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Incorporating Corrigenda Nos. 1 and 2



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**Recommendations for the design of
structures subject to traffic loading to
BS EN 1997-1:2004+A1:2013**



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Foreword

Publishing information

This Published Document is published by BSI Standards Limited, under licence from The British Standards Institution, and came into effect on 31 May 2011. It was prepared by Technical Committee B/526, *Geotechnics*. A list of organizations represented on this committee can be obtained on request to the committee manager.

Supersession

PD 6694-1:2011+A1:2020 supersedes PD 6694-1:2011 which was withdrawn.

Relationship with other publications

This Published Document gives non-contradictory complementary information for the design of structures subject to traffic loading for use in the UK with BS EN 1997-1 for geotechnical design and its UK National Annex. Background is provided to some of the National Annex provisions where these differ from the values recommended in BS EN 1997-1.

Information about this document

Text introduced or altered by Amendment No. 1 is indicated in the text by tags **A1** **A1**. Minor editorial changes are not tagged.

Text introduced or altered by Corrigenda Nos. 1 and 2 are indicated in the text by tags **C1** **C1** and **C2** **C2**. Minor editorial corrections are not tagged.

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This publication is not to be regarded as a British Standard.

Presentational conventions

The provisions in this Published Document are presented in roman (i.e. upright) type. Its recommendations are expressed in sentences in which the principal auxiliary verb is “should”.

Commentary, explanation and general informative material is presented in smaller italic type, and does not constitute a normative element.

The word “should” is used to express recommendations of this Published Document. The word “may” is used in the text to express permissibility, e.g. as an alternative to the primary recommendation of the clause. The word “can” is used to express possibility, e.g. a consequence of an action or an event.

Notes and commentaries are provided throughout the text of this Published Document. Notes give references and additional information that are important but do not form part of the recommendations. Commentaries give background information.

Where words have alternative spellings, the preferred spelling of the Shorter Oxford English Dictionary is used (e.g. “organization” rather than “organisation”).

As a UK Published Document, these presentational conventions are in accordance with BS 0 and national British Standard drafting rules. Therefore, the conventions might differ from the Eurocode that this Published Document supports.

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0 Introduction

BS EN 1997-1 sets out principles and requirements for the geotechnical design of structures, but in many cases it does not identify specific requirements for particular structure types. This document provides non-contradictory complementary information relating to the design of structures subject to traffic loading to accompany BS EN 1997-1.

1 Scope

This document covers some geotechnical aspects of bridges and other structures subject to traffic loading designed to BS EN 1997-1 and other relevant Eurocodes. The document includes information relating to the design of:

- spread foundations;
- piled foundations;
- gravity retaining walls;
- embedded retaining walls;
- integral bridges; and
- buried concrete structures.

It does not cover the design of reinforced soils.

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes provisions of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

BS EN 1990:2002+A1:2005, *Eurocode – Basis of structural design*¹⁾

[BS EN 1991](#), *Eurocode 1 – Actions on structures*

BS EN 1991-1-5, *Eurocode 1 – Actions on structures – Part 1-5: General actions – Thermal actions*

BS EN 1991-2:2003, *Eurocode 1 – Actions on structures – Part 2: Traffic loads on bridges*

BS EN 1992-2, *Eurocode 2 – Design of concrete structures – Part 2: Concrete bridges – Design and detailing rules*

BS EN 1997-1:2004+A1:2013, *Eurocode 7 – Geotechnical design – Part 1: General rules*

NA to BS EN 1990:2002+A1:2005, *UK National Annex for Eurocode – Basis of structural design*

NA to BS EN 1991-1-5, *UK National Annex to Eurocode 1 – Actions on structures – Part 1-5: General actions – Thermal actions*

NA to BS EN 1991-2:2003, *UK National Annex to Eurocode 1 – Actions on structures – Part 2: Traffic loads on bridges*

NA to BS EN 1997-1, [\[C2\]](#) *UK National Annex to Eurocode 7 – Geotechnical design – Part 1: General rules* [\[C2\]](#)

¹⁾ Informative references are made to the dated version.

