VOLUME 2 HIGHWAY STRUCTURES: DESIGN (SUBSTRUCTURES, SPECIAL STRUCTURES AND MATERIALS)

SECTION 1 SUBSTRUCTURES

PART 2

BD 42/00

DESIGN OF EMBEDDED RETAINING WALLS AND BRIDGE ABUTMENTS

SUMMARY

This Standard implements the relevant parts of BS 8002 Annex B and CIRIA Report 104 but adopts limit state principles, compatible with BS 5400. It extends the scope of CIRIA Report 104 to include walls embedded in firm clay and granular materials. The Standard includes steel sheet piled walls in aggressive and non-aggressive environments (as classified in Tables 5.1 and 5.2), concrete diaphragm walls and various forms of concrete piled walls. Embedded walls which are unpropped, propped or anchored at either the top or at excavation level, and doubly-propped/anchored are covered. This Standard includes requirements for reinforcement of cast-in-place embedded retaining walls.

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2. Insert new contents pages for Volume 2, dated May 2000.

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THE HIGHWAYS AGENCY

THE SCOTTISH EXECUTIVE DEVELOPMENT DEPARTMENT

THE NATIONAL ASSEMBLY FOR WALES
CYNULLIAD CENEDLAETHOL CYMRU

THE DEPARTMENT FOR REGIONAL DEVELOPMENT*

Design of Embedded Retaining Walls and Bridge Abutments

* A Government Department in Northern Ireland

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Contents

1. INTRODUCTION

1.1 In urban and semi-urban areas, it is often necessary to construct roads on retained embankments or in retained cuttings, or to build cut-and-cover tunnels. This has meant an increase in the use of free or propped cantilever embedded retaining walls which has coincided with the advent of new piling techniques and slurry trench methods of in-situ wall construction.

1.2 The design of embedded walls differs from backfilled retaining walls with spread footings, as the designer does not have a choice of the retained material. Therefore, earth pressures generated by whatever retained soil is in situ have to be accommodated. A further distinction is that embedded walls usually have a more stringent serviceability condition imposed upon them because of the greater risk of damage to existing neighbouring structures, carriageways and underground services since they will be closer to such items.

1.3 The recommendations contained in Annex B of BS 8002 Earth Retaining Structures and CIRIA Report 104, Design of Retaining Walls Embedded in Stiff Clay, (1) are considered to be excellent guidance on this subject. The design of retaining walls embedded in over-consolidated clay has been the subject of wide-ranging research by Transport Research Laboratory (TRL) and others and their findings are incorporated in this Standard. This Standard states how to use Annex B of BS 8002 and CIRIA Report 104 in a way which is compatible with the limit state principles adopted for BS 5400, Steel, Concrete and Composite Bridges as implemented by the relevant DMRB Standards. This Standard also gives criteria for the use of steel sheet piling in permanent retaining walls and bridge abutments.

1.4 CIRIA Report 104 is only concerned with stiff clay. TRL Research Report 359 provides a valuable commentary on the approach recommended by Annex B of BS 8002 and CIRIA Report 104. More recently Eurocode 7 (1994) has been issued (as a draft for development) giving advice on geotechnical design and there are analogies between the factor on soil strength employed in some of the earlier design approaches and the partial factors on ground properties employed in the Eurocode. TRL Report 320 provides a comparison of the design of embedded walls using BS 8002, CIRIA Report 104 and Eurocode 7.

1.5 The purpose of this Standard is to draw together recent advances in practice, based on simple limit equilibrium methods, and to set out the requirements of design and construction to achieve greater consistency in design approach. In most situations specialised knowledge and experience of soil-structure interaction will be required.

Equivalence

1.6 The construction of embedded retaining walls and bridge abutments will normally be carried out under contracts incorporating the Specification for Highway Works (MCHW 1). In such cases products conforming to equivalent standards or technical specifications of other states of the European Economic Area and tests undertaken in other states of the European Economic Area will be acceptable in accordance with the terms of Clauses 104 and 105 of Series 100 of that Specification. Any contract not containing these Clauses shall contain suitable clauses of mutual recognition having the same effect regarding which advice should be sought.

Scope

1.7 This Standard applies to earth retaining structures where the main stability is provided by a significant length of wall stem embedded in the ground. For the purposes of this Standard, embedded retaining walls are walls which are usually formed by first constructing the wall in undisturbed ground and then by excavating on one side to the required depth.

1.8 This Standard is based on simple limit equilibrium methods of analysis. However, innovation and the use of advanced numerical analytical methods is equally permissible although outside the scope of this Standard.

1.9 This Standard does not cover vertical load capacity. Design for the vertical load capacity of the walls and individual piles shall be in accordance with BS 8004 Foundations as implemented by BD 74 (DMRB 2.1.8). It should be noted that vertical loading on the wall might reduce or reverse the direction of wall friction and result in increased earth pressure on the retained side. Biddle (1997) gives advice on vertical load capacity of steel sheet pile walls.

1.10 This Standard covers embedded retaining walls constructed by the use of steel sheet piles in environments with corrosivity classifications of aggressive and non-aggressive (as classified in Tables 5.1 and 5.2), concrete diaphragm walls, secant piles and contiguous piles. Structures, which are unpropped or propped at the top, are covered in Chapters 2 and 3. Additional requirements for structures which have a single prop near excavation level are covered in Chapter 7, those for doubly-propped structures are covered in Chapter 8, and those for structures with a stabilising base are given in Chapter 9.

1.11 Guidance is given for the design of permanent walls in over-consolidated stiff or firm clay and also in granular materials. Walls in soft clay are excluded. Both overall stability and structural design are considered.

1.12 This Standard is not applicable to walls with spread foundations. However, embedded retaining walls installed through and supporting existing backfilled materials, as for bridge abutments in embankments are covered. Special requirements for partially backfilled embedded retaining walls are given in Clauses 6.2 and 6.3 of this Standard.

1.13 Design of ground anchorages shall be in accordance with BS 8081 and Clause 6.5 of this Standard. Ground anchors should be used with caution since the consequences of their failure may be serious.

1.14 Guidance is given on the use of the Observational Method (Chapter 10) and the use of soil berms as a method of temporary support (Chapter 11). However, temporary walls are outside the scope of this Standard.

Implementation

1.15 This Standard shall be used forthwith on all schemes for the construction and improvement of trunk roads, including motorways, currently being prepared, provided that, in the opinion of the Overseeing Organisation this would not result in significant additional expense or delay progress. Design Organisations shall confirm its application to particular schemes with the Overseeing Organisation. In Northern Ireland, the use of this Standard will apply on those roads designated by the Overseeing Organisation.

Mandatory Requirements

1.16 Sections of this document which form mandatory requirements of the Overseeing Organisation are highlighted by being contained within boxes. The remainder of the document contains advice and enlargement which is commended to designers for their consideration.

2. DESIGN PRINCIPLES FOR STRUCTURES WHICH ARE UNPROPPED OR PROPPED AT THE TOP

2.1 The design of embedded cantilever retaining walls shall be carried out using limit state design principles. The limit states to be considered are described in Clauses 2.2 to 2.9 affecting the soil, adjacent and supported structures and the structural elements.

Ultimate Limit State of Overall Stability

2.2 In this limit state, only the stability of the soil/structure cross-section is considered and the design is required to prevent the types of failure described in Clauses 2.3 and 2.4, and illustrated in Figure 2.1.

2.3 Collapse can occur as failure by rotation about the position of a prop or anchorage, if present or about a point near the toe of the wall if there is no prop or anchorage. Such failures are prevented by providing a suitable depth of embedment, with or without propping or anchoring.

2.4 Collapse of an embedded wall can also occur if the whole mass of soil around the structure fails by a deep-seated slip failure.

Serviceability Limit State of Deformation

2.5 This limit state represents the deformation limits beyond which unacceptable tilting and deformation of the wall occur.

Ultimate and Serviceability Limit States of Adjacent and Supported Structures

2.6 These limit states represent the limits of acceptable soil movement from consideration of the effects on supported buildings, statutory undertakers' equipment, carriageways, other structures etc. The movements considered shall be those induced by deformation of the soil and of the wall elements at both ultimate and serviceability limit states. It is necessary to consider both the global movements occurring in response to mass excavation and changes in ground water levels and the local movements near to the boundaries of the excavation caused by swelling or shear deformation of the ground and deflections of the walls. The magnitude and pattern of local movements are dependent on the construction method, on the position and effectiveness of any temporary and permanent props and on the flexibility of the wall. Both short and long term movements shall be considered.

Ultimate Limit State of Structural Elements

2.7 This limit state corresponds with the failure of the stem of the wall in bending or shear. The definitions and requirements of this limit state are as given in BS 5400: Parts 3, 4 and 5 (implemented by BD 13 (DMRB 1.3), BD 24 (DMRB 1.3.1) and BD 16 (DMRB 1.3)) for steel, concrete and composite elements. Tyson (1995) considers the development of the philosophy for designing reinforcement in piles and embedded walls and the applicability of the current design codes. Requirements for the reinforcement of cast-in-place embedded retaining walls, based on this work, are given in Annex A of this Standard.

2.8 This limit state is also applicable to the total failure of props in compression and anchorages in tension, if included in the design.

Serviceability Limit State of Structural Elements

2.9 The definitions and requirements for this limit state are given in BS 5400. Parts 3, 4 and 5, (implemented by BD 13 (DMRB 1.3), BD 24 (DMRB 1.3.1) and BD 16 (DMRB 1.3)) for steel, concrete and composite elements. Requirements for the reinforcement of cast-in-place embedded retaining walls, based on Tyson (1995), are given in Annex A of this Standard.

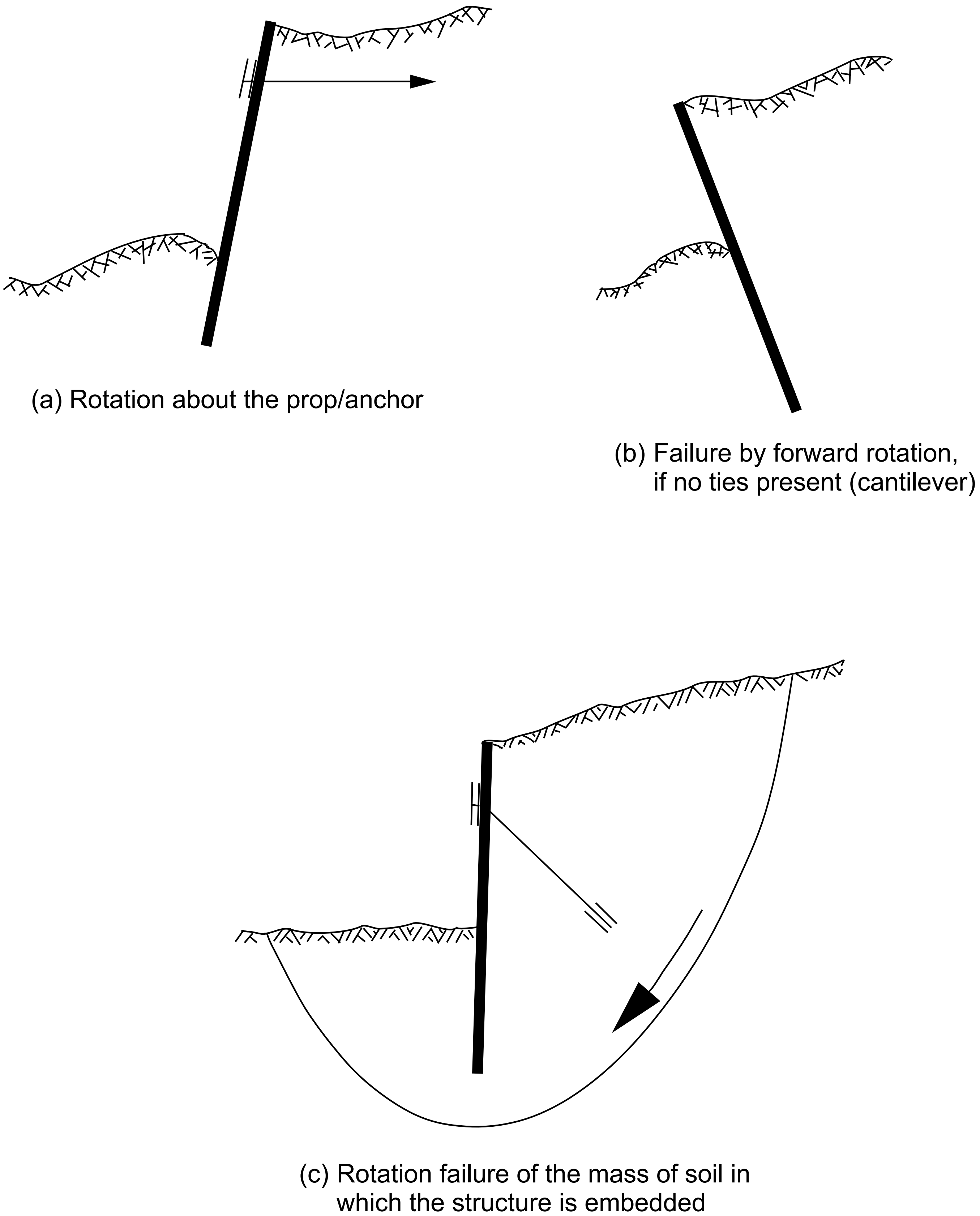


Figure 2.1: Overall Stability - Failure Conditions

Design Life

2.10 All permanent highway structures are required to have a design life of 120 years.

Construction and Long Term Effects

2.11 Both long and short-term conditions shall be considered so that the final design satisfies the most onerous requirements encountered during both the construction period and design life of the structure.

Construction Sequence and Temporary Loads

2.12 The design shall accommodate the intended method and sequence of construction with separate checks on the adequacy of the design made at each stage. This will require that consideration be given at the design stage to the sequence of installing both temporary and permanent supports and to the effect of both permanent and temporary loads. Allowance shall be made in the design for excavation below the formation level in front of the wall for installation of drains and services, for any over-excavation that may be involved in the construction and also for any temporary increases in loading on the retained ground during construction. The design shall consider the possibility of outward rotation of the top of the wall resulting from ingress of water into tension cracks in clay soils formed in the retained ground at the soil structure boundary. Note a water-filled crack can theoretically extend to a very much greater depth than a dry crack. The effect of ground movements caused by the method and sequence of construction on adjacent buried services and structures shall also be considered. The sequence of construction, which forms the basis of wall design, shall be made known to the contractor. Each stage of the operation shall be clearly defined to remove any ambiguity or overlap of activities.

Post-Construction Changes in Loading

2.13 The design shall consider the limit states of the structure and soil and of adjacent and supported structures under long term equilibrium conditions associated with the new ground water flow pattern imposed by the wall construction (CIRIA Report 104, Section 3.6). This will require the assessment of the equilibrium pore water pressure distribution in the vicinity of the wall. Consideration shall be given to the effects of any long term rise in the natural water table, for example general rises caused by the cessation of pumping from underlying aquifers or local rises resulting from malfunction of drainage works. Allowance shall also be made for any excavation in front of the wall, necessary for reinstatement or maintenance of drains and services, and for any likely increases in loading imposed on the retained ground behind the structure. Consideration shall also be given to the effects of future development.

