A.5.3 Total axial load

The total axial design load at the top of the pile abutment is given by the summation of the dead and the live load, i.e.

\[ 168.7 + 231.9 = 401 \text{ kN per m width of sheet pile} \]
A.6 Bridge abutment analysis

Figure A.5  Elevation of sheet pile abutment showing loading

A.6.1 Relevant soil parameters

Angle of effective shearing resistance $\phi'$

- Moderately conservative $\phi' = 37^{\circ}$
- Worst credible $\phi' = 33^{\circ}$

A.6.2 Earth pressure coefficients

The earth pressure coefficients (based on Caquot and Kerisel) for the at-rest, active, and passive states for $\phi' = 37^{\circ}$ and $33^{\circ}$ are given in the table below.

<table>
<thead>
<tr>
<th>Earth pressure coefficient</th>
<th>$\phi'$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$37^{\circ}$</td>
</tr>
<tr>
<td>$k_a$ (at-rest)</td>
<td>0.398</td>
</tr>
<tr>
<td>$k_a$ (active)</td>
<td>0.207</td>
</tr>
<tr>
<td>$k_p$ (passive)</td>
<td>7.923</td>
</tr>
</tbody>
</table>

A.6.3 Water tables

For the bridge abutment under consideration, the water table on both sides of the wall is assumed to be at ground level on the excavated side (see Figure A.5). It is assumed that adequate drainage will be provided, preventing the water level on the retained side rising.
A.6.4 Retained height

The retained height of the bridge abutment is 4.0 m. This is made up of the following:

- Clear height = 2.75 m
- Steel deck beam plus decking etc. = 1.25 m

To allow for any unplanned excavation in front of the abutment, an additional 10% (minimum 0.5 m) of the bridge abutment height is added (BS 8002).

The design retained height for the bridge abutment is therefore:

\[ 4.0 + 0.5 = 4.5 \text{ m} \]

A.6.5 Surcharge

The surcharge acting on the retained side of the bridge abutment is taken to be 10 kN/m (BD 42/BD 37).

A.6.6 Design condition

The most appropriate design condition for a bridge abutment in sand is a

LONG-TERM EFFECTIVE STRESS ANALYSIS.

A.6.7 Horizontal earth pressures

Horizontal earth pressures for the stability analysis are obtained from the ReWaRD 2 analysis program.

A.7 Stability analysis and determination of the depth of embedment

The cantilever sheet pile abutment is assumed to be fixed at the toe. To simplify the calculation procedure, a simplified earth pressure distribution at the toe is assumed. A correction factor \( C_c \), taken to be 0.2, is applied to the depth of embedment calculation.

The depth of embedment is obtained by taking moments about the toe of the pile.

Two methods are used to compare and choose the depth of embedment for both moderately conservative and worst credible soil parameters. They are the Factor on Strength method and the Burland-Potts method.
The factors of safety (FOS) are:

\( F_s \) for the Factor on Strength method  
\( F_r \) for the Burland-Potts method

The stability analyses are performed using the latest version of ReWaRD (Version 2). In the table below, the factors recommended by CIRIA 104 are used to determine the required length of the pile.

<table>
<thead>
<tr>
<th>Method</th>
<th>Moderately conservative ( \phi' = 37^\circ )</th>
<th>Worst credible ( \phi' = 33^\circ )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FOS Pile length (m)</td>
<td>FOS Pile length (m)</td>
</tr>
<tr>
<td>Factor on Strength</td>
<td>1.5 10.7</td>
<td>1.2 10</td>
</tr>
<tr>
<td>Burland-Potts</td>
<td>2 9.6</td>
<td>1.5 9.8</td>
</tr>
</tbody>
</table>

It is proposed that a pile length of 11 m is used for the design. This gives a depth of embedment of:

\[ 11 - 4.5 = 6.5 \text{ m} \]

**A.8 Vertical resistance capacity of sheet pile**

It is assumed that only the soil on the passive side contributes to vertical resistance.

The Standard Penetration Test parameter \( N \) can be used to obtain a minimum size of pile that is suitable for driving. For a medium dense sand \( N \) is 35, therefore from Table 12.1 the minimum wall modulus is 1300 cm\(^3\) per m of wall. Hence the minimum section size for the pile is to be Larssen LX16 or greater. Although a section has not been sized for bending, it is assumed that an appropriate size will be LX16 - LX32.

As the retained height is near the limit of cantilevered sheet pile retaining walls, it has been assumed that a Larssen LX25 will be appropriate.