3 Terms, definitions, symbols and abbreviations

3.1 Terms and definitions

For the purposes of this Published Document, the terms and definitions given in BS EN 1990, BS EN 1997-1 and the following apply.

3.1.1 ductile structure

structure with sufficient deformation capacity for it to be able to sustain the full plastic collapse load, so that lower-bound and upper-bound theorems of limit analysis are applicable

3.1.2 earth cover

depth of fill measured between ground level and the top of the roof of a buried structure

3.1.3 excavation level

ground level in front of an abutment or embedded wall taking account of any anticipated excavation, and including any unplanned excavation where appropriate

NOTE $\boxed{A_1}$ See BS EN 1997-1:2004+A1:2013, 9.3.2.2. $\boxed{A_1}$

3.1.4 ground level

level of the surface of the ground supported by the structure, including the carriageway or temporary surface carrying traffic where present

3.1.5 hard material

material which requires the use of blasting, breakers or splitters for its removal

3.1.6 K^* pressure

enhanced earth pressure caused by strain ratcheting, with an earth pressure coefficient of K^*

NOTE See 3.1.13 for the definition of strain ratcheting. See Table 1 for the definition of K^* .

3.1.7 $K_{\max}$ pressure

maximum pressure applied behind the abutment of a buried structure

3.1.8 limit equilibrium analysis (for integral bridge design)

analysis method for integral bridges in which the response of the ground is modelled using specified values of mobilized earth pressure coefficients that are independent of the ground stiffness

3.1.9 longitudinal (with reference to a buried structure)

perpendicular to the end walls (or parallel to the traffic direction)

3.1.10 $\boxed{A_1}$ Text deleted $\boxed{A_1}$

3.1.11 quasi-passive limit

limiting passive resistance of the soil at a given average rotational strain in the soil used in soil-structure interaction analysis

NOTE See Annex A.

3.1.12 soil-structure interaction (for integral bridge design)

analysis method for integral bridges in which the response of the ground depends upon the movement of the structure and the relative stiffness of the structure and the ground

3.1.13 strain ratcheting

repeated backward and forward movement of an abutment caused by expansion and contraction of the deck of an integral bridge or the application of live load surcharge behind a bridge abutment which, with time, causes a change in the properties of granular backfill

3.1.14 traffic surcharge

traffic applied behind bridge abutments and retaining walls which generates horizontal pressure on the walls

3.1.15 transverse (with reference to a buried structure)

parallel to the abutments

3.2 Symbols

3.2.1 Latin letters

For the purposes of this Published Document, the symbols given in BS EN 1990, BS EN 1997-1 and the following apply. Other symbols are defined in the clause in which they occur. Symbols for typical buried box structures are shown in [10.1](#).

NOTE The symbol K_0 is defined in BS EN 1997-1 and is included here for completeness.

Table 1 — Latin letters

d	thermal movement of the end of a bridge deck
d'	deflection of an integral bridge abutment at a depth $H/2$ below ground level (see also 9.4.3)
E	Young's modulus
H	height of wall or end screen
H'	depth of soil influenced by abutment movement and used in a soil-structure interaction analysis of an integral bridge
H_c	depth of earth cover between ground level and the top surface of the roof of a buried structure
K	earth pressure coefficient
K_a	coefficient of active earth pressure
K_d	design value of K based on φ'_d and including a model factor $\gamma_{Sd,K}$ if relevant, see 4.7
K_0	coefficient of earth pressure at rest
K_{max}	coefficient of earth pressure applied to buried structures which takes account of pressure increases caused by expansion of the structure
K_{min}	coefficient of minimum earth pressure applied when earth pressure is favourable
K_p	coefficient of passive earth pressure
$K_{p;t}$	coefficient of passive earth pressure used in the calculation of K^* and determined using the design value of the triaxial φ' , see 9.4.1
K_r	coefficient of earth pressure resisting overturning or sliding
K^*	earth pressure coefficient applied to integral bridge abutments subject to strain ratcheting
L_x	expansion length measured from the end of the bridge to the position on the deck that remains stationary when the bridge expands
z	depth below ground level or top of wall

3.2.2 Greek letters

For the purposes of this Published Document, the symbols given in BS EN 1990, BS EN 1997-1 and in [Table 2](#) apply. Other symbols are defined in the clause in which they occur.