3. DESIGN OF STRUCTURES WHICH ARE UNPROPPED OR PROPPED AT THE TOP

Ultimate Limit State of Overall Stability

3.1 Design for the overall stability of the soil/structure shall be carried out in accordance with the recommendations of CIRIA Report 104 Design of Retaining Walls Embedded in Stiff Clay (which are also applicable in principle to granular soils) and BS 8002, Earth Retaining Structures. These design methods use ‘lumped’ factors of safety applied to unfactored loads (not partial load factors). In the context of CIRIA Report 104 these factors are described as ‘recommended factors of safety’ (CIRIA Report 104 Table 5, p73). In using CIRIA Report 104, the following additional provisions shall apply:

i. To establish the required depth of embedment, design shall be carried out using the ‘Moderately Conservative Parameter’ approach (Design Approach A of Table 5, CIRIA Report 104). Moderately conservative soil strength parameters are considered to be close to the representative or conservative values required by BS 8002 and also the characteristic values used as a cautious estimate of the mean in Eurocode 7. A final check on the depth of embedment shall also be carried out using the ‘worst credible’ soil parameters and ground water conditions (Design Approach B of Table 5, CIRIA Report 104).

ii. Design shall be based on one of the recommended methods in Table 5 of CIRIA Report 104 and checked using at least one of the other recommended methods (see also 3.1 iv). The adoption of ‘recommended factors of safety’ with this procedure will normally satisfy the serviceability limit state of deformation (Clause 3.2) but see Clause 3.3 for supported structures.

iii. The ‘recommended factors of safety’ given in Table 5 of CIRIA Report 104, for any given method, shall be adopted. Due note shall be taken of the relationship between the factor of safety and Ø’ as recommended in Table 5. For example, in the case of the “CP2 Method of CIRIA Report 104”, the appropriate factor of safety depends on the value of Ø′ i.e. where it is stated in Table 5 that for Ø′ = 20° to 30°, for permanent works using Design Approach A, F_p = 1.5 to 2.0, it would be appropriate to adopt F_p = 1.5 for Ø′ ≤ 20°, F_p = 2.0 for Ø′ ≥ 30°, and interpolate for intermediate values of Ø′.

iv. Although not described in CIRIA Report 104, the method described as the “Net Total Pressure” method may also be used, but only in predominantly granular soil conditions, provided worst credible soil parameters are used in the calculations. A factor of safety of not less than 2 is required. The design shall be checked against at least one of the recommended methods in CIRIA Report 104.

v. Unfactored nominal values of the highway loading, as given in BD 37 (DMRB 1.3) shall be applied at the relevant points and at the appropriate levels, in combination with the unfactored permanent and other loads as required by BD 37 (DMRB 1.3). The partial load factors for earth pressure, given in BD 37 (DMRB 1.3) shall not be applied in these loading combinations because a lumped factor of safety approach is to be adopted as described in 3.1 (iii). Live load surcharge shall be applied on the retained side of the wall, as described in Clause 5.8.2 of Appendix A of BD 37 (DMRB 1.3).

vi. A method for calculating active earth pressures for localised surcharges, is given in paragraph 9.3 of the Institution of Structural Engineers report ‘Soil Structure Interaction - the Real Behaviour of Structures’ (1989).

Serviceability Limit State of Deformation

3.2 The adoption of 'recommended safety factors' at the ultimate limit state for overall stability of the embedded wall should also prevent undesirable tilting and deformation, as described in Clause 3.1 (ii) and (iv).

Ultimate and Serviceability Limit States of Adjacent and Supported Structures

3.3 The likely ground movements shall first be assessed and calculated from relevant field data and from experience of similar structures in similar ground conditions. A database of the magnitudes of ground and wall movements measured at sites of bored pile and diaphragm retaining wall construction has been established by Carder (1995). Similar databases have also been presented by Clough and O'Rourke (1990) and Fernie and Suckling (1996). A deformation analysis shall be carried out to determine the ground movements. The analysis shall take into account the effects outlined in Clause 2.6. The usefulness of current computer programs is reviewed in publications such as Ground Engineering and the reader is directed to these. Deformation of the structural elements including that of props and anchorages shall be taken into account for these limit states.

3.4 Soil deformation analysis shall be carried out for both the construction and working load stages. Nominal values of dead and highway live loading, as given BD 37 (DMRB 1.3) shall be used in the calculations where necessary.

Ultimate Limit State of Structural Elements

3.5 Design of the structural elements for this limit state shall be carried out in accordance with BS 5400: Parts 3, 4 and 5, (implemented by BD 13 (DMRB 1.3), BD 24 (DMRB 1.3.1) and BD 16 (DMRB 1.3)) for steel, concrete and composite elements, except that the design load effects shall be calculated as described below, and not by using the partial load factors (γ_{fL}) for earth pressures given BD 37 (DMRB 1.3). Tyson (1995) considers the development of the philosophy for designing reinforcement in piles and embedded walls and the applicability of the current design codes. Requirements for the reinforcement of cast-in-place embedded retaining walls, based on this work, are given in Annex A of this Standard.

3.6 The bending moments and shear forces developed in the wall shall be calculated in accordance with Section 8 of CIRIA Report 104 except that the earth pressures used shall be calculated using worst credible strength parameters in conjunction with worst credible ground water conditions, geometry and loading. A limit equilibrium analysis shall be used to obtain the load effects. The load effects due to earth pressures ie moments, and shear forces in the wall, shall be multiplied by a partial load factor (γ_{fL}) of 1.0 and then by the appropriate value of γ_{f3} from BS 5400: Parts 3, 4 and 5 (implemented by BD 13 (DMRB 1.3), BD 24 (DMRB 1.3.1) and BD 16 (DMRB 1.3)). Where failure occurs in bending, the shear forces are likely to be less than those experienced at the serviceability limit state (see Clause 3.9). However, where soil conditions are outside the scope of CIRIA Report 104, the following shall be considered in calculating the bending moments and shear forces. Where a sheet, contiguous bored or secant pile is embedded into strata that, as an upper bound, could be sufficiently stiff to produce earth pressure at rest over the retained height, care should be taken to ensure these is compatibility between structure deformation and soil strain especially where there is additional support from prestressed or benign anchors. Where the embedded strata is relatively softer and the retained fill relatively stiffer, account should be taken of the possible rotation of parts of the wall, particularly where there is additional support from ground anchors or propslabs.

3.7 All the relevant highway loading, including parapet loads, shall be applied in accordance with BD 37 (DMRB 1.3). Live load surcharge shall be applied on the retained side of the wall as described in Clause 5.8.2 of Appendix A of BD 37 (DMRB 1.3).

3.8 Axial forces applied to props, assumed to be pinned, and to ground anchorages shall be determined in accordance with Section 8.5.2 of CIRIA Report 104, except that the earth pressures applied shall be calculated using worst credible strength parameters in conjunction with worst credible geometry, ground water conditions and loading. A partial load factor (γ_{fL}) of 2.0 shall be used, multiplied by the appropriate value of (γ_{f3}) for concrete or steel construction, to obtain the design axial force in props at this limit state. (This is instead of the factor of safety of at least 2 required in CIRIA Report 104: Section 8.5.2). For ground anchorages reference shall be made to Clause 6.5 of this Standard for the factors of safety to be applied.

Serviceability Limit State of Structural Elements

3.9 BS 5400: Parts 3, 4 and 5 (as implemented by BD 13 (DMRB 1.3), BD 24 (DMRB 1.3.1) and BD 16 (DMRB 1.3)) do not include a method for designing structural elements for this limit state. The earth pressures mobilised in the retained ground at small deformations under working conditions corresponding to the serviceability limit state may well exceed the earth pressures mobilised at larger ground deformations corresponding to the ultimate limit state. This is likely to be the case particularly for stiff walls in over-consolidated clays where large in situ lateral earth pressures exist in the ground prior to construction. Therefore to avoid any possible underdesign of the structural elements, the designer shall also consider the structural loading effects caused by the earth pressures acting on the structure at the serviceability limit state. The design load effects shall be calculated using the partial load factors for earth pressures given below and not the factors given in BD 37 (DMRB 1.3). Requirements for the reinforcement of cast-in-place embedded retaining walls, based on Tyson (1995), are given in Annex A of this Standard.

3.10 For this limit state an assessment shall also be made of the greatest pressures and loads likely to act on the structure during its design life, taking into account the changes from the initial in situ earth pressure coefficient, caused by the installation of the wall, excavation in front of the wall and any long term changes in pore water pressures. Worst credible strength parameters shall be adopted and a limit equilibrium method of analysis used to obtain the load effects.

i. __**For cantilever walls in all soils**__

The moments and shear forces shall be calculated in accordance with Section 8 of CIRIA Report 104. A partial load factor (γ_{fL}) of 1.0 shall be applied to the calculated moments and shears except for stiff walls in heavily over-consolidated clays where a partial load factor (γ_{fL}) of 1.3 shall be used for the permanent condition. In all cases, after applying the appropriate partial load factor (γ_{fL}) (as described above), the design loads shall be multiplied by the appropriate value of γ_{f3} from BS 5400: Parts 3, 4 or 5 (implemented by BD 13 (DMRB 1.3), BD 24 (DMRB 1.3.1) and BD 16 (DMRB.1.3)), to obtain the design load effects.

ii. __**For walls propped at the top in soils other than heavily over-consolidated clays**__

The moments and shear forces shall be calculated in accordance with Section 8 of CIRIA Report 104. A partial load factor (γ_{fL}) of 1.0 shall be applied to the calculated moments and shears. In all cases, after applying the appropriate partial load factor (γ_{fL}) (as described above), the design loads shall be multiplied by the appropriate value of γ_{f3} from BS 5400: Parts 3, 4 or 5 (implemented by BD 13 (DMRB 1.3), BD 24 (DMRB 1.3.1) and BD 16 (DMRB.1.3)) to obtain the design load effects.

iii. __**For walls propped at the top in heavily over-consolidated clays**__

Where the initial in situ earth pressure coefficient K_o is likely to be high (greater than 1.5), the earth pressures acting on the retained side in the long term shall be determined from a limit equilibrium analysis assuming full passive pressures with full wall friction, acting on the excavated side. If the earth pressures calculated on the retained side, using this method, are in excess of those that would be obtained using a K of 1.5, the latter value shall be adopted to allow for some stress relief. The moments and shears calculated by this method are appropriate for stiff walls and stiff props and are therefore likely to be upper bounds. Accordingly a partial load factor (γ_{fL}) of 1.0 shall be applied to the calculated moments. In these cases, after applying the appropriate partial load factor (γ_{fL}) (as described above), the design loads shall be multiplied by the appropriate value of γ_{f3} from BS 5400: Parts 3, 4 or 5 (implemented by BD 13 (DMRB 1.3), BD 24 (DMRB 1.3.1) and BD 16 (DMRB 1.3)), to obtain the design load effects.

3.11 Unfactored nominal values of all the relevant highway loading shall be applied in accordance with BD 37 (DMRB 1.3). Live load surcharge shall be applied on the retained side of the wall, as described in Clause 5.8.2 of Appendix A of BD 37 (DMRB 1.3).

3.12 Axial forces applied to props, assumed to be pinned, and to ground anchorages shall be calculated for a limit equilibrium condition as in CIRIA Report 104, Section 8 but using the pressure distributions described in Clause 3.10 of this Standard. In the case of props a partial load factor (γ_{fL}) of 1.5 shall be applied multiplied by the appropriate value of γ_{f3} to obtain the design axial force but the factor of safety of at least 2 required by CIRIA Report 104 shall not be used. However, for ground anchorages reference shall be made to Clause 6.5 of this Standard for the factors of safety to be applied.

4. MATERIAL AND CONSTRUCTION

4.1 The Overseeing Organisation's requirements for piling and diaphragm walling are contained in MCHW 1 Series 1600. Requirements for concrete are contained in MCHW 1 Series 1700 and requirements for structural steel in MCHW 1 Series 1800.

4.2 The Overseeing Organisation's requirements for ground anchorages are contained in BS 8081 and MCHW 1 Series 600. Clauses 6.4, 6.5 and 6.6 of this Standard are also relevant.

Drainage

4.3 An effective drainage system shall be incorporated whenever possible. Such drainage can be in the form of counterfort drains or trench drains containing suitable filter material, for which the Overseeing Organisation's requirements are contained in MCHW 1 Series 500. Tunnels have internal drains and possibly external roof drains. Drains shall be oversize to allow for blockage during working life to enable access for rodding. Suitable exits with positive outfall shall be provided for any drainage system. Account shall be taken, in the long-term design, of any possible rise in pore water pressures caused by malfunction of the drainage system or resulting from fracture of water mains located in the vicinity. Care shall be taken to avoid damage to adjacent structures due to any depression of the water table. The influence of permanent drainage on water pressures in the backfill or retained soil is discussed in CIRIA Report 104 Section 5.4 and particularly in Section 5.4.3. Consideration shall be given to rising groundwater levels, particularly in urban environments, and their effect on the long-term magnitude and distribution of water pressures.

Construction tolerances

4.4 The designer shall consider the effect construction tolerances of the embedded retaining wall, as specified in MCHW 1 Series 1600, will have on the clearances from carriageways, fixing of claddings, and cover to reinforcement and shall make sufficient allowances in dimensioning the design.

5. DURABILITY

Concrete Piles and Diaphragm Walls

5.1 The minimum requirements for the strength and durability of concrete in the hardened state shall be decided from consideration of BD 57 (DMRB 1.3.7) "Design for durability" and "Concrete - General Requirements" contained in Notes for Guidance on the Specification for Highway Works (MCHW 2), Series 1700. Consideration shall be given to the effect construction tolerances that excavations for embedded retaining wall, as specified in MCHW 1 Series 1600, will have on the minimum cover to reinforcement as required by BD 57 and sufficient allowance shall be made in dimensioning the design. Special consideration shall be given to the effect on durability of breaking out concrete on the remaining cover to the reinforcement.

5.2 Guidance on measures to mitigate the effects of sulfate attack on concrete is contained in BRE Digest 363 "Sulfate and acid resistance of concrete in the ground." However, this has now been extended and to some extent superseded by the DETR (1999) Report of the Thaumasite Expert Group "The thaumasite form of sulfate attack." Advice should be sought from both documents.

Steel Piles

5.3 All steel piles shall be designed with sacrificial thicknesses applied to each surface depending on the exposure conditions to provide a design life of 120 years. The thicknesses described are based on the rates of corrosion given in BS 8002.

5.4 For steel piles embedded in natural undisturbed soils (except peat), the sacrificial thickness on each relevant surface of the pile shall be 2mm.

5.5 For steel piles, embedded in disturbed soils, peat or contaminated ground, the surrounding soil shall be classified using the aggressiveness points system for each property as shown in Table 5.1. The ground water, water and/or effluent in contact with steel piles (if any) shall be classified according to the highest aggressivity indicated for any property in Table 5.2. The tests shall be carried out using the methods given in Table 5.3. When sampling soil for organic content testing, great care shall be taken to avoid contamination with topsoil, roots or overlying made ground. The overall corrosivity classification applicable to the piles shall be the more severe of the aggressivities indicated by Tables 5.1 and 5.2 respectively.

5.6 For steel piles, embedded in disturbed soils, peat or contaminated ground, where the overall corrosivity classification is 'non - aggressive', the sacrificial thickness on each relevant surface of the pile shall be 2mm.

5.7 For steel piles, embedded in disturbed soils, peat or contaminated ground, where the overall corrosivity classification is 'aggressive', the sacrificial thickness on each relevant surface of the pile shall be 4mm.

5.8 For steel piles, embedded in disturbed soils, peat or contaminated ground, where the overall corrosivity classification is 'very aggressive', specialised advice shall be sought on the practicality of protecting the steel so that a design life of 120 years can be ensured. Guidance on methods of increasing the effective life of unprotected steel piles is given in Clause 4.4.4.4.3.5 of BS 8002 (1994).

5.9 For steel piles exposed to the atmosphere, the sacrificial thickness on each exposed surface of the pile shall be 4mm unless a 'very aggressive' atmospheric environment is present. This can be ascertained by consulting the map 'Relative Values of Acid Deposition in the United Kingdom 1986 - 1991' published by ADAS, Reading (2), supplemented by observations on site. An acid deposition value greater than 4 is regarded as 'very aggressive' and indicates that specialised advice should be sought.

5.10 For steel piles continuously and completely immersed in water or effluent, the sacrificial thickness on each relevant surface of the pile shall be 4mm unless the criteria in Table 5.2 indicate 'very aggressive' conditions. In this case, specialised advice should be sought.

5.11 For steel piles in the splash zone or in zones of wetting and drying, the sacrificial thickness on each relevant surface of the pile shall be 9mm. Further information on rates of corrosion in marine environments is given in Clause 4.4.4.4.3.4 of BS 8002 (1994).

5.12 The total sacrificial thickness required at any level shall be determined by adding together the thicknesses required for either side of the pile. As the exposure conditions on either side of the pile may be different at different levels, the total sacrificial thickness required may vary along the length of the pile.

5.13 The steel section chosen shall at any level in the structure provide the total sacrificial thickness required, as described, in addition to the thickness required for strength purposes. However, as the worst exposure conditions may not coincide with the worst positions of load effect, economies can be made in the overall steel section chosen provided the safety of the structure is not compromised.

5.14 When a portion of a steel pile wall is under surface water, for example in a river or a canal, consideration shall be given to the need to provide protection to the front of the piles to resist the abrasive action of any waterway traffic or floating debris. Protection may be in the form of casing or fendering and may extend above and below the water line as required. If a casing is to be constructed in concrete, the requirements of the Overseeing Organisation for concrete are contained in MCHW 1 Series 1700 and the casing shall be connected to the piling with shear connectors.