The surface perimeters of LX16 - LX32 Larssen piles range from 1.77 to 1.8 m\(^2\) per m length of pile. Assuming the minimum value of 1.77 m\(^2\) per m length, the surface area for one side of the sheet pile is \( 1.77/2 = 0.885 \text{ m}^2 \text{ per m length of pile} \). This is based on a single Larssen of width 600 mm.
For a depth of embedment of 6.5 m, the surface area per m width of sheet pile wall is:

\[
6.5 \times 0.885 / 0.6 = 9.6 \text{ m}^2 \text{ per m width}
\]

The cross-section area properties of a range of Larssen sheet piles are:

<table>
<thead>
<tr>
<th>Larssen</th>
<th>Cross-section area ( A_b ) (cm(^2) per m)</th>
<th>Surface area ( A_s ) (m(^2) per m width)(^*)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uncorroded</td>
<td>Corroded*</td>
<td></td>
</tr>
<tr>
<td>LX16</td>
<td>157</td>
<td>110</td>
</tr>
<tr>
<td>LX20</td>
<td>177</td>
<td>124</td>
</tr>
<tr>
<td>LX25</td>
<td>200</td>
<td>140</td>
</tr>
<tr>
<td>LX32</td>
<td>242</td>
<td>169</td>
</tr>
</tbody>
</table>

\(^*\) Based on corrosion of 2 mm on each side of the pile (soil on both sides of the pile), from hand calculations the corroded cross-sectional area is approximately 70% of the uncorroded value.

\(^\dagger\) It is assumed that the corroded surface area of a pile is similar to that of an uncorroded pile.

The unit wall friction capacity \( f_s \) is given by:

\[
f_s = 2N \text{ kN/m}^2
\]

For a medium dense sand with SPT \( N \) value of 35

\[
f_s = 2 \times 35
\]

\[= 70 \text{ kN/m}^2 \text{ per m width}\]

The unit end bearing capacity is given by:

\[
q_b = 400 N_b \text{ kN/m}^2
\]

Therefore the unit end bearing capacity \( q \) is:

\[
400 \times 35 = 14000 \text{ kN/m}^2
\]
The ultimate design bearing capacity of the sheet pile is given by

\[
R_{\text{design}} = \frac{f_s A_s}{\zeta_\gamma_s} + \frac{q A_b}{\zeta_\gamma_b}
\]

where \( \gamma_s \), \( \gamma_b \) are obtained from Table 7.2 of DD ENV 1997-1: 1994.

Therefore:

\[
\gamma_s = 1.3 \\
\gamma_b = 1.3 \\
\zeta = 1.0
\]

(Soil properties assumed to be characteristic values)

(Partial factor \( \gamma_s \) is assumed to be incorporated with \( \gamma_s \) and \( \gamma_b \))

It is assumed that only 80% of the surface area of the pile in the passive zone acts to provide skin friction (British Steel advice). Therefore for corroded section properties

\[
R_{\text{design}} = 9.6 \times 0.8 \times 70 / 1.3 + 14000 \times A_b / 1.3 \text{ kN}
\]

For an axial design force of 401 kN, the worst utilisation factor for the range of Larssen piles LX16 - LX25 sheet piles (corroded section properties) is:

\[
401/532 = 0.8
\]

The depth of embedment is therefore adequate to resist the vertical loading.

### A.9 Structural forces due to soil acting on the wall

The structural forces acting on the sheet pile wall were obtained using the limit equilibrium approach and the ultimate conditions method. The maximum bending moment and shear force were obtained based on the requirements of BD 42, i.e. using worst credible soil properties and the application of a factor equal to 1.0 to the calculated structural forces. ReWaRD 2 analysis software was used.

The magnitudes of the maximum bending moment and shear force calculated based on BD 42 requirements are tabulated below:
The horizontal displacement at the pile head was assumed to be 20 mm.

A.10 Estimation of steel sheet pile section

A.10.1 Bending only

The design bending moment at ultimate limit state due to soil/water pressure is given by:

\[ M_{\text{design}} = \gamma_{fl} M \]

where \( \gamma_{fl} = 1.0 \) (BD 42)

\[ M = \text{moment derived from limit equilibrium analysis using worst credible soil parameters and surcharge on the retained side.} \]

The total bending moment acting on the sheet pile comprises the sum of the bending moment due to soil/water pressures and a component arising from eccentricity of the bridge deck reaction acting at the top of the pile. A bending moment is produced from this reaction because the pile displaces horizontally and the point of action of the vertical load does not act directly on the neutral axis of the sheet pile section.