NOTE The symbols φ' and φ'_{cv} are defined in BS EN 1997-1 and are included here for completeness.

Table 2 — *Greek letters*

α	coefficient of thermal expansion
β	angle of inclination of backfill
γ_M^*	the reciprocal of the value of γ_M given in the UK National Annex to BS EN 1997-1 used for unfavourable passive pressure and favourable active or favourable at rest pressure
$\gamma_{Sd,K}$	model factor to be applied to earth pressure at ULS
$\gamma_{Sd,ec}$	model factor to be applied to the weight of earth cover at SLS and ULS
δ	structure–ground interface friction angle or friction angle on a vertical or inclined virtual face
θ	skew angle of bridge
φ'	angle of shearing resistance in terms of effective stress
φ'_{cv}	$\langle A1 \rangle$ critical state angle of shearing resistance $\langle A1 \rangle$

3.3 Abbreviations

For the purposes of this Published Document, the abbreviations given in BS EN 1990, BS EN 1997-1 and in [Table 3](#) apply.

Table 3 — *Abbreviations*

Limit states	
EQU	equilibrium ultimate limit state
GEO	geotechnical ultimate limit state
SLS	serviceability limit state
STR	structural ultimate limit state
ULS	ultimate limit state
Other abbreviations	
UDL	uniformly distributed load

4 Basis of design

4.1 Geotechnical category

The geotechnical category should be as given in the project specification $\langle A1 \rangle$ text deleted $\langle A1 \rangle$.

Structures considered as Category 3 should include structures for which there is limited comparable experience, either due to the form of the structure or the nature of the materials used or due to the nature of the geology or hydrogeology, and for which the consequence of failure is severe and/or the performance is predominantly governed by soil–structure interaction.

4.2 Design methods

The Eurocodes give different rules and partial factors for the design of bridges and the design of buildings. Parts of structures, including earth retaining structures, which are subject to significant effects of traffic loading or traffic surcharge should be verified using the rules and partial factors specified for the design of bridges $\langle A1 \rangle$ text deleted $\langle A1 \rangle$.

The geotechnical design should normally be carried out by calculation in accordance with BS EN 1997-1:2004+A1:2013, **2.4**, although prescriptive methods described in BS EN 1997-1:2004+A1:2013, **2.5**, and the Observational Method described in BS EN 1997-1:2004+A1:2013, **2.7**, may be used for appropriate structures $\langle A1 \rangle$ text deleted $\langle A1 \rangle$.

4.3 Actions

4.3.1 Actions to be assessed

The actions to be assessed for inclusion for the geotechnical design of structures subject to traffic loading should include those listed in BS EN 1997-1:2004+A1:2013, **2.4.2**, together with the following:

- a) the effects of wind on the structure and on the traffic carried by the structure;
- b) the effects of stream flow (current velocity); and
- c) impact from floating debris.

4.3.2 Traffic surcharge loading

The traffic surcharge loading to be applied behind the abutments of highway structures and adjacent to retaining walls is specified in the UK National Annex to BS EN 1991-2. The horizontal pressures resulting from this loading are discussed in [7.6](#).

4.4 Dispersion of vertical loads through the fill

Unless a more rigorous analysis is carried out in evaluating vertical stresses, vertical loads can be dispersed from the edge of the load through the fill at an angle of 30° to the vertical. Where the dispersion zone of one or more loads overlaps or where the dispersion zone is curtailed by a retaining wall, reference should be made to [10.2.7](#). Where the loads are being dispersed through strata of varying stiffnesses, reference may be made to Poulos and Davis [[1](#)].

4.5 The serviceability limit state

The serviceability limit state (SLS) is represented by the condition beyond which a loss of utility or cause for public concern might be expected. Settlement, differential settlement, sliding movement, sway of piles, structural deformations and vibrations should be limited at SLS to prevent the following occurring:

- a) danger to the public;
- b) damage to services, drainage and adjacent structures;
- c) encroachment on headrooms, verges and visibility splays;
- d) excessive cracking of concrete elements (see BS EN 1992-2);
- e) excessive opening of joints;
- f) unacceptable visual effects;
- g) unacceptable damage to the carriageway;
- h) vibrations which cause discomfort to the public or potential fatigue problems;
- i) rapid deterioration of elements of the structure (i.e. inadequate durability);
- j) excessive loading or rotation of bearings; and
- k) other limit states being exceeded.