Property		**Characteristics**	**Points**
Soil type	1	Backfills meeting requirements for structural backfills (MCHW 1)	+2
	2	Backfills containing < 10% of particles (by weight) passing the 63µm sieve. Plasticity Index (PI) of fraction passing 425µm sieve < 2	+2
	3	Backfills containing < 75% passing the 63µm sieve and < 10% passing the 2µm sieve and PI of fraction passing 425µm sieve ≥ 2,≤ 6	0
	4	Any grading backfill with PI of fraction passing 425µm sieve ≥ 6,≤ 15	-1
	5	Any grading, backfill with PI of fraction passing 425µm sieve ≥ 15	-2
	6	Soils having an organic content of > 2% (other than cinder or coke)	-4
	7	Contaminated soils (Including those containing cinder, coke)	*
Ground water level		Above ground water level At ground water level, or In zones with periodic flow or seepage Permanently below ground water level	0 -4 0
Resistivity (ohm-cm)		≥ 10,000 < 10,000 but ≥ 3,000 < 3,000 but ≥ 1,000 < 1,000 but ≥ 100 < 100	+2 +1 -1 -3 -4
pH of soil and groundwater		≥ 5 < 5	0 -3
Potential water soluble sulfate (ppm)		≤ 200 > 200 but ≤ 500 > 500 but ≤ 1000 > 1000	0 -1 -2 -4
Chloride ion (ppm)		≤ 50 > 50 but ≤ 250 > 250 but ≤ 500 > 500	0 -1 -2 -4
Sulphide and hydrogen sulphide		No discolouration of lead acetate paper Slight darkening of lead acetate paper Moderate blackening of lead acetate paper Rapid blackening of lead acetate paper	0 -2 -3 -4

POINTS TOTAL	**CORROSIVITY CLASSIFICATION**
0 or more	Non-aggressive
- 1 to – 4	Aggressive
- 5 or less	Very aggressive

***Seek specialist advice**

TABLE 5.1: CORROSIVITY CLASSIFICATION OF DISTURBED, PEATY OR CONTAMINATED SOILS, IN CONTACT WITH STEEL PILES

Corrosivity Classification	Properties of water of effluent		
	pH	Chloride ion (ppm)	Soluble sulphate (ppm)
Non-aggressive	≥ 6	≤ 50	≤ 200
Aggressive	$5 \leq pH < 6$	>50 but ≤ 250	>200 but ≤ 500
Very aggressive	<5	>250	>500

ppm ≅ parts per million

TABLE 5.2: CORROSIVITY CLASSIFICATION OF WATER OR EFFLUENT IN CONTACT WITH STEEL PILES

Property	Test method
Soil type Particle size Plasticity index (PI) Organic content	 BS 1377 Part 2 BS 1377 Part 2 BS 1377 Part 3
Ground water level	By observation
Resistivity	Clause 637, MCHW 1
PH	BS 1377 Part 3
Potential water soluble sulfate	TRL Report 447, Test nos 1 and 3
Chloride ion	BS 1377 Part 3
Sulphide or hydrogen sulphide	Standard textbook of qualitative inorganic analysis eg Ref 3

TABLE 5.3: TEST METHODS FOR WATER, EFFLUENT AND SOILS IN CONTACT WITH STEEL PILES

6. SPECIAL REQUIREMENTS

Steel Sheet Pile Walls as Bridge Abutments

6.1 In the design of steel sheet piles for bridge abutments and their associated wing walls, particular attention shall be given to the following aspects:

i. Braking and other horizontal loads which are applicable from both directions across the wall.

ii. Fatigue of the welding present in high modulus sheet piles.

iii. Bearing capacity of sheet piles under dynamic axial loading.

iv. Effectiveness of any restraint anchors used.

Yandzio (1997) provides guidance on design of steel sheet pile bridge abutments. Biddle (1997) gives advice on vertical load capacity of steel sheet piles.

Partially Backfilled Embedded Retaining Walls

6.2 When the retained side of an embedded wall is partially backfilled, as in the cases of bridge abutments in embankments, the Overseeing Organisation's requirements for backfill material and its drainage requirements for the backfilled height of the wall are as described in MCHW 1. The corrosivity requirements for steel piles, as described, shall apply to the backfill material used.

6.3 Consideration shall be given to the effect of backfill compaction on lateral pressures acting on the embedded wall in accordance with Section 3.1.9 of BS 8002. In addition, for compacted cohesive backfill, account shall be taken of swelling pressures acting on the embedded wall. Clayton et al (1987 and 1991), O'Connor and Taylor (1993) and Brookes et al (1995) provide guidance on the effects of swelling on earth pressures in cohesive materials.

Ground Anchorages

6.4 The use of ground anchorages in permanent structures shall be considered with caution since the consequences of their failure will be serious. Inclined anchorages increase the vertical loading on a structure (see 1.8).

6.5 Ground Anchorage design shall be in accordance with the recommendations of BS 8081 as modified below:

i. The factor of safety at the grout/tendon or grout/encapsulation interface shall not be less than 3.

ii. Anchorages shall be double corrosion protected and shall all be restressable. Anchor heads shall be exposed or located in a recessed inspection chamber, to ensure easy accessibility.

iii. Proving tests shall be carried out on 3 anchorages of each type and capacity prior to installation of any anchorages in the permanent works.

iv. On-site suitability tests shall be carried out on the first three anchorages of each type and capacity.

v. Acceptability tests shall be carried out on all anchorages, to a proof load of 150% of working load.

vi. One anchorage shall be fitted with a load cell for every 10 anchorages installed or multiple thereof. This condition shall apply at each location.

vii. The structure shall be designed such that failure of a single anchorage will not precipitate any instability or collapse.

6.6 Assurance is required about the design life and load carrying capacity of ground anchorages especially with respect to creep and material durability in the particular ground conditions, since BS 8081 does not provide assurance in these respects.

Cosmetic Treatment of Exposed Steel Pile Walls

6.7 If for aesthetic reasons, the appearance of uncoated sections of steel piling exposed to the atmosphere is unacceptable, consideration shall be given to the use of suitable claddings, or facings or the application of a bridge painting system. The Overseeing Organisation's requirements are contained in MCHW 1 under the appropriate Series.

Hard-soft piling system

6.8 Secant bored pile walls may be constructed using alternate "hard" reinforced concrete piles and "soft" cement-bentonite piles. However concerns exist about the permeability of the cement-bentonite piles and their long-term durability. For this reason on recent schemes sprayed concrete facing have been employed over the exposed sections of soft piles. The performance of a hard-soft piled embedded retaining wall was monitored over a two year period and has been reported by Carder and Steele (2000).

Embedded walls as integral bridge abutments

6.9 Continuous bridge deck construction with integral abutments eliminates the need for movement joints and results in better durability of the structure. However with integral bridges, the abutments are subjected to repeated lateral displacements as the bridge superstructure expands and contracts with diurnal and seasonal temperature changes. Over a period of time these cyclic movements are likely to result in increased lateral earth pressures due to strain ratcheting in the retained ground and possible distress to the structure. These high pressures shall therefore be taken into account during the design; advice on their magnitudes is given in BA 42 Design of Integral Bridges (DMRB 1.3).

7. STRUCTURES WITH A SINGLE PROP NEAR EXCAVATION LEVEL

Introduction

7.1 This Chapter applies to embedded retaining structures where the stability is provided by a significant length of wall stem embedded in the ground and a single level of propping near excavation level.

This Standard may be used for the design and construction of these types of retaining structure, subject to the qualifications of the clauses of chapters 2 and 3 set out below.

Design Principles

7.2 Clause 2.1. It shall be noted that for structures propped near excavation level the type of connection between the wall and prop can affect the behaviour of the wall (Powrie and Li, 1991a; Potts, 1993) and the dominant modes of failure that need to be assessed for limit state design. Connections can be designed as sliding, pin-jointed or integral. All types of connection shall transmit axial thrust. Pin-joints also transmit shear, whilst integral connections provide full moment transfer.

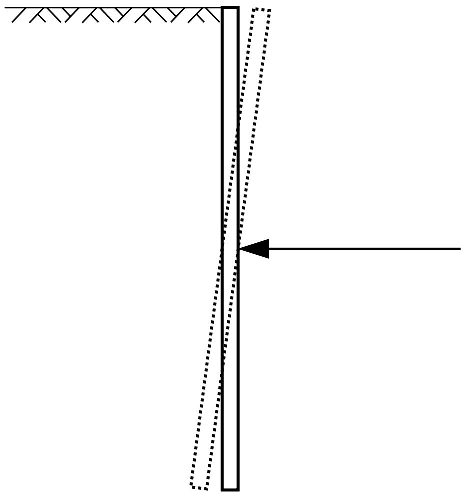
7.3 Clause 2.2. The potential modes of failure relevant to a retaining structure propped near excavation level are illustrated in Figure 7.1.

7.4 Clause 2.3. For retaining structures supported by a single prop located near excavation level failure can occur by rotation of the wall about the position of the prop. Two modes of wall movement are theoretically possible (Powrie and Li, 1991b; Carder and Symons, 1989; Symons 1992). These are as shown in Figure 7.1 (a) and (b). The most likely mode is forward rotation of the wall about the prop, but backward rotation also needs to be considered. Where the prop does not react against an opposing wall, other modes of wall movement need evaluation see Chapter 9.

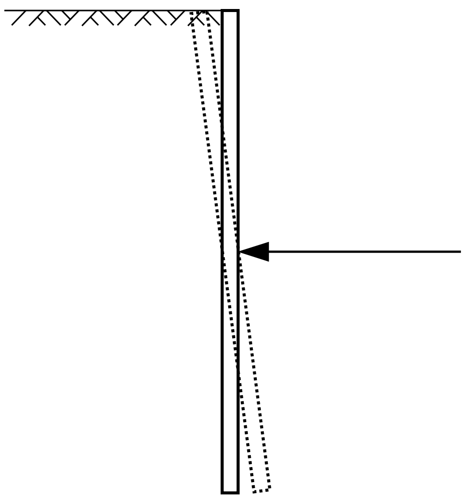
7.5 Clause 2.4. For deep excavations the prop near excavation level may need to be designed to either accommodate or withstand heave (Terzaghi, 1943; Bjerrum and Eide, 1956). Hydraulic failure/piping beneath the prop slab, particularly in cohesionless soils, shall also be considered. The available movement or rotation at the connection between the prop and the wall, and any other connection in the prop itself, shall be limited to prevent loss of serviceability or failure through heave. In addition, where small embedment depths are used, the possibility of bearing capacity failure beneath the toe of vertically loaded walls shall be examined. These factors may govern the embedment depth of the wall.

7.6 Clause 2.8. For structures propped near excavation level both forward and backward rotation of the wall about the permanent prop position are possible. Forward movement of the wall may also occur during or shortly after construction as the resisting force in the permanent prop slab develops (Symons and Carder, 1991). The mode of wall movement has a considerable effect on the distribution of earth pressure acting on the wall (Potts and Fourie, 1986). For this reason it is necessary to consider a wide range of admissible stress distributions and modes of wall behaviour in assessing the bending moments and prop forces in service for embedded walls propped near excavation level.

The design load in the permanent prop may be based on horizontal equilibrium of the earth pressures acting in front of and behind the wall over the total wall depth. The sequence of installing temporary props will, however, influence the magnitude of the load in the permanent prop at, or near, excavation level.



(a) Forward rotation about the prop



(b) Backward rotation about the prop

Figure 7.1: Overall Stability - Failure Conditions (Structures with a Single Prop Near Excavation Level)

i. Minimising wall movements during excavation by the use of temporary props at high level will increase the compressive load taken in the permanent prop after the former are removed. This is an efficient method of pre-loading the permanent prop to restrain wall movement (Richards and Powrie, 1994). The design of temporary propping at high level may be undertaken as for walls propped at, or near, the top.

ii. Using temporary props at low level will decrease the compressive load taken in the permanent prop after the former are removed. The additional movement of the wall which will occur using this construction sequence leads to some lateral stress relief in the retained ground.

Guidance on temporary prop loads is given by Twine and Roscoe (1999).

Design of structures

7.7 Clause 3.1 (i). For walls propped near excavation level a relatively small embedment depth may be sufficient for stability. In practice, as it may not be feasible to install the permanent prop until after the excavation has been made, the wall may require a larger depth of embedment in order to limit movements during excavation when support may be provided by temporary props or soil berms (see Chapter 11).

7.8 Clause 3.1 (ii). In determining factors of safety for forward and backward rotation about the permanent prop, the same principles of determining the ratio of restoring moments to overturning moments may be applied as described in CIRIA Report 104. The “CP2 Method of CIRIA Report 104” or the Strength Factor Method shall be used for these calculations as numerical problems arise in employing the other methods for walls propped near excavation level. In these calculations, account shall be taken of the mode of movement and active or passive soil pressure distributions selected accordingly.

7.9 Clause 3.1(iii). The determination of wall embedment depth for overall stability will tend to be dominated by the temporary works condition. For this, the depth of embedment can be determined adopting the moderately conservative parameter approach in conjunction with a low factor of safety as set out in Table 5 of CIRIA Report 104 assuming either:

i. effective stress parameters and a factor of safety in the range 1.1 to 1.5 according to the method used

ii. undrained strength and a high factor of safety in the range 1.5 to 2 according to the method used.

Method (i) is generally preferred for calculation of stability of the temporary works. However, if using method (ii), the adoption of undrained strength parameters shall be carefully chosen to account for effects such as softening. Section 6 of CIRIA Report 104 gives guidance.

7.10 Clauses 3.2 and 3.3. Measurements, at a number of construction sites, of the magnitude and extent of horizontal and vertical movements of the ground and wall for structures propped near excavation level and founded in stiff clay have been reported by TRL. These results are summarised by Carder (1995). The report provides upper bound values of measured movements, as a function of excavation depth, at the end of construction of relatively rigid walls.

7.11 Clause 3.5. If the worst credible approach is used to assess the adequacy of the structural design for temporary construction stages, the analysis may lead to overly conservative wall sections. In these circumstances the design for the temporary construction stages shall be checked using moderately conservative soil strength parameters.

7.12 Clause 3.10(iii). For walls with a single prop near excavation level and retaining heavily over-consolidated clays (where the initial K_o is greater than 1.5), the serviceability limit state of the structural elements can be evaluated using a K of 1 in the retained ground. This is because more lateral stress relief will occur with this type of structure than with walls permanently propped at the top. This approach has been validated by site measurements carried out by TRL and summarised by Carder and Darley (1999).

7.13 Clause 3.12. The design of a permanent structural prop near excavation level needs to take both axial forces and bending moments into account. Bending moment development is particularly likely in the longer term where over-consolidated clays underlie the prop. During construction, pore water pressures in the clay will reduce and then build-up again over a long time: associated softening will be accompanied by swelling and potential heave of the prop. Guidance on the magnitude of the long term heave may be obtained from Carder and Darley (1999) who report measurements in over-consolidated London Clay over an 11 year period following the construction of the Bell Common Tunnel. Carder et al (1997) present a case history study during the construction of a hinged structural prop on top of a sand drainage blanket overlying London Clay where both heave movements and vertical pressures were measured.

8. STRUCTURES WHICH ARE DOUBLY-PROPPED

Introduction

8.1 This Chapter applies to embedded retaining structures where the main stability is provided by a significant length of wall stem embedded in the ground and incorporating either props or anchors at two levels, eg. a cut-and-cover tunnel which may incorporate a structural slab at both roof and carriageway levels. It is anticipated that similar principles could be applied to multi-propped structures.

This Standard may be used for the design and construction of these types of retaining structure, subject to the qualifications of the clauses of chapters 2 and 3 set out below.

Design Principles

8.2 Clause 2.1. It shall be noted that for doubly-propped retaining structures the type of connection between the wall and props can affect the behaviour of the wall and the dominant modes of failure that need to be assessed for limit state design. Connections can be designed as sliding, pin-jointed or integral. All types of connection will transmit axial thrust. Pin-joints will also transmit shear, whilst integral connections will additionally provide full moment transfer.

8.3 Clauses 2.2 and 2.3. For retaining structures with integral props significant wall rotation or movement can only occur if failure of the structure is involved (Phillips et al, 1993). Where sliding or pin-jointed connections between wall and props are employed, any additional mechanisms of failure need to be identified which may be site specific. For structures retained by anchors, failure mechanisms associated with anchor pullout shall be considered.