Using the worst maximum bending moment obtained from the ReWaRD analysis of 147 kNm (at zero shear force):

\[ M_{\text{design}} = 1.0 \times 147 \text{ kNm} = 147 \text{ kNm} \]

The total design bending moment acting on the steel bridge abutment taking into account forces from the foundations and the bridge superstructure is given by:

\[ M_{\text{total}} = M_{\text{design}} + P_{\text{design}} (y_p + e_p) \]

where \( y_p = 20 \text{ mm (assumed displacement)} \)

\( e_p = 50 \text{ mm (assumed eccentricity of load)} \).
Figure A.6 Eccentricity of loading for a cantilever abutment

Therefore

\[ M_{\text{total}} = 147 + 401 (20 \times 10^3 + 50 \times 10^3) \]

\[ = 147 + 28 \]

\[ = 175 \text{ kNm per m width of pile} \]

As the bending moments for both corroded and uncorroded sections are similar (ReWaRD does not take into account the stiffness of the pile for cantilevers), the design strength check will be performed on the corroded section properties.

Referring to BS 5400: Part 3: Clause 9.9.1.2:

The moment capacity of a sheet pile section \( M_D \) is given by:

\[ M_D = \frac{Z \sigma_{lc}}{\gamma_m \gamma_{f3}} \]

where

\[ Z = \text{elastic section modulus of the pile} \]

\[ \gamma_m = 1.2 \text{ (BS 5400: Part 3, table 2)} \]

\[ \gamma_{f3} = 1.1 \]

\[ \sigma_{lc} = \sigma_f = 270 \text{ N/mm}^2 \text{ (Grade S270GP).} \]

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Using the corroded sheet pile section properties (maximum bending moments for uncorroded and corroded sections are very similar) gives utilisation factors \( M/M_D \) of:

<table>
<thead>
<tr>
<th>Corroded section properties</th>
<th>Z (cm³/m)</th>
<th>( M_D ) (kNm/m)</th>
<th>M (kNm/m)</th>
<th>Utilisation factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Larssen</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LX16</td>
<td>1000</td>
<td>205</td>
<td>175</td>
<td>0.9</td>
</tr>
<tr>
<td>LX20</td>
<td>1325</td>
<td>271</td>
<td>175</td>
<td>0.7</td>
</tr>
<tr>
<td>LX25</td>
<td>1800</td>
<td>368</td>
<td>175</td>
<td>0.5</td>
</tr>
<tr>
<td>LX32</td>
<td>2525</td>
<td>516</td>
<td>175</td>
<td>0.3</td>
</tr>
</tbody>
</table>


In bending alone, adequate sections include LX16 Larssen sheet pile or higher.

A.10.2 Shear

The shear force at ultimate limit state due to soil/water pressure is given by:

\[
V_{\text{design}} = \gamma_f V
\]

where \( \gamma_f = 1.0 \) (BD 42)

\[
V = \text{shear force derived from limit equilibrium analysis using worst credible soil parameters and surcharge on retained side.}
\]

From the ReWaRD analysis, the unfactored maximum shear force for a corroded or uncorroded Larssen sheet pile is 208 kN.

The maximum design shear force is given by:

\[
V_{\text{design}} = 1.0 \times 208 \text{ kN} = 208 \text{ kN}
\]

The shear resistance from BS 5400: Part 3: Clause 9.9.2.2 is given by:

\[
\left( \frac{t_w(d_w - h_h)}{\gamma_m\gamma_f} \right) t_i
\]
For a LX20 Larssen section

\[ t_w = 9 - 4 = 5 \text{ mm (corroded section)} \]
\[ d_w = 405 \text{ mm} \]
\[ h_h = 0 \]
\[ \gamma_{f3} = 1.1 \]
\[ \gamma_{m} = 1.05 \]

where

\[ \lambda = \frac{d_w \sqrt{\gamma_{m}}}{t_w} \]

Therefore

\[ \lambda = \frac{405}{5} \sqrt{\frac{270}{355}} = 71 > 50 \]

and as \( m_{f_w} = 0 \) and \( \phi > 50 \)

from BS 5400: Part 3, page 33, figure 11

\[ \tau_i / \tau_f = 0.84 \]

Therefore

\[ \tau_i = 0.84 \tau_f = 0.84 \frac{270}{\sqrt{3}} = 130.9 \text{ N/mm}^2 \]

\[ V_D = (1/0.6) \times 5.0 \times 405 \times 130.9 \times 10^{-3}/(1.05 \times 1.1) \]
\[ = 383 \text{ kN per m width} \]

For pure shear only, the utilisation factor is:

\[ 208/383 = 0.54 \]

A Larssen LX20 pile section is adequate to resist the shear stresses.