Serviceability limit states can be the critical consideration in design, particularly in soils such as soft clays or loose sand.

4.6 Treatment of permanent actions arising from a single source

The note at the end of [A1](#) BS EN 1997-1:2004+A1:2013 [A1](#), **2.4.2**, explains that in some situations a single partial factor can be used for permanent actions (favourable and unfavourable) which arise from a single source. In contrast to most other actions, design values of geotechnical actions

are dependent upon both γ_F and γ_M (see BS EN 1997-1:2004+A1:2013, 2.4.7.3.2), and this “single source principle” may therefore be applied to both γ_F and γ_M for geotechnical actions. However, as in most cases the destabilizing action is dependent on active or at rest pressure and the stabilizing action is dependent on passive pressure, a single value of γ_M normally produces the most unfavourable effect.

For bridges and earth retaining structures, the “single source principle” can be applied to the following for the structural ultimate limit state (STR) and the geotechnical ultimate limit state (GEO):

- the soil either side of the embedded length of piles of an embedded abutment;
- natural ground above excavation level on an embedded wall, if this is similar to the soil in the embedded section below excavation level;
- earth pressure on different bays of a horizontal spanning counterfort or buttress wall;
- backfill from the same source and compacted to the same specification installed behind propped abutments or the abutments of integral bridges or buried structures; and
- fill placed on different spans of a multispan buried structure.

In accordance with BS EN 1990, the “single source principle” is not applied at the equilibrium limit state (EQU).

The “single source principle” should not normally be applied to the fill placed in front of a retaining structure (i.e. if the earth pressure arising from the fill is treated as an action, it should be taken as favourable), since it might not be compacted to the same quality as the fill behind the retaining structure or it might subsequently be altered to accommodate road widening, installation of services, etc.

Vertical and horizontal components of a single action should be subject to the same partial factor.

4.7 Model factors on earth pressure coefficients

Designs of gravity retaining structures (abutments and walls) carried out in accordance with BS EN 1997-1 using the partial factors on soil weights and soil properties given in the UK National Annex frequently result in structures with lower overall factors of safety on stability and lower ultimate design load effects than structures designed using previous British Standards [including BS 5400 (all parts)²⁾] and the *Design Manual for Roads and Bridges (DMRB)* [2] (prior to the introduction of the Eurocodes).

For example, according to BS 5400 and the DMRB [2], the partial factor combination $\gamma_{fl} \times \gamma_{f3}$ specified for horizontal earth pressure in the ULS was equal to $1,5 \times 1,1 = 1,65$. According to BS EN 1990:2002+A1:2005, the relevant partial factor γ_F is 1,35 for Design Approach 1, Combination 1, for STR/GEO ultimate limit states. In order to achieve a similar level of reliability as before, a model factor $\gamma_{Sd;K} = 1,65/1,35 \approx 1,2$ should be applied to the horizontal earth pressure coefficients (i.e. to K_a or K_0) (for further discussion, see Denton, Christie, Shave and Kidd [3]). Considerations of overall factors of safety for sliding and bearing resistance also show that a value of $\gamma_{Sd;K} = 1,2$ should be applied for Design Approach 1, Combination 2 to maintain a similar level of reliability for these verifications as previously used.

Where it is required to maintain a similar level of reliability to that specified by BS 5400 (all parts)³⁾ and the DMRB [2], a model factor $\gamma_{Sd;K} = 1,2$ should be applied to the earth pressure coefficients K_a and K_0 for unfavourable pressure at all ultimate limit states for Design Approach 1, Combinations 1 and 2. However, for structures where a lower level of reliability can be accepted, a reduced value of $\gamma_{Sd;K}$ may be used.

²⁾ Withdrawn.

³⁾ Withdrawn.