8.4 Clause 2.4. For deep excavations in cohesive or cohesionless soils, foundation heave may need to be considered (Terzaghi, 1943; Bjerrum and Eide, 1956). In addition hydraulic failure/piping beneath the lower prop slab, particularly in cohesionless soils, shall also be considered. The available movement or rotation at the connection between the lower prop and the wall, and any other connection in the prop itself, shall be limited to prevent loss of serviceability or failure through heave.

8.5 Clause 2.8. The analysis and design of doubly-propped walls is complicated by the numerous possibilities for the sequence of propping (both temporary and permanent) during excavation of the soil in front of the wall. The sequence of propping will affect ground movements, wall bending moments and prop loads.

The design load in the permanent props can be based on horizontal equilibrium of the earth pressures acting in front of and behind the wall over the total wall depth. The sequence of installing temporary props will, however, influence the magnitude of the load in the permanent props. Minimising wall movements during excavation by the use of temporary props will, after their removal, increase the compressive load carried by the lower permanent prop. This is an efficient method of pre-loading the lower permanent prop to restrain wall movement (Richards and Powrie, 1995). If temporary props are not used, then the upper prop is likely to carry more load than the lower prop.

Various methods of analysis are available for the design of temporary and permanent props. The most widely used are:

i. pressure envelope semi-empirical method given by Terzaghi and Peck (1967) and also by BS 8002

ii. staged excavation - limit equilibrium method of analysis with hinges introduced at each prop position below the top prop so that each wall span is analysed as a simply supported beam (Williams and Waite, 1993; BS 8002)

iii. as (ii) above but assuming the wall acts as a continuous beam which is supported at some point below formation level (Tamaro and Gould, 1993)

iv. numerical analysis closely modelling the construction sequence (Richards and Powrie, 1995)

v. the empirical method of design developed from a comprehensive survey of field measurements of prop loads (Twine and Roscoe 1999)

The first three methods generally assume fully active conditions in the retained soil. In over-consolidated soils, the magnitude of the prop loads may therefore be underpredicted unless the higher lateral stresses are taken into account. This is particularly the case with doubly-propped walls where the structural system may be very stiff, and the opportunity for lateral stress relief may be minimal.

For the above reasons CIRIA Report 104 recommends that for doubly-propped walls the calculated prop forces are increased by 25% for the upper prop and 15% for the lower, to allow for the unquantified effects of pressure redistribution owing to wall yielding.

Design of structures

8.6 Clause 3.1(i). For a doubly-propped retaining structure the vertical loading could be a governing factor in the determination of the required wall embedment depth, particularly in cases where any roof slab is subjected to traffic or other live loads in addition to self weight.

One of the main purposes of the embedded portion of a doubly-propped retaining wall is to maintain stability during construction of the wall and base of the excavation, until the lower prop is in place and functioning. The depth of embedment of the wall may therefore be governed by temporary works considerations. In practice, as it is not usually feasible to install the lower permanent prop until after the excavation has been made, the wall may require a larger depth of embedment in order to limit movements during excavation when support is provided by the upper permanent prop and possibly temporary props. In the case where support during excavation relies on the upper permanent prop and the depth of embedment only, design for short-term conditions shall follow the recommendations given for walls propped at the top.

8.7 Clause 3.1(iii). In cases where excavation takes place below the upper permanent prop (with no temporary propping), the determination of wall embedment depth for overall stability will tend to be dominated by the temporary works condition. For this, the depth of embedment can be determined adopting the moderately conservative parameter approach in conjunction with a low factor of safety as set out in Table 5 of CIRIA Report 104) assuming either:

i. effective stress parameters and a factor of safety in the range 1.1 to 1.5 according to the method used

ii. undrained strength and a high factor of safety in the range 1.5 to 2 according to the method used.

Method (i) is generally preferred for calculation of stability of the temporary works. However, if using method (ii), the adoption of undrained strength parameters shall be chosen carefully to account for effects such as softening. Section 6 of CIRIA Report 104 gives guidance.

8.8 Clauses 3.2 and 3.3. Information on the measured magnitude and extent of horizontal and vertical movements at ground and wall surface for doubly-propped retaining structures constructed in stiff clay is given by Carder (1995). This report provides upper bound values of measured movements as a function of excavation depth at the end of construction. For doubly-propped walls with high stiffness support during excavation, eg. top-down construction, the maximum horizontal movement of the wall generally occurs at depths of between 0.7 to 0.9 times the excavation depth.

8.9 Clauses 3.6 and 3.10. If the worst credible approach is used to assess the adequacy of the structural design of the wall during the temporary construction stages, the analysis may lead to overly conservative wall sections. In these circumstances the design for the temporary construction stages shall be assessed using moderately conservative soil strength parameters.

8.10 Clauses 3.8 and 3.12. Load in the upper permanent prop is generally at a maximum during the temporary works stage when excavation has taken place prior to the installation of the lower prop. Load in the lower prop will depend on the depth of wall embedment and whether temporary props are used during excavation in front of the wall. The magnitude of the load, in the lower prop, may be best predicted by numerical analysis which takes the construction sequence into account and any possible load increase with time due to soil consolidation or swelling effects. Case history studies during the construction of two cut-and-cover tunnels by Brookes and Carder (1996) and Carder et al (1999) give further guidance on load development in a doubly-propped system.

9. STRUCTURES WITH A STABILISING BASE NEAR EXCAVATION LEVEL

Introduction

9.1 This Chapter applies to embedded retaining structures where stability is provided by a significant length of wall stem embedded in the ground and a stabilising base (ie. a horizontal stub base) at formation level that extends only a short distance from the wall.

This Standard may be used for the design and construction of these types of retaining structure, subject to the qualifications of the clauses of chapters 2 and 3 set out below.

Design principles

9.2 Clause 2.1. It shall be noted, however, that for retaining structures with a stabilising base the principle is that any rotation of the wall into the excavation would increase the bearing pressure below the stabilising base. This would impart a resisting moment to the wall, thus reducing the bending moment in the wall and arresting further wall movement. The principle only operates successfully when there is a full moment connection between the wall and the stabilising base.

9.3 Clause 2.2. The potential modes of failure relevant to a retaining structure with a stabilising base are illustrated in Figure 9.1.

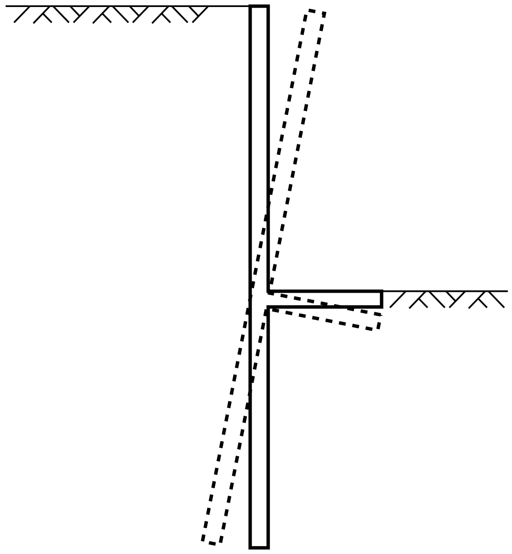
9.4 Clause 2.3. For retaining structures supported by a stabilising base at formation level, failure can theoretically occur by forward or backward rotation about the base, or forward rotation about either the wall toe or a point between the toe and formation level. The most likely mode is forward rotation about a point between the wall toe and formation level, although the other modes need to be considered. In all cases where forward rotation is the failure mechanism, the bearing capacity of the ground beneath the stabilising base is an important design consideration which shall be taken into account.

9.5 Clause 2.4. As for structures with a single prop at carriageway level (see Clause 7.5), the stabilising base shall be designed to accommodate or withstand heave and any possible hydraulic failure/piping.

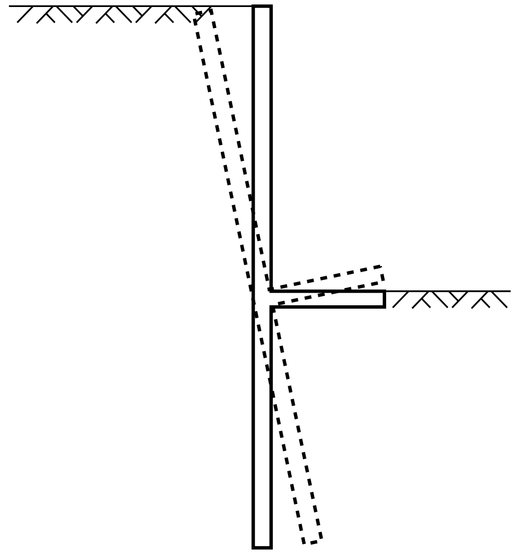
Design of structures

9.6 Clause 3.1 (i). For walls with a stabilising base a relatively small embedment depth may be sufficient for stability. In practice, as it may not be feasible to install the base until after the excavation has been made, the wall may require a larger depth of embedment in order to limit movements during construction when support may be provided by temporary props or soil berms (see Chapter 11).

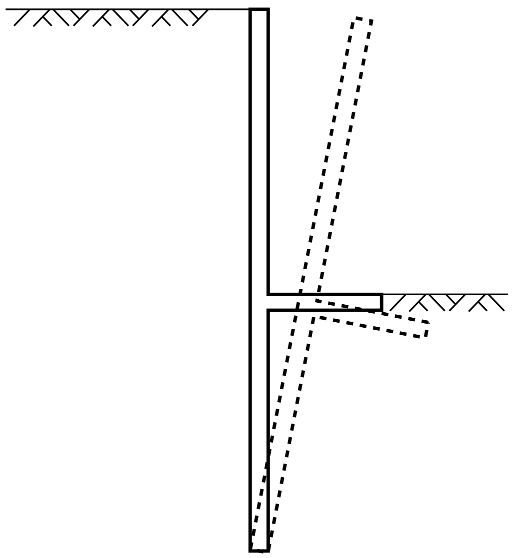
9.7 Clause 3.1 (ii). In determining factors of safety for forward and backward rotation about the base and forward rotation about either the wall toe or a point between the toe and formation level, the same principles of determining the ratio of restoring moments to overturning moments shall be applied as described in CIRIA Report 104. However in all cases where forward rotation occurs the restoring moment developed in the stabilising base by soil pressures beneath it shall be included. Account shall be taken of the mode of movement and active or passive soil pressure distributions selected accordingly. The most likely mode of failure is forward rotation about a point between the toe and formation level and in this case the analysis shall follow the “fixed earth support” principles for cantilever walls given in CIRIA Report 104. Only the “CP2 Method of CIRIA Report 104” or the Strength Factor Method shall be used for these calculations as numerical problems arise in employing the other methods for walls with a stabilising base.



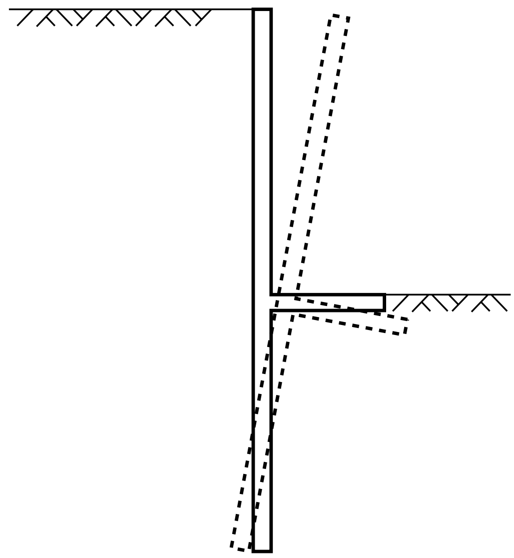
(a) Forward rotation about the stabilising base



(b) Backward rotation about the stabilising base



(c) Forward rotation about the wall toe



(d) Forward rotation about a point between the base and wall toe

Figure 9.1: Overall Stability - Failure Conditions (Structures with a Stabilising Base Near Excavation Level)

9.8 Clause 3.1(iii). The determination of wall embedment depth for overall stability will tend to be dominated by the temporary works condition. For this, the depth of embedment can be determined adopting the moderately conservative parameter approach in conjunction with a lower factor of safety as set out in Table 5 of CIRIA Report 104 assuming either:

i. effective stress parameters and a factor of safety in the range 1.1 to 1.5 according to the method used

ii. undrained strength and a high factor of safety in the range 1.5 to 2 according to the method used.

Method (i) is generally preferred for calculation of stability of the temporary works. However, if using method (ii), the adoption of undrained strength parameters shall be carefully chosen to account for effects such as softening. Section 6 of CIRIA Report 104 gives guidance.

9.9 Carder et al (1999) and Powrie et al (1999) have reported on the measured and predicted long term performance of an embedded wall with a stabilising base constructed in London (A406) in 1976. Hayward et al (2000) report on the behaviour during construction of a stabilised wall base in Coventry. Results from centrifuge modelling and a field monitoring study during construction are reported by Daly and Powrie (1999) and provide further design guidance. The use of numerical methods in the design of this type of structure is advantageous to confirm the likely movement mechanisms.

10. USE OF THE OBSERVATIONAL METHOD

Introduction

10.1 Use of the Observational Method in the construction of highway structures is likely to produce more economic structures by reducing undue conservatism in design whilst still assuring safe performance. With this method, feedback from monitoring is used to modify the original design and construction procedures according to a pre-determined plan which ensures that performance remains within acceptable limits. Peck (1969) formulated the original concepts of the Observational Method.

Design Principles

10.2 The main requirements of the approach are given in Eurocode 7 (1994) and can be summarised as:

i. Establish the limits of acceptable soil-structure behaviour

ii. Assess the range of possible behaviour and show that actual behaviour will probably be within the acceptable limits

iii. Devise and carry out a suitable plan of monitoring to reveal whether actual behaviour lies within the acceptable limits. Adjust to lower bounds if the design is over conservative

iv. Implement pre-planned contingency actions if the monitoring reveals behaviour outside acceptable limits.

10.3 Detailed guidance on the choice of soil design parameters when using the Observational Method is given by CIRIA Report 185 (Nicholson et al, 1998). These parameters may be progressively modified during the course of the construction.

10.4 One criterion for construction control of retaining walls is that of lateral movement. However it is envisaged that other criteria such as load measurements in structural props could also be employed. In the design of monitoring systems it is imperative to ensure that there is sufficient response time in which to implement a contingency action. This normally means that trends in behaviour need to be monitored and the results compared with well defined trigger points.

10.5 Generally the range of wall movement that can be accepted for a particular construction condition is sub-divided into green, amber and red zones following the “traffic light” approach of Glass and Powderham (1994). These zones are defined as follows:

i. ***green zone.*** Measured movement is less than the most probable prediction, so that it may be possible to achieve economies or speed up the construction.

ii. ***amber zone.*** Measured movement is between most probable and upper bound predictions, so that the frequency of monitoring needs to be increased to ensure that the upper limit of the amber zone is not exceeded.

iii. ***red zone.*** Measured movement is greater than the upper bound prediction, so that contingency actions need to be promptly initiated and monitoring continued at a high frequency.

Selection of trigger limits between construction condition zones based on lateral wall movement is described by Card and Carder (1996). Values are proposed for embedded structures with low, moderate and high stiffness support during excavation and founded in stiff clay. The limits are derived from TRL studies of wall movements on schemes where bored pile and diaphragm walls have been constructed and so are not applicable to more flexible sheet pile walls.

Benefits of implementation

10.6 For embedded walls it is likely that the Observational Method will be implemented in order to achieve economies or ensure safety, primarily for one of the following reasons:

i. To eliminate or reduce temporary propping during excavation in front of a wall.

ii. To enable soil berms to be used effectively as a method of support during excavation in front of the wall.

iii. To rationalise wall design by reducing penetration or thickness in the construction of subsequent sections.

iv. To optimise permanent prop design.

v. To ensure that damage does not occur to adjacent structures and utilities and to minimise disruption to road users.

11. USE OF SOIL BERMS FOR TEMPORARY SUPPORT

Introduction

11.1 Traditionally support for embedded retaining walls constructed with a single prop near excavation level has been provided during bulk excavation by temporary steel props. These temporary props are often installed horizontally so as to span between the opposing walls of an underpass or tunnel. However, there are many construction situations where this technique is either excessively costly or is impractical, eg. if there is no opposing wall to react against or if the carriageway is very wide. In these situations the use of soil berms as a method of temporary support may be appropriate.