A.10.3 Combined bending and shear

From the bending moment and shear force output from ReWaRD, using trial and error, the worst combination of bending moment and shear force acting together is:

Bending moment \( = 120 \text{ kNm per m width} \)

Shear force \( = 67 \text{ kN per m width} \)

acting 7.0 m from the top of the pile.
The total design bending moment including moment due to eccentricity of loading (conservatively taken as for the maximum bending moment case) is:

\[ M = 1.0 \times 120 + 28 = 148 \text{ kNm per m width} \]

The design shear force is given by:

\[ V = 1.0 \times 67 = 67 \text{ kN} \]

From BS 5400: Clause 9.9.3:

\[ V_D = 383 \text{ kN per m width (as previously)} \]
\[ V_R = V_D \text{ (as } m_{Fw} = 0) = 392 \text{ kN per m width} \]

\[ M_D = \frac{Z\alpha}{\gamma_m \gamma_{F3}} = \frac{1325 \times 10^3 \times 270}{1.2 \times 1.1} = 271 \text{ kNm} \]

\[ M_R = \frac{F_f d_f}{\gamma_m \gamma_{F3}} = \frac{929 \times 0.418 \left( \frac{1}{1.2} \right)}{1.2 \times 1.1} = 267 \text{ kNm} \]

where \[ F_f = 405 \times (12.5 - 4) \times 270 = 929 \text{ kN} \]
\[ d_f = 430 - 2 \times 12.5/2 = 417.5 \text{ mm} \]

As \( V \leq V_D, M \leq M_D, \text{ and } M \leq M_R, \) a combined bending and shear check is not required.

A LX20 sheet pile is adequate to resist combined bending and shear stresses.

A.10.4 Combined bending and axial load

1. Yielding of pile

\[ M = 175 \text{ kNm per m} \]
\[ F = 401 \text{ kN per m} \]
\[ A_e = 124 \text{ cm}^2 \text{ per m} \]
\[ Z_Y = 1325 \text{ cm}^2 \text{ per m} \]

From BS 5400: Part 3: Clause 9.9.4.1, the following is to be satisfied:

\[ \frac{P}{A_c} \frac{M_x}{Z_x} + \frac{M_y}{Z_y} \leq \frac{\sigma_y}{\gamma_m \gamma_{F3}} \]
Therefore \[
\frac{401 \times 10^4}{124 \times 10^2} + \frac{175 \times 10^6}{1325 \times 10^3} \leq \frac{270}{1.2 \times 1.1}
\]

and \[32 + 132 = 164 < 205\]

Therefore the condition is satisfied and the LX20 pile is adequate to resist the combination of bending and axial stresses.

2. Buckling of pile

The equivalent length of the pile is obtained from table 10, page 73 of BS 5400: Part 3, where

\[l_e = 0.7L\]

and \(L\) is the pile length corresponding to a factor of safety \((F_s)\) for stability equal to 1.0.

A stability analysis performed using ReWaRD with a factor of safety of 1.0 gave the length of the pile \(L\) to be 8.7 m.

Therefore \[l_e = 0.7 \times 8.7 = 6.1 \text{ m}\]

The radius of gyration \(r\) of the section is given by:

\[r = \sqrt{\frac{I}{A}} = \sqrt{\frac{1325 \times 21.5}{124}} = 15.1 \text{ cm}\]

where \(I\) = second moment of area of the section

\(A\) = cross-sectional area of the section

\[\frac{r}{y} = \frac{15.1}{21.5} = 0.70\]

\[\frac{l_e}{r} \sqrt{\frac{\sigma_e}{355}} = \frac{6100}{151} \sqrt{\frac{270}{355}} = 35.2\]

From Figure 37, page 75, using curve A:

\[\frac{\sigma_e}{\sigma_y} = 0.94\]

and \[\sigma_e = 0.94 \times 270 = 254 \text{ N/mm}^2\]
Referring to Clause 9.9.4.2 of BS 5400: Part 3

\[
\frac{P_{\text{max}}}{P_D} + \frac{M_{\text{max}}}{M_{Dx}} + \frac{M_{\text{max}}}{M_{Dy}} \leq 1
\]

where \( P_D \) is given by:

\[
P_D = \frac{A \sigma_c}{\gamma_m \gamma_f} = \frac{124 \times 10^2 \times 254 \times 10^{-3}}{1.2 \times 1.1} = 2386 \text{ kN}
\]

Hence

\[
\frac{401}{2386} + \frac{175}{271} = 0.17 + 0.65 = 0.82 < 1.0
\]

Larssen section LX20 is therefore adequate to resist buckling.

THE SOLUTION FOR THIS CANTILEVER BRIDGE ABUTMENT IS:
LX20 LARSSEN SHEET PILE, LENGTH 11.0 m,
MATERIAL TYPE S270GP.