The model factor $\gamma_{\text{sd};K}$ should not be applied to earth pressures in the design of embedded walls nor to the effects of traffic surcharge, nor in calculating K^* , nor for serviceability limit states.

4.8 Constant volume (critical state) angles of shearing resistance

4.8.1 Fine soils

In the absence of reliable laboratory test data, the value of the constant volume (critical state) angle of shearing resistance φ'_{cv} (with effective cohesion $c' = 0$) for fine soils (i.e. clays and silts) may be estimated conservatively from the A1 expressions given in BS 8002:2015, 4.3.1.4 and values in BS 8002:2015, Table 2. The same expressions and values are also presented in [BS 8004:2015](#), 4.3.1.4 and Table 2 respectively. A1

If samples of clay containing A1 partings A1 of sand or silt are remoulded for the plasticity index tests the test results give lower plasticity indices than the clay itself. The tests should be carried out on the clay alone. If there are doubts as to the inclusion of sand or silt A1 a conservative estimate of the plasticity index higher than recorded in the tests should be used to determine the constant volume angle of shearing resistance. A1

A1 Table deleted. A1

4.8.2 Coarse soils

The strength and stiffness of coarse (cohesionless) soils are determined indirectly by in situ static or dynamic penetration tests. Details of three types of penetration tests as well as plate loading tests are given in BS EN ISO 22476 (all parts).

A1 The critical state angle of shearing resistance for siliceous sands and gravels may be estimated from the expressions given in BS 8002:2015, 4.3.1.3 and values in BS 8002:2015, Table 1. The same expressions and values are also presented in [BS 8004:2015](#). A1

4.8.3 A1 Direct determination of design value of the angle of shearing resistance

When the critical state angle of shearing resistance is the value that affects the occurrence of the limit state, it can be more appropriate that the design value of the angle of shearing resistance (φ'_d) is determined directly (see NA+A1:2014 to BS EN 1997-1:2004+A1:2013, **Table A.NA.2**).

For the determination of the design value of the angle of shearing resistance (φ'_d), reference may be made to the expressions in [BS 8004:2015](#), 4.6.2.

NOTE The material partial factor applied to the angle of shearing resistance has been calibrated to be applied to characteristic values of geotechnical parameters which provide the minimum level of safety for conventional designs (see BS EN 1997-1:2004+A1:2013, 2.4.6.2). However, for materials which have reached a critical state condition, the application of this material partial factor could lead to design values which are unreasonable or even physically impossible. Where the strength of a material has reached critical state conditions, a reduced material partial factor can be justified, or the design value of the critical state angle of shearing resistance can be directly determined while maintaining the minimum level of safety required by BS EN 1997-1. A1

5 Spread foundations

5.1 Horizontal earth pressures to be used for the design of spread foundations

When designing spread foundations, it can be assumed that fully active pressure is applied to the earth retaining elements except when the structure is propped or tends to rotate backwards, in which case the enhanced earth pressures described in A1 7.3.2 A1 should be applied. At ultimate limit states, the selected value of the model factor, A1 $\gamma_{\text{sd};K}$ A1 , described in 4.7 should be applied to unfavourable active and at rest earth pressures.

5.2 Bearing resistance

5.2.1 Bearing resistance at the ultimate limit state (ULS)

The bearing resistance of a foundation should be verified at GEO. The resistance may be determined using the methods given in BS EN 1997-1:2004+A1:2013, Annex D, or other recognized methods, provided these methods incorporate the appropriate partial factors γ_F and γ_M given in the National Annex to BS EN 1997-1 and comply with the requirement of BS EN 1997-1:2004+A1:2013, **2.4.1(6)** that any calculation model is either accurate or errs on the side of safety.

The ultimate bearing resistance of the foundation is acceptable if:

- a) the design bearing pressure does not exceed the design bearing resistance; and
- b) settlements under ultimate loading do not lead to another ultimate limit state being exceeded.

Special precautions should be taken when the eccentricity of the design load exceeds one third of the base width [see BS EN 1997-1:2004+A1:2013, **6.5.4(1)**].

5.2.2 Bearing pressures at SLS

Acceptable bearing pressures at SLS are normally governed by settlement considerations.