11.2 Soil berms have been shown to be an effective method of temporary support for retaining walls. Carder and Bennett (1996) have given preliminary guidance based on finite element modelling of the effect of berm height on performance, and Easton and Darley (1999) have reviewed and back-analysed four road construction schemes where soil berms were successfully used for wall support.

11.3 Further information on the design of soil berm geometries is given by Potts et al (1993) and Easton et al (1999). Case history studies are also presented by Powrie et al (1993) and Gourvenec et al (1996).

Design Principles

11.4 The construction sequence when using soil berms for temporary support needs careful consideration. Excavation of the soil berm normally takes place in bays with the permanent prop being constructed as soon as each bay is excavated. The excavation sequence is scheduled so that the adjacent bays to either side of the excavated bay are always supported, either by an unexcavated soil berm or a completed section of permanent prop slab.

11.5 In cases of uncertainty the use of soil berms shall be combined with the monitoring of lateral wall movements in conjunction with the Observational Method (see Chapter 10). Uncertainties may exist where softening of the ground is likely because of high water levels or where permeable or soft soil layers exist. Implementation of the Observational Method is also particularly advisable where only a small berm is being used and its effectiveness needs to be carefully monitored.

Design of soil berms

11.6 Tentative design methods for assessing the performance of soil berms are given in CIRIA Report 104. Two of these methods involve fairly complex calculations, as follows:

i. Using a slope-stability method taking into account the weight of the berm and the forces from the wall.

ii. Using a Coulomb wedge type of analysis for a number of trial failure surfaces emanating from the toe of the wall.

Alternatively, two empirical methods, which have been used successfully, are:

i. Converting the weight of the berm to an effective surcharge acting at final excavation level on the potential passive failure zone (CIRIA Report 104; Fleming et al, 1992).

ii. Considering the berm as an increase in the effective ground level on the passive side of the wall. The design height of the berm is limited to one third of the berm width and the increase in effective ground level is then taken as one half of this design height (Fleming et al, 1992).

11.7 The above empirical design methods could be over-conservative in so far as the lateral resistance provided by the soil berm is either ignored or treated in an empirical manner.

11.8 Easton et al (1999) used results from finite element work together with factor of safety calculations to create “user friendly” design aids in the form of charts, aimed at both rationalising and producing economy in the design of soil berms for the temporary support of retaining walls during construction.

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BD 13 Design of Steel Bridges. Use of BS 5400: Part 3 (DMRB 1.3)

BD 16 Design of Composite Bridges. Use of BS 5400: Part 5: (DMRB 1.3)

BD 24 Design of Concrete Bridges. Use of BS 5400: Part 4 (DMRB 1.3.1)

BD 37 Loads for Highway Bridges. Use of BS 5400: Part 2. (DMRB 1.3)

BA 42 The Design of Integral Bridges (DMRB 1.3.12)

BD 57 Design for durability (DMRB 1.3.7)

BD 74 Foundations, use of BS 8004: 1986 (DMRB 2.1.8)

12.2 Manual of Contract Documents for Highway Works (The Stationery Office)

Volume 1: Specification for Highway Works. (MCHW 1)

Volume 2: Notes for Guidance on the Specification for Highway Works. (MCHW 2)

12.3 British Standards

BS 1377: 1990 Methods of Test for Soils for Civil Engineering Purposes.

Part 2: Classification Tests
Part 3: Chemical and Electro-chemical Tests.

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Part 4: 1989: Code of Practice for Design of Concrete Bridges

Part 5: 1979: Code of Practice for Design of Composite Bridges

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14. ENQUIRIES

All technical enquiries or comments on this Standard should be sent in writing as appropriate to:

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Annex A: The Reinforcement of Cast-in-Place Embedded Retaining Walls

CONTENTS

A1 Introduction and Use of Annex A

A1.1 This Annex states the requirements for the reinforcement of cast-in-place embedded retaining walls. It has been prepared as a stand alone document.

This Annex shall be used for the structural design of cast-in-place embedded retaining walls comprising the following forms of construction:

1) Contiguous bored pile walls.

2) Hard/hard secant pile walls.

3) Hard/soft secant pile walls (hard part only).

4) Diaphragm walls.

The design requirements for secant walls incorporating soft/soft construction or incorporating plugged structural steelwork sections are outside the scope of this Annex.

Design requirements in this Annex are suitable for:

- free standing retaining walls
- retaining walls incorporating propping or anchor systems
- retaining walls in cut and cover construction

A1.2 Reinforcement for cast-in-place retaining walls is based on the requirements of BS 5400: Part 4: 1990, as implemented by BD 24 (DMRB 1.3.1), and as further modified by the recommendations of TRL report 144.

A1.3 Units and symbols in this Annex are as used in BS 5400: Part 4: 1990 as implemented by BD 24 (DMRB 1.3.1).

A1.4 Typical embedded retaining wall types within the scope of this Annex are shown in Figures A1.1 and A1.2.

A1.5 A flow diagram, Figure A1.3, demonstrates the use of this Annex.

Figure A1.1 - Embedded Retaining Wall Types

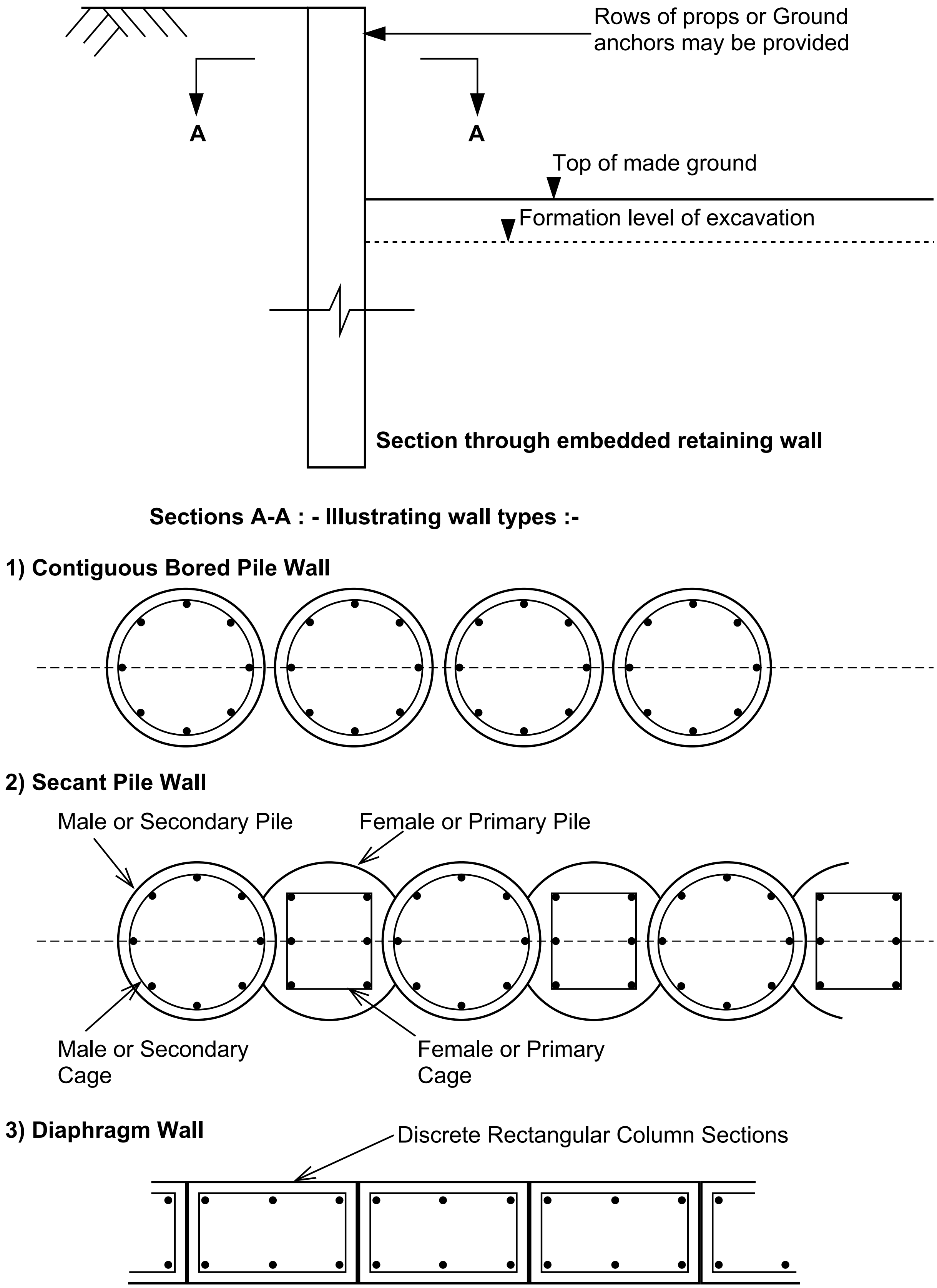

Figure A1.2 - Section Showing Cut and Cover Construction

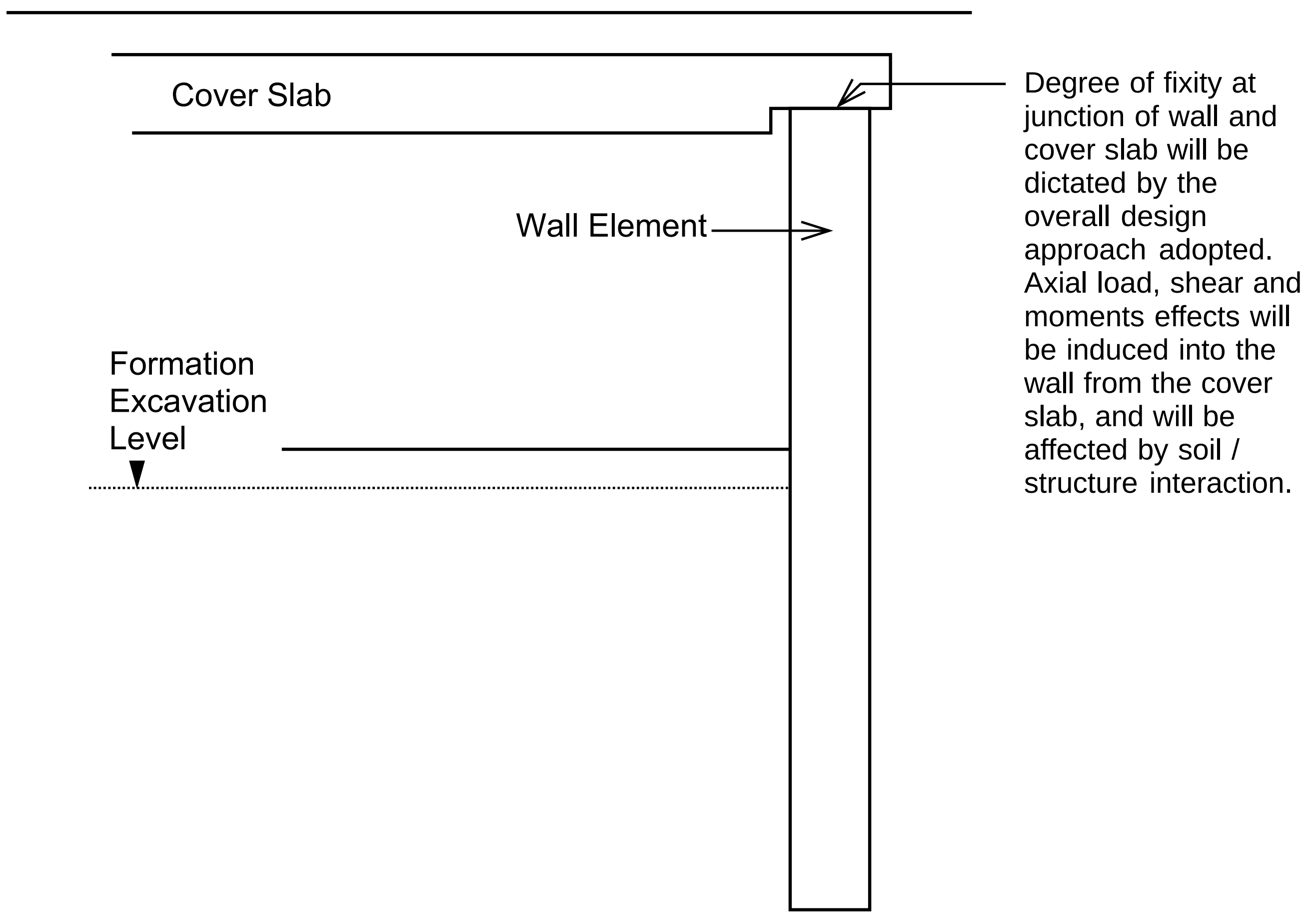

Figure A1.3 - Use of Annex A - Flow Diagram

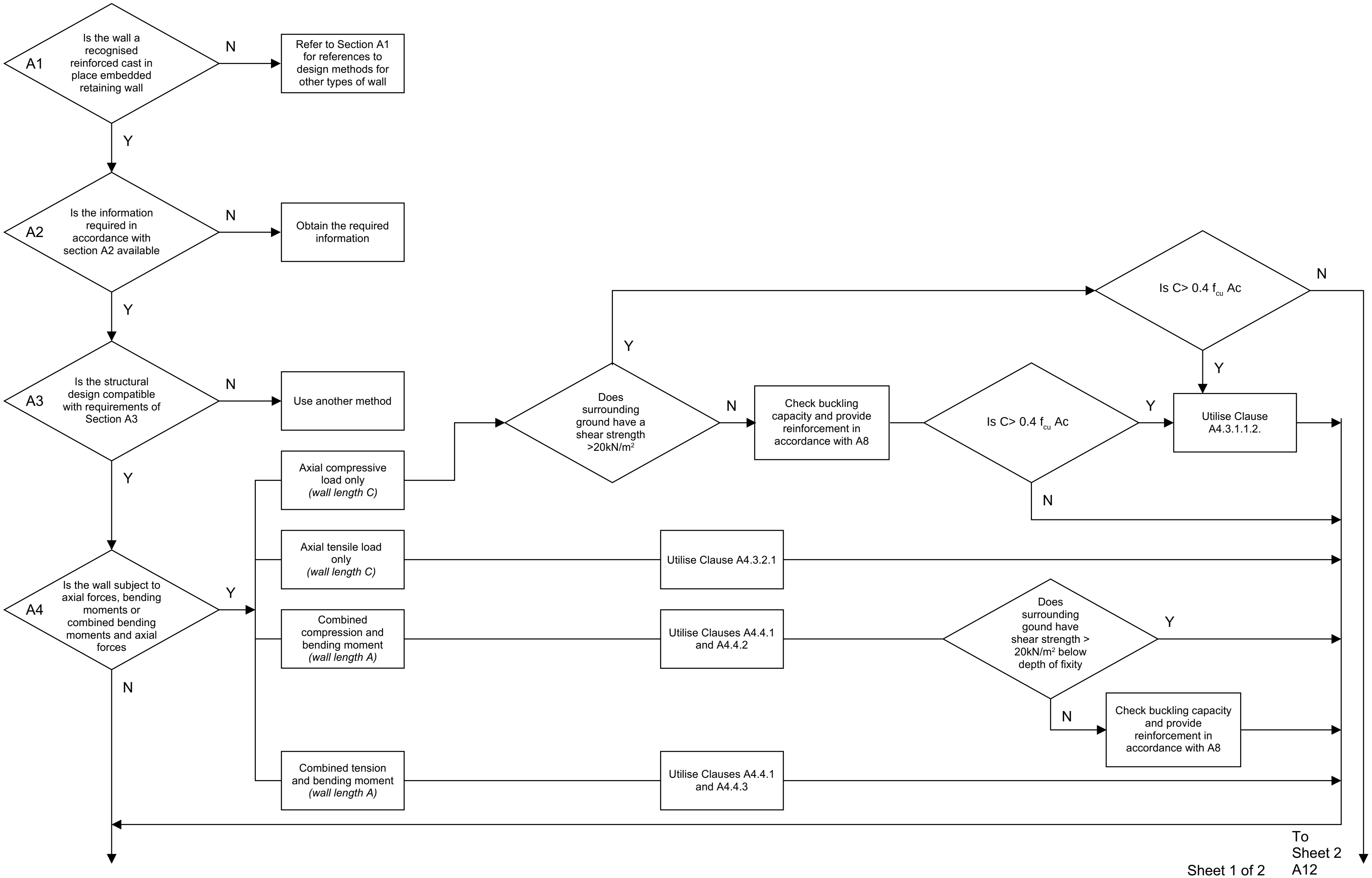

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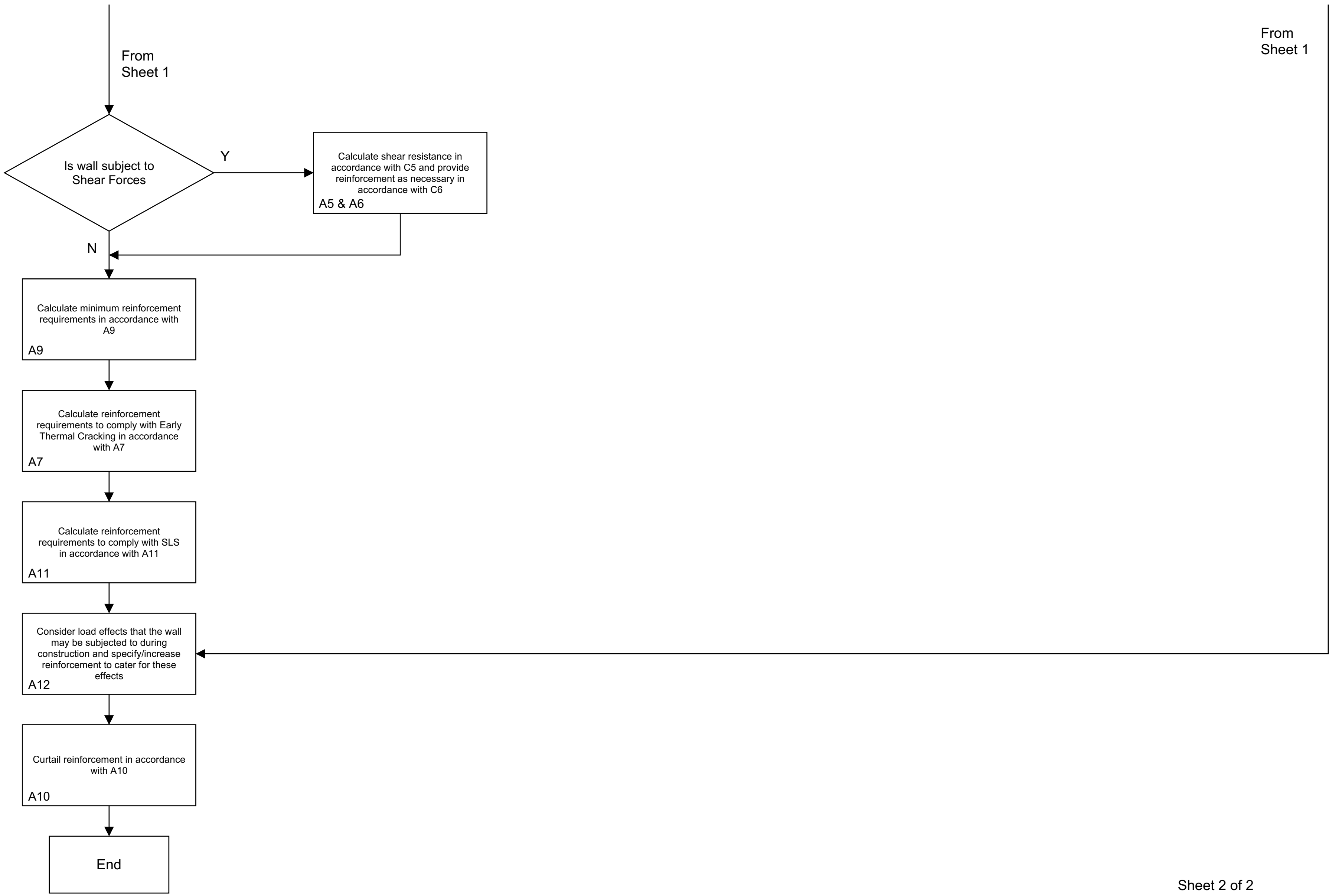
Figure A1.3 - Use of Annex A - Flow Diagram (continued)



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A2 Load Effects and Design Considerations for Structural Design

A2.1 At this stage in the design process the size and overall depth of the elements comprising the wall will have been determined using the design procedures required and referred to in this Standard.

A2.2 It should be appreciated (refer to Figure A2.1) that the depth of wall necessary to resist applied bending moments and horizontal load effects (designated wall depth A), to satisfy ultimate limit state and serviceability limit state requirements for soil-structure interaction, may be less than the length of wall necessary to resist axial load effects (designated wall depth B) in instances where the wall is subject to vertical loading. The additional depth of wall required to resist axial loading is designated wall depth C.

A2.3 There will be no structural design requirement for the wall below wall depth A to carry bending moment or shear load effects. Instead this section of wall (wall depth C) must be structurally capable of carrying the design axial load effects only. In such cases, for compression loads, no reinforcement will be required within wall depth C, providing the plain concrete can carry the design axial load effect in accordance with the requirements of this Annex, and providing that the buckling resistance of the wall as described in Section A9, need not be considered.

A2.4 Before using Annex A, design information should be available upon the minimum depth of wall required to withstand externally applied vertical and horizontal load and moment effects (wall depth A) and the depth of wall required purely to resist vertical (axial) load effects (wall depth B).

Moments and shears will be applied to the wall due to earth pressures and surcharge loading effects. In addition, moments will occur as a result of axial load effects applied eccentrically to the wall, either by design or resulting from construction tolerances, or due to the wall deflecting under load. In instances where uneven down drag effects occur, moments, shears and axial loads will be induced.

In cut and cover construction where reinforcement continuity is provided between the wall and cover slab loading effects on the cover slab will induce moments, shears and axial loads into the wall.

A2.5 The design of the structural elements of the wall requires a limit state approach based on factored load effects. It is important that the structural engineer responsible for design of the structural elements of the wall in accordance with this Annex communicates with the geotechnical engineer to ensure there is a mutual understanding of each party's requirements. This is to include ensuring that the geotechnical engineer is made aware of any performance requirements such as limits on vertical and horizontal deflections that need to be complied with. In addition, for cut and cover construction with continuity between cover slab and wall, dialogue between geotechnical and structural engineers will be required to ensure a mutual understanding of load effects induced in the walls and cover slab as a result of dead and live loadings on these elements.

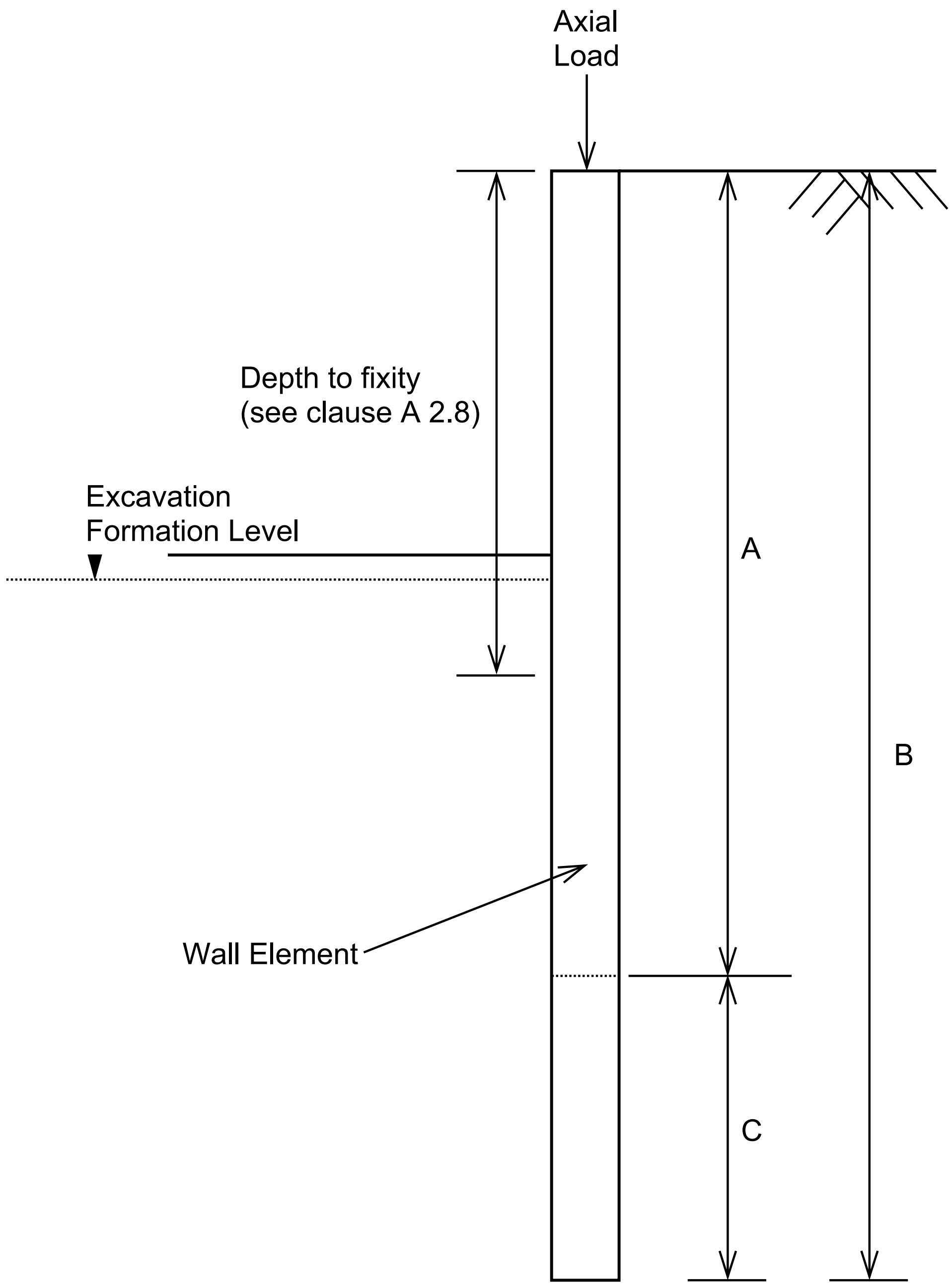
A2.6 Load effects acting on structural elements shall be provided for both the serviceability limit state (SLS) and the ultimate limit state (ULS) conditions using the requirements for partial load factor (γ_{fL}) and partial safety factor (γ_{f3}) in accordance with Section 3 of this Standard.

A2.7 Envelope diagrams for axial force, shear force bending moments and deflections at SLS and ULS shall be made available including details of how the various effects interact to form the basis of the structural design. The load effects shall include temporary construction conditions, as well as the permanent condition.

A2.8 The load effect diagrams shall be suitable to identify the depths to fixity, this being taken as the depth to the first position below the excavation formation level where the shear force in the wall is zero, for the load case under consideration.

A2.9 When applying the structural design rules in this Annex to co-existent load effects, care must be taken to apply the correct (reduced) partial load factors to load effects which have a relieving effect.

Figure A2.1: Diagram Showing Wall Depths as Defined in Section A2



Key to wall depths

A : Depth fo wall element subject to axial forces, shear forces and bending moments.
B : Overall depth of wall element.
C : Depth of wall element subject to axial forces only.

Note; When axial load is zero, wall depth C is not required.

A3 Wall Strength and Stiffness

A3.1 General

A3.1.1 Wall strength and stiffness is related to the size of the wall elements incorporated, and the strength of concrete used to construct the wall elements. An increase in concrete strength will result in an increase in concrete stiffness, and therefore the strength of concrete used in design has an effect on both wall capacity and the forces attracted to the wall.

A3.1.2 **For laterally loaded walls** an increase in concrete strength results in an increase in the relative differences between soil and wall stiffnesses. This may have the effect of increasing the magnitude of shear forces and bending moments acting on a wall element.

A3.1.3 **For axially loaded walls** concrete strength is a governing factor for wall load carrying capacity and an increase in concrete strength will directly reduce any requirement for compression reinforcement.

A3.1.4 Increased concrete strength, although allowing greater axial and lateral loads to be carried, has the disadvantage of producing a greater tendency for thermal cracking to occur. The specification of concrete mixes for use in walls therefore needs to be carefully selected to provide the optimum design solution.

A3.2 Concrete

Concrete shall be in accordance with MCHW 1 Series 1600 and have a minimum characteristic strength at 28 days of 30N/mm².

A3.3 Reinforcement

Reinforcement shall be in accordance with MCHW 1 Series 1700.

A3.4 Cover to Reinforcement

Cover to reinforcement shall be in accordance with BS 5400: Part 4: 1990 Table 13 as implemented by BD 24 (DMRB 1.3.1) plus the additional requirements of BS 8004 Clause 2.4.5. The requirements of BD 57 (DMRB 1.3.7) shall be deemed to be satisfied.

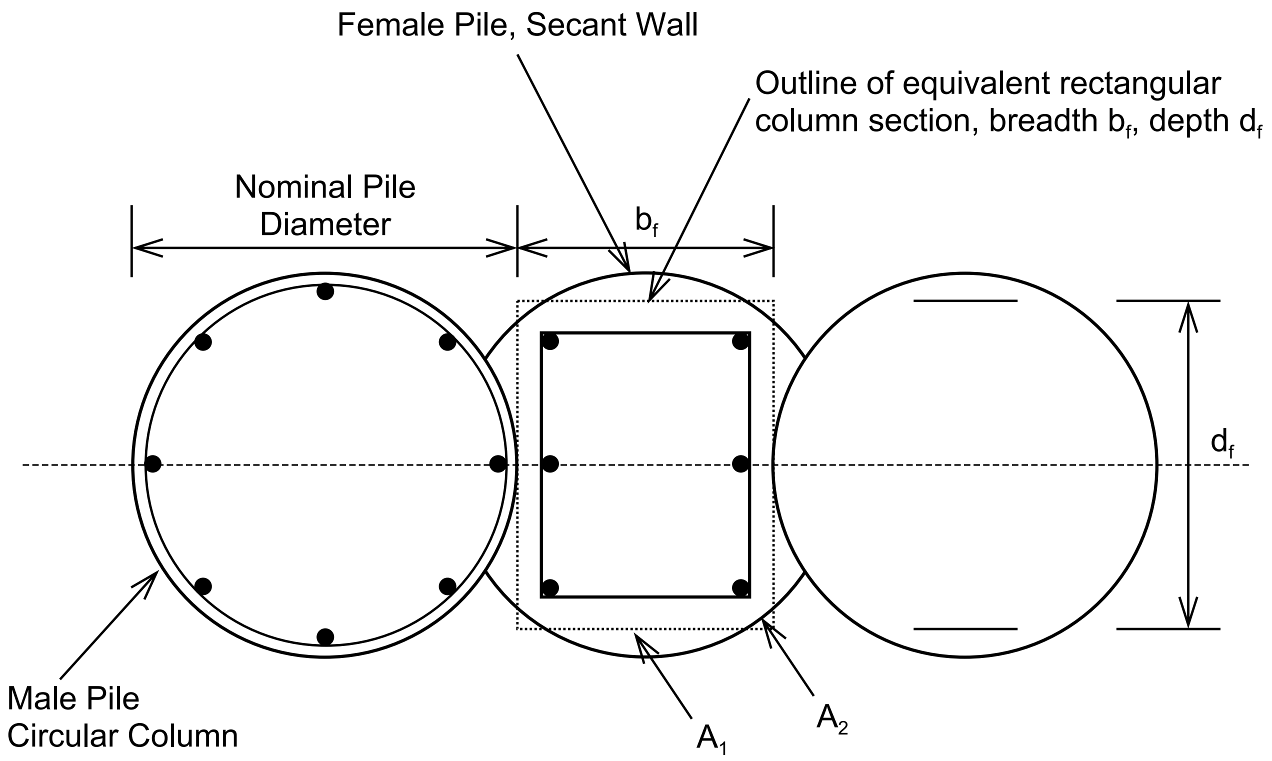
A3.5 Wall Element Sections

A3.5.1 Piles in a contiguous bored pile wall, and male piles in a secant pile wall shall be treated as circular columns.

A3.5.2 For female piles in secant walls it is recommended that an equivalent rectangle be utilised for determining both stiffness and strength characteristics. See Figure A3.1.

A3.5.3 In diaphragm walls the discreet rectangular wall sections shall be treated as rectangular column elements. See Figure A3.2.