Where there is a specific limit on the amount of settlement that can be accommodated at SLS, settlement calculations should be carried out using the method given in BS EN 1997-1:2004+A1:2013, Annex F, or other recognized methods.

Where there is no specific limit on the amount of settlement that can be accommodated at SLS, and where there is comparable experience with similar ground, structures and application method, BS EN 1997-1:2004+A1:2013, **2.4.8(4)**, allows the serviceability limit state for settlement to be verified by ensuring that a “sufficiently low fraction of the ground strength is mobilized”.

This requirement can be deemed to be satisfied if the maximum pressure under a foundation at SLS does not exceed one third of the design resistance R/A' calculated in accordance with BS EN 1997-1:2004+A1:2013, Annex D, using characteristic values of ϕ' , c_u and γ' and representative values of horizontal and vertical actions [see also BS EN 1997-1:2004+A1:2013, **6.6.2(16)**].

When determining the dimensions of the foundations, it is prudent to ensure that no uplift occurs anywhere under the foundations at SLS under the characteristic combination of actions, because, once uplift has started to occur, the foundation rotation becomes increasingly sensitive to variations in the eccentricity of the applied load.

5.3 Drained and undrained bearing resistance

The value of the bearing resistance of granular foundations which drain rapidly can be calculated using the method given for drained conditions in BS EN 1997-1:2004+A1:2013, **D.4**, or other recognized methods for drained conditions. Where the foundation is submerged or the water level occurs closely below foundation level, the value of γ' used in the calculation for bearing capacity should be taken as $\gamma_{\text{sat}} - \gamma_w$, where γ_{sat} is the saturated weight density of the soil and γ_w is the weight density of the water.

The value of the bearing resistance of clay foundations and other foundations which might not drain quickly should be calculated using the method given for undrained conditions in BS EN 1997-1:2004+A1:2013, **D.3**, or other recognized methods for undrained conditions, and verified for the drained condition.

NOTE BS EN 1997-1:2004+A1:2013, Annex D omits depth and ground inclination factors which are commonly found in bearing resistance models. Expressions for these factors can be found in [BS 8004:2015](#), **5.4.1**.

5.4 Sliding

BS EN 1997-1:2004+A1:2013, 6.5.3(8) and 6.5.3(11), require that the design shearing resistance be calculated either by factoring the ground properties or the ground resistances. In Design Approach 1, as adopted in the UK, non-unity partial factors are applied to ground properties and therefore only Equations (6.3a) and (6.4a) should be used. Equations (6.3b) and (6.4b) should not be used in conjunction with Design Approach 1.

6 Piled foundations

6.1 Piles subject to horizontal loading

6.1.1 Horizontal loading

There have been cases of excessive movement of piled foundation's supporting abutments that retain adjacent embankments. These movements have been attributed to lateral pressures in the underlying ground caused by the imposed loading of the embankment. In every case, the piling was carried out before all the movement caused by the construction of the embankment had taken place. Where horizontal pressure might develop due to this cause, this should be taken into account in the design in addition to the usual loading which has to be resisted at pile cap level. In the case of a full height piled bridge abutment, it is also possible that additional interaction between the fill and underlying material and the abutment structure can result in increased horizontal loading on the structure and foundation.

Where it is not possible or economical to overcome these effects of soil-induced horizontal loading using construction methods, the effects of this loading should be taken into account in the pile design.

6.1.2 Pile design for soil-induced horizontal loading

For the design of soil-induced loading, reference can be made to CIRIA Report R103 [4] and TRL Project Report 71 [5].

6.2 Design of pile groups

The analysis of pile groups may be carried out using linear elastic methods (as embodied in some design charts), approximate elastic solutions and non-linear elasto-plastic methods. Pile groups are commonly designed using commercially available software. Experience has shown that linear elastic methods can be unduly conservative because they predict unrealistically high axial forces in corner piles.

When designing pile groups, therefore, it is not necessary to check the ultimate axial resistance of every pile, provided that the pile group as a whole has sufficient ultimate resistance to vertical, horizontal and overturning actions, and the following conditions are met.