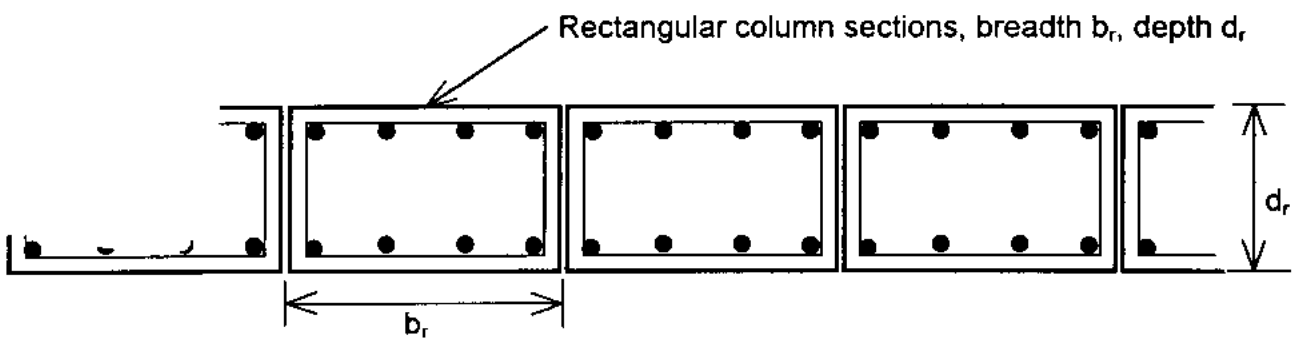
Figure A3.1 - Secant Wall, Effective Size of Female Pile



Equivalent gross concrete area of female pile $A_c = b_f \times d_f$.

Depth of equivalent rectangular section (symetrical about centre line of wall) of female pile d_f may be determined such that area $A_1 = 2 \times$ area A_2. (Note: A_1 is the area between the chord and radius and A_2 is the area outside the radius.)

Figure A3.2 - Effective Size of Diaphragm Wall Element



Gross concrete area of column element A_c= breadth b_r x depth d_r

A4 Walls Subject to Axial Forces and Bending Moments (ULS)

A4.1 Whatever method of analysis is adopted for soil-structure interaction, the requirements of Section A2 shall be adopted.

A4.2 In addition to the Ultimate Limit State requirements of Section A4 the adequacy of wall elements shall also be considered at Serviceability Limit State, in accordance with Section A11.

A4.3 Walls Subject to Axial Forces Only (Wall Depth C – refer to figure A2.1)

A4.3.1 **Compressive Axial Forces**

A4.3.1.1 The resistance of lengths of wall elements subject to compressive axial forces, where the ground surrounding the wall has a shear strength $\geq$ 20kN/m², shall be calculated using the following formulae:

A4.3.1.1.1 For an applied axial force $C < 0.4\, f_{cu}\, A_c$ no reinforcement is required, subject to the requirements of A12.

A4.3.1.1.2 For an applied axial force $C \geq 0.4\, f_{cu}\, A_c$ provide reinforcement in accordance with Clauses 5.5.3.2 and 5.5.3.4 of BS 5400: Part 4: 1990 implemented by BD 24 (DMRB 1.3.1).

A4.3.1.1.3 Reinforcement provided for an applied axial force $C \geq 0.4 f_{cu}\, A_c$ shall be at least equal to the minimum reinforcement requirements stated in Section A9. The requirements of Section A12 shall also be applied.

In the above formulae:

f_{cu} is the characteristic compressive strength of the concrete.

A_c is the gross cross sectional area of the wall element at the section considered (see figures A3.1 and A3.2).

A4.3.1.2 Where the ground surrounding the wall has a shear strength < 20kN/m² the effects of buckling of the wall shall be considered in accordance with the requirements of Section A8.

The value of P_{CR} calculated in accordance with Section A8 shall not exceed the applied compressive axial force.

Minimum reinforcement in accordance with the requirements of A9 shall be specified.

A4.3.2 **Tension**

A4.3.2.1 The resistance of wall elements subject to tensile axial forces shall be calculated using the following formula:

$$A_{st} = \frac{T}{0.87 f_y}$$

where A_{st} is the area of longitudinal reinforcement (which shall not be less than the minimum area of reinforcement stated in Section A9).

T is the applied tensile axial load.

f_y is the characteristic tensile strength of the reinforcement.

A4.3.2.2 The requirements of Sections A7, A8, A9, and A12 shall be applied.

A4.4 Wall Elements Subject to Axial Forces Combined with Bending Moments (Wall Depth A)

A4.4.1 **Additional Effects and Considerations**

In addition to the load effects caused by soil structure interaction additional moment effects due to axial load shall be considered in the upper section of wall elements exposed by the excavation, down to the depth of fixity where the element is fully embedded, and beyond to the depth where the moment effects have dissipated. These effects comprise:

i) wall element deflection

ii) eccentrically applied axial load

In addition to the above the effects, buckling for slender wall elements shall also be considered. The depth of fixity is defined in A4.4.1.2 a).

A4.4.1.1 **Wall Deflection**

Where other effects cause wall deflections, axial loading may induce additional bending and shear load effects in the wall. These shall be catered for in the structural design.

A4.4.1.2 **Wall Buckling and Eccentric Axial Loading**

When axial compressive load effects act upon the column elements comprising the wall additional bending moment effects shall be considered as required in BS 5400: Part 4: 1990 as implemented by BD 24 (DMRB 1.3.1) and as modified below:

a) Column effective height and slenderness

The effective height l_e and slenderness l_e/h, of the wall elements shall be determined in accordance with Clause 5.5.1.2 of BS 5400: Part 4: 1990 as implemented by BD 24 (DMRB 1.3.1).

When using this clause, the appropriate idealised column and buckling mode or modes shall be selected from Table 11 of BS 5400: Part 4: 1990 as implemented by BD 24 (DMRB 1.3.1). The use of multiple rows of props or ground anchors will influence the idealised buckling modes for various lengths of column between points of restraint.

The depth of fixity in A4.4.1 is defined as follows:

The depth of fixity may be taken as being the depth to the first position below the excavation level where the shear force in the wall is zero, for the load case under consideration.

In the case of a propped or anchored wall the depth to fixity may coincide with the end of wall depth A, at which point moments in the wall are zero, and no rotational restraint is provided.

b) Short Columns

For short columns, with a ratio $l_e/h < 12$, of the eccentricity given in Clause 5.5.3 in BS 5400: Part 4: 1990 as implemented in BD 24 (DMRB 1.3.1) for determining additional moment effects shall be modified to suit appropriate construction tolerances, and should not normally be less than 75mm.

c) Slender Columns

For slender columns, with a ratio of $l_e/h \geq 12$, the requirements of Section 5.5.5 in BS 5400: Part 4: 1990 as implemented in BD 24 (DMRB 1.3.1) for additional moment effects shall be utilised with an eccentricity as described in A4.4.1.2 b) above.

A4.4.2 **Compressive Axial Forces and Bending Moments**

A4.4.2.1 The design shall be in accordance with the requirements of Clauses 5.5.3.2 and 5.5.3.4, as appropriate, of BS 5400: Part 4: 1990 as implemented by BD 24 (DMRB 1.3.1). The further provisions of Sections A5, A6, A11 and A12 shall also be applied. Minimum reinforcement requirements shall be in accordance with Section A9.

A4.4.2.2 When the shear strength of the surrounding ground below the depth of fixity is ≤ 20kN/m² the effects of buckling need to be considered in accordance with the requirements of Section A8.

A4.4.2.3 Where an axial compressive force acting on a wall element provides a beneficial effect upon reinforcement demand the value of this force must be carefully evaluated, both in terms of the likely variation of its magnitude along the length of the wall element and the partial load factor employed, to ensure an appropriate value is utilised.

A4.4.2.4 Unless accurate positioning of the reinforcement within the wall can be ensured, the analysis should take into account the most onerous orientation to ensure that the required value of bending resistance is achieved.

A4.4.3 Tensile Axial Forces and Bending Moments

A4.4.3.1 Bending moments acting in conjunction with tensile axial forces will, depending on their respective magnitudes result in a wall element, at any particular cross section remaining wholly in tension or partly in tension and part compression.

A4.4.3.2 Design shall be carried out in accordance with Clauses 5.5.3.2 of BS 5400: Part 4: 1990 as implemented by BD 24 (DMRB 1.3.1). The further provisions of Sections A5, A6, A11 and A12 shall also be applied. Minimum reinforcement requirements shall be in accordance with Section A9.

A4.4.3.3 Unless accurate positioning of the reinforcement within the wall can be ensured, the analysis should take into account the most onerous orientation to ensure that the required value of bending resistance is achieved.

A5 Wall Elements Subject to Shear Forces (ULS), (Wall Depth A)

A5.1 Lateral forces calculated in accordance with Section A2 set up shear forces within the wall.

The shear stress, v, at any cross section shall be calculated from

$$v = \frac{V}{bd}$$

where V is the shear force due to ultimate loads

b is the wall pile diameter, or in the case of a rectangular section, its breadth.

d is the distance from the extreme fibre with maximum compression to the centroid of the reinforcement in the half of the wall pile opposite the extreme compression fibre in a circular pile, or the effective depth in the case of a rectangular section.

To prevent crushing of concrete v shall not exceed $0.75\sqrt{f_{cu}}$ or 4.75N/mm^2 whichever is the lesser, whatever shear reinforcement is required.

A5.2 For design of shear reinforcement refer to Section A6.

A6 Shear Reinforcement

A6.1 Shear link reinforcement and additional longitudinal reinforcement shall be provided in accordance with the following criteria:

$v \le \frac{\xi_s v_c}{2}$ *	No shear reinforcement required (refer to Section A9)
$\frac{\xi_s v_c}{2} \le v \le \xi_s v_c$ *	Minimum reinforcement (refer to Section A9).
$v > \xi_s v_c$ *	The greater of

$$A_{sv} = \frac{S_v\, b(v - \xi_s v_c)}{\frac{f_{yv}}{\gamma_{ms}}}$$

or minimum reinforcement requirements in accordance with Section A9.

where

A_{sv} is the cross sectional area of all legs of the links at a particular cross section

v is the applied shear stress

v_c is the ultimate shear stress in concrete calculated in accordance with Table 8 of BS 5400: Part 4: 1990 where the area of longitudinal reinforcement A_s to be used to calculate v_c shall be taken as the area of reinforcement which is in the half of the column opposite the extreme compression fibre. The effective depth shall be taken as the distance from the extreme fibre with maximum compression to the centroid of this reinforcement. The web width shall be taken as the column diameter.

ξ_s is the depth factor (refer to Table 9 of BS 5400: Part 4: 1990)

s_v is the spacing of the links along the member

b is the wall pile diameter, or in the case of a rectangular section, its breadth.

f_{yv} is the characteristic strength of link reinforcement but not greater than 460N/mm²

γ_{ms} is the partial safety factor for steel reinforcement

* $\xi_s v_c$ shall be multiplied by $1+\frac{0.05N}{A_c}$ in these instances.

where N is the ultimate axial load (in newtons).

A_c is the area of the entire concrete section (in mm²).

A6.2 The spacing of the legs of links in the direction of the span and at right-angles to it shall not exceed 0.75d.

A6.3 At any cross section additional longitudinal reinforcement is required in the tensile zone (in excess of that required to resist bending) such that

$$A_{sa} \ge \frac{V}{2(0.87 f_y)}$$

where

A_{sa} is the area of effectively anchored additional longitudinal tensile reinforcement

f_y is the characteristic strength of the longitudinal reinforcement

V is the shear force due to ultimate loads

Note: For symmetrical arrangements of reinforcement this will result in a total required area of reinforcement equal to 2 x A_{sa} spaced evenly round the perimeter of the wall pile.

A6.4 In circular columns the ultimate shear stress in concrete, v_c, calculated in accordance with Table 8 of BS 5400: Part 4: 1990 shall be determined taking the longitudinal reinforcement A_s used to calculate v_c as the area of reinforcement which is in the half of the column opposite the extreme compression fibre. The effective depth shall be taken as the distance from the extreme fibre with maximum compression to the centroid of this reinforcement. The web width shall be taken as the column diameter.

A6.5 The minimum reinforcement requirements in Section A7 and A9 shall be provided.

A7 Early Thermal Cracking

A7.1 General

A7.1.1 Fully embedded walls restrained along their outer surfaces by the surrounding ground can suffer from the effects of both externally and internally restrained early thermal cracking. Due to the insulating properties of the surrounding ground externally restrained cracking is usually the dominant factor. This type of cracking can penetrate through the entire concrete section and is generally horizontal. For the upper section of wall elements subject to the application of corrosion and durability requirements (SLS) in Section A11.1 consideration shall be given to the appropriate application of BD 28 (DMRB 1.3) “Early Thermal Cracking.”

A7.1.2 For all parts of wall element depth A checks shall be carried out to ensure that the longitudinal steel does not yield and that the cracked wall section shall be capable of resisting applied shear forces.

A7.1.3 Sections 7.2 and 7.3 need only be implemented if reinforcement is required to comply with other sections of this Annex.

A7.2 Control of Thermal Cracking

To ensure the reinforcement in the wall will not yield before the tensile strength of the immature concrete is exceeded the following equation shall be satisfied:

$$A_s = \frac{f_{ct}*}{f_y} \quad A_c$$

where

A_s is the total cross sectional area of effectively anchored longitudinal reinforcement distributed evenly around the perimeter of the wall element.

A_c is the effective concrete area of the wall element which is equal to the gross cross sectional area for sections up to 500mm thick. For sections with a thickness greater than 500mm it shall be taken as that area of concrete which lies within 250mm of the surface.

f_y is the characteristic tensile strength of the reinforcement

$f_{ct}*$ is the tensile strength of immature concrete which may be taken as $0.37 \sqrt{(f_{cu})}$

f_{cu} is the characteristic cube strength of concrete

A7.3 Shear Resistance of Sections at Ultimate Limit State Subject to Early Thermal Cracking

A7.3.1 Wall elements shall be designed to withstand applied shear forces in conjunction with the effects of early thermal cracking. The shear resistance of walls subject to early thermal cracking should be calculated using the following method:

Longitudinal reinforcement shall be provided in accordance with the following equation:

$$A_s \geq \frac{V}{0.73f_y} + \frac{1.15T}{f_y}$$

where

V is the shear force due to ultimate loads

T is the tensile axial force due to ultimate loads

A_s is the total cross sectional area of effectively anchored longitudinal reinforcement crossing a cracked section

f_y is the characteristic strength of effectively anchored reinforcement crossing a cracked section

Notes

i) The first term of this equation is derived from equation reference 61 in BS 8110: Part 1, "Structural Use of Concrete" with tan α taken as 1.4. The constant 0.73 is the product of 0.6 (constant stated in equation reference 61), 1.4 (value of tan α) and 0.87 ($1/\gamma_m$)

A7.4 **Requirements for Longitudinal Reinforcement**

A7.4.1 The area of longitudinal reinforcement calculated in accordance with Sections A7.2 and A7.3 shall be compared to the summed longitudinal reinforcement requirements, calculated in accordance with Sections A4 and A6. The greatest area calculated in accordance with Section A7.2, A7.3 and the summed requirements of Section A4 and A6 shall be specified.

A8 Buckling Below Depth of Fixity

A8.1 The effects of buckling in a wall element below its depth of fixity (see Section A4.4.1.3 a) shall only be considered when the embedded wall is subjected to an axial load and the ground surrounding the compression part of the wall has a shear strength less than 20kN/m². In such cases the designer should refer to Francis et al (1962).

A8.2 In such cases minimum reinforcement shall be provided in accordance with Section A9 for the length of wall over which buckling needs to be considered. Reinforcement continuity above and below the wall length affected by buckling shall be considered.

A8.3 When considering buckling loads on walls the effect of any lateral displacement of the wall due to horizontal loading shall be taken into account.

A8.4 The buckling resistance of a wall where the surrounding ground has a shear strength less than 20kN/m² may be calculated using the methods of Francis et al (1962) or Poulos and Davis (1980). The following example shows how the buckling resistance of a wall can be calculated and compared with applied compressive axial forces as described in Section A4.

A8.5 Buckling load of a wall, P_{cr}, can be found from:

$$P_{cr} / P_E = (n^2 + \beta / n^2) \qquad \text{(A8.5.1)}$$

where n = number of half sine waves caused by buckling load in wall (see figure A8.1)

P_E = buckling load of a pin-ended strut in air = $\pi^2 EI / L^2$

$\beta = (L / L')^4$

L = length of wall

L′ = length of half sine wave

If L_e is the effective length of the wall considered as a pin-ended strut ie $P_{cr} = \pi^2 EI / L_e^2$

Then $$1/(L_e / L')^2 = 1/(L/ n.L')^2 + (L/ n.L')^2 \qquad \text{(A8.5.2)}$$

and equation A8.5.2 is plotted as figure A8.2.

From figure A8.2 it is found that:

For $L/L' < (1/\sqrt{2})$, ie $L < 0.71.L'$, it can be assumed that the soil offers no support and $L_e = L$

For $L/L' > (1/\sqrt{2})$, ie $L > 0.71.L'$, it can be assumed that $L_e = L' / (\sqrt{2})$ and P_{cr} is twice the buckling load of a pin-ended column in air so $P_{cr} = 2\sqrt{(EIk)}$.

L′ is governed by the soil properties and may be given by:

$L' = (\pi^4.EI/k)^{1/4}$ for a uniform soil or

$L' = (2\pi^4.EI/k_o)^{1/5}$ where soil stiffness is proportional to depth ie k_o is the rate of increase in k with depth

k = coefficient of lateral displacement of soil and may be taken as

$$k = \frac{8\pi E_s (1-\mu)}{1.13(3-4\mu)(1+\mu)(2(\log_e(2L/b)) - 0.443)}$$

where L = length, b = breadth of wall and E_s = modulus of soil (reference Glick (1948).

A8.6 Example

A8.6.1 For a typical uniform soft soil of undrained shear strength Cu = 10 kN/m², the modulus of elasticity of the soil E_s = 500 Cu = 5000 kN/m² and μ = 0.4. For a 15m long, 0.5 m diameter wall:

$$k = \frac{8\pi 5000(1-0.4)}{1.13(3-4\times 0.4)(1+0.4)(2\log_e(2\times 15/0.5)-0.443)} = 4395 kN/m^3$$

$L' = (\pi^4.EI/4395)^{1/4}$ = 6.4m (E = 25 x 10^6 kN/m² and I = $\pi.b^4/64$)

L > 0.71 . L′ therefore $P_{cr} = 2\sqrt{(EIk)}$ = 37MN.

which is equivalent to an applied stress of 188 MN/m², well in excess of the 28 day characteristic concrete strength of say 40 MN/m².

Figure A8.1: Buckled Form of Walls of Various Depths in a Uniform Medium (after Francis et al 1962)

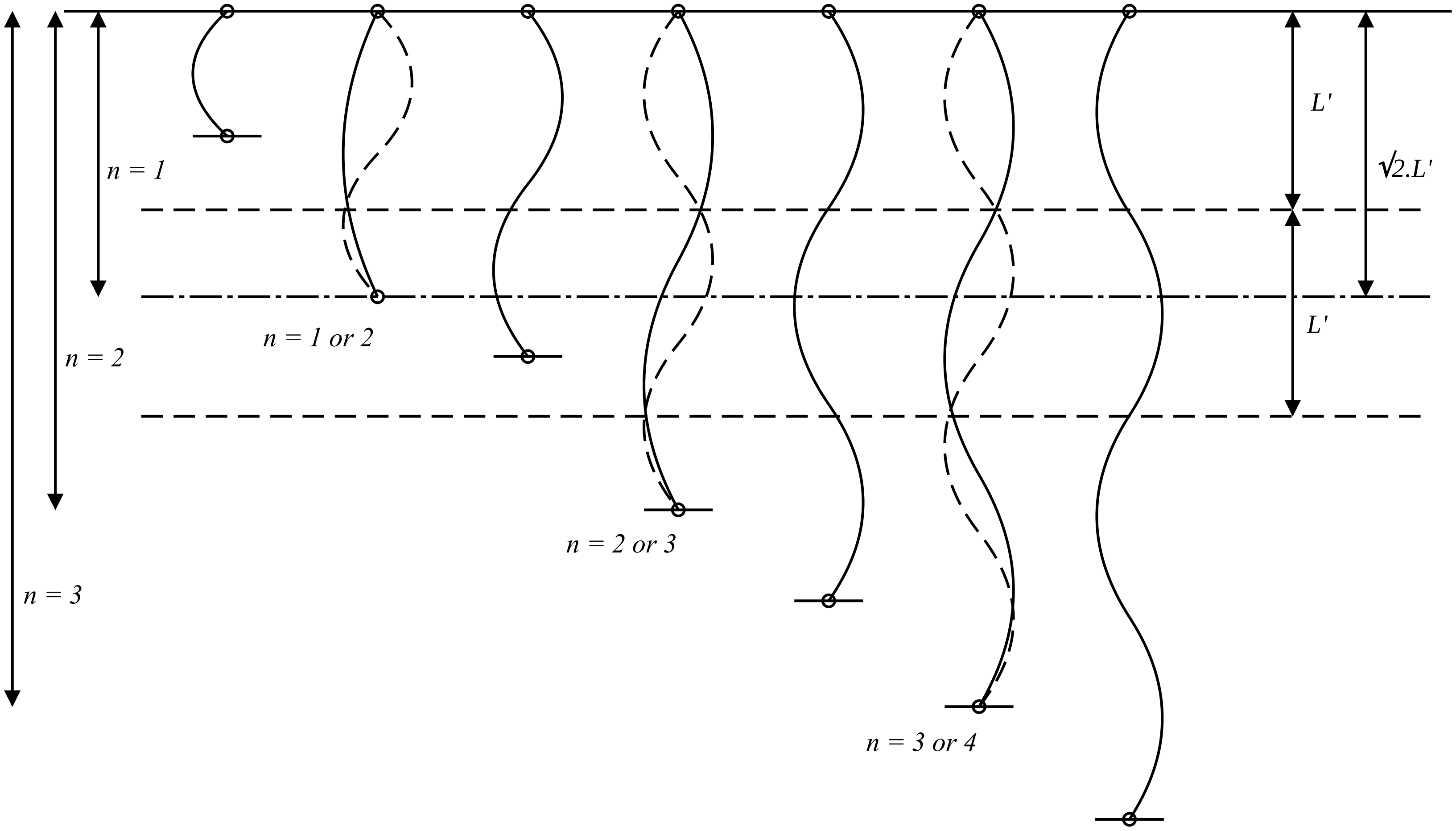


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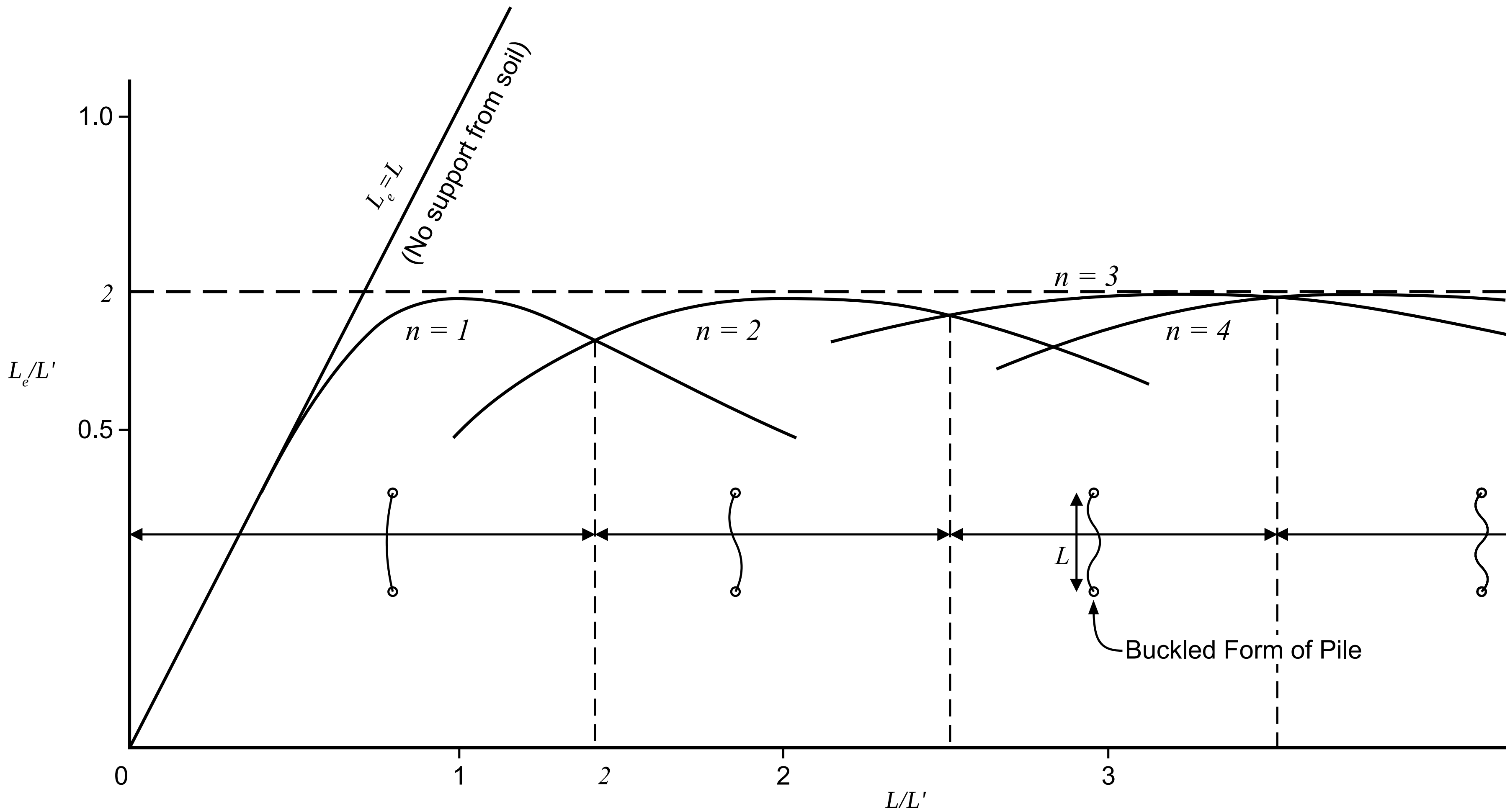
Figure A8.2: Graph of L_e / L′ and L/ L′ for Pin-Ended Wall in Uniform Soil (after Francis et al 1962)



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A9 Minimum Reinforcement Requirements

A9.1 Where the application of this Annex, including Section A12, does not lead to a requirement for wall reinforcement, there is no need to comply with the minimum reinforcement requirements in this Section. Where reinforcement is required to comply with Sections A4, A6, A7 and A8 it shall not be less than the minimum areas stated below, or less than the reinforcement areas that might be required to comply with Section A12.

A9.2 Longitudinal Reinforcement

A9.2.1 Where reinforcement is required to resist bending or axial tension forces, the area of reinforcement spaced evenly around the perimeter of the wall element shall be not less than 0.3% of the cross sectioned area.

A9.2.2 Where reinforcement is required to resist axial compression forces or where the effects of buckling (Section A8) need to be considered, the greater of the following areas of reinforcement shall be provided:

i) Minimum of 0.3% of the cross sectional area of the wall element.

ii) Minimum of 6 No bars spaced evenly around the perimeter of a circular section subject to a maximum spacing of 300mm or a maximum of 4 No bars in a rectangular section, subject to a maximum spacing of 300mm.

Minimum bar size, shall be 12mm diameter.

iii) Not less than $0.15N/f_y$, where N is the ultimate axial load acting on the wall and f_y is the characteristic strength of the longitudinal reinforcement.

A9.3 Transverse Reinforcement (Links)

The provision of nominal transverse reinforcement is required to;

i) resist the buckling of compression reinforcement

ii) to provide minimum reinforcement where buckling capacity is to be considered in Section A8

iii) to provide minimum reinforcement for shear

iv) to provide a rigid reinforcement cage for handling and installation purposes (see Section A12).

Items i) and ii) above shall be met with the provision of minimum links suitable for beams and column elements as set out in Clause 5.8.4.3 of BS 5400: Part 4: 1990 as implemented by BD 24 (DMRB 1.3.1). Item iii) shall be met with the provision of minimum links suitable for resisting shear in beam elements as described in Clause 5.8.4.3 of BS 5400: Part 4: 1990 as implemented by BD 24 (DMRB 1.3.1). Item iv) shall be required to suit the method of construction.

A10 Curtailment of Reinforcement

A10.1 Longitudinal Reinforcement

Longitudinal reinforcement may be curtailed to suit the applied bending moments, axial and shear forces acting on the wall element.

Gradual curtailment of reinforcement is to be considered to discourage horizontal cracking at the point of sudden curtailment of all longitudinal steel. For gradual curtailment the following aspects should be considered:

- Depending on the way in which reinforcement is curtailed, the orientation of the cage may be significant;
- In the case of wall elements where reinforcement is inserted after placing the concrete, the rigidity of the curtailed steel cage shall be designed to ensure that handling and installation is not affected. (See Section A12)

Sudden curtailment of reinforcement can be accepted provided that the wall is checked at the point of curtailment to ensure it is adequate in its cracked state.

A10.2 Transverse Reinforcement

Transverse reinforcement (links) may be curtailed to suit the applied shear forces acting on the wall, in accordance with the requirements of Section A6, A9 and A12.

A11 Serviceability Limit State

A11.1 Corrosion and Durability/Control of Crack Widths

A11.1.1 For the general case of a fully embedded wall in non aggressive ground as defined in Section 5, ready access to oxygen is restricted to a distance of one metre below the surface of formation level of the excavation below which naturally occurring ground surrounds the element. Corrosion action on reinforcement within the wall below this point is likely to be initially slow and, once started, quickly stopped by the deposition of solids. However, if the natural ground at depth below 1 metre is loose granular material with brakish or saline groundwater, the requirements for the first metre shall be applied over this type of ground.

A11.1.2 Protection of reinforcement in walls from corrosion under all conditions is best achieved by good initial site investigation and the provision of dense durable concrete in accordance with Sections A2 and A3.

A11.1.3 Control of crack widths has little effect on the corrosion of reinforcement for buried elements in non aggressive ground. The control of crack widths in accordance with the requirements of Clause 5.8.8.2 of BS 5400: Part 4: 1990 as implemented by BD 24 (DMRB 1.3.1) shall be restricted in such instances to any section of wall element which is not classed as embedded as in Section A11.1.1. The cover for the purposes of this calculation shall be as stated in Section A3 ignoring the additional requirements of BS 8004.

A11.1.4 Heavy corrosion can occur when one or more of the following is present:

- rapid flow of oxygen or carbon dioxide rich groundwater
- highly acidic groundwater
- heavy carbonation of the concrete
- wall deflection or permeable ground permits ingress of water contaminated with chlorides from de-icing salts.

Where aggressive environments are identified crack widths shall be controlled in accordance with the requirements of Clause 5.8.82 of BS 5400:Part 4:1990 as implemented by BD 24 (DRMB 1.3.1) over the length of wall surrounded by ground classified as aggressive or very aggressive in accordance with Section 5. In addition to the specification of concrete resistant to carbonation attack, a protective cover to the reinforcement or protection with a sleeve of a non corrosive or sacrificial material may be considered.

A11.1.5 The effects of sulfate resisting cement and Thaumasite action should be considered by reference to the following documents:

BRE Digest 363: 1991: Sulfate and acid resistance in the ground

DETR (1999) The thaumasite form of sulfate attack: Risks, diagnosis, remedial works and guidance on new construction. Report of the Thaumasite Expert Group. January 1999. The Stationery Office.

A11.2 Fatigue

A11.2.1 Fatigue shall be checked in accordance with BS 5400: Part 4: 1990 as implemented by BD 24 (DMRB 1.3.1) for wall length A.

A12 Construction Considerations Affecting Design

A12.1 Whatever design requirement there may be for reinforcement it shall be increased if necessary to ensure a rigid cage for handling and installation purposes to suit the proposed method of construction.

A12.2 Consideration shall be given to events that may arise on site during construction of the wall, and all subsequent construction operations that could result in the wall elements being subjected to load effects that are more onerous than design load effects already considered in the foregoing sections. The following list, which is not intended to be exhaustive, details considerations that could result in a decision to increase the quantity of reinforcement required to ensure the wall elements remain serviceable.

- Wall elements subject to accidental impact from construction equipment.
- Wall elements subject to disturbance during construction operations.
- Wall elements subject to load testing, static or dynamic.
- Wall elements subject to lateral loading from:

  i) differences in ground levels due to excavation/surcharging of adjacent areas before propping is effective.

  ii) passage of construction traffic within influencing distance of the wall elements.

  iii) variance between the construction sequence implemented and that assumed.