- a) The pile cap has adequate stiffness and structural resistance to transfer loads across the pile group, from more heavily loaded piles (typically at edges and corners of the group) to less heavily loaded piles.
- b) The design value of the ultimate structural axial resistance of each pile exceeds the upper characteristic geotechnical axial resistance.
- c) The piles exhibit ductile load-settlement behaviour.
- d) Verification of the serviceability limit state is undertaken to confirm that settlement of the pile group is acceptable.

7 Gravity bridge abutments and retaining structures

7.1 Backfill parameters

BS EN 1997-1 requires the soil properties of the backfill to be determined by tests. In practice, it is unusual at the design stage for the source of the backfill to have been tested or even identified. In these circumstances, values of soil properties might therefore need to be assumed.  Assumed values should be realistic and the backfill supplied to site tested to verify that it conforms to the design assumptions (see also [9.10](#)). 

7.2 Earth pressures

7.2.1 Active and at rest pressure

In the design of gravity earth retaining structures, different values of the coefficient of earth pressure (K) might have to be used for the design of the foundations and the structural design of the walls. For the foundation design, active earth pressure may normally be used because sufficient movements can be expected at the GEO limit state (see also [5.1](#)). For the wall design, this might not be the case and active pressure may only be assumed to be mobilized when the wall movements under active pressure are greater than those specified in BS EN 1997-1:2004+A1:2013, [9.5.2\(2\)](#) and Table C.1 (see also [7.3.1](#)). For smaller movements, a pressure between active pressure and at rest pressure should be used, depending on the amount of movement. The value of  K_0  to be used in this situation should be taken from BS EN 1997-1:2004+A1:2013, [9.5.2\(3\)](#) and (4).

7.2.2 Wall friction

In past practice, where there have been doubts as to whether friction on the wall of a retaining structure will be mobilized, the value of the active pressure coefficient for a structural wall has often been calculated using an angle of wall friction, δ_w , of zero  [see for example, *Code of practice for earth retaining structures* (CP2) [1.435](#)⁴⁾]. BS EN 1997-1:2004+A1:2013, [9.5.1\(6\)](#) limits the design value of wall friction to $2/3 \varphi'_{cv,d}$ for precast concrete and steel sheet piling. This limit may also be applied to in-situ concrete walls cast against formwork.  However, BS EN 1997-1:2004+A1:2013, [9.5.1\(4\)](#) notes that the mobilized wall friction and adhesion are functions of the direction and amount of movement of the wall relative to the ground.  Hence, where there is any doubt as to whether wall friction will be mobilized, a cautious value of δ_w should be used.

NOTE Examples of where there can be doubt about how much wall friction will be mobilized include:

- a) situations where the presence of a heel prevents the backfill from moving downwards relative to the wall; or
- b) situations where lateral displacement of the wall is accompanied by vertical movements so that the relative vertical displacement of the soil and structure is reduced. 

7.2.3 Active pressure on gravity cantilever walls with long heels

For a retaining wall with a planar backfill surface inclined at an angle β to the horizontal, the value of the active horizontal thrust applied by the backfill may normally be taken as the horizontal component of the force on the vertical virtual face CD shown in [Figure 1a](#)) calculated using a value of $\delta = \beta$. The justification for this is that it can be shown using wedge analysis that the horizontal thrust calculated in this manner is the same as the horizontal thrust on the critical inclined virtual plane with δ on that plane taken as φ' , provided the critical inclined plane is not curtailed by the face of the structural wall (as described in [7.2.4](#)). Values of K_a for a vertical face with $\delta = \beta$ are given in [Table 4](#).

⁴⁾ Withdrawn.

The critical inclined virtual plane is the plane which requires the maximum horizontal thrust to prevent a slippage. It is not usually the plane joining the back of the heel to the top of the wall. It is inclined at an angle ψ to the horizontal where ψ may be found from the equation:

$$\psi = \frac{1}{2} \left[90^\circ + \varphi' - \beta + \sin^{-1} \left(\frac{\sin \beta}{\sin \varphi'} \right) \right]$$

NOTE This equation is quoted (with a minor typographical error) in Clayton, Milititski and Wood [6] and its derivation is presented in Denton, Christie, Shave and Kidd [3].

Table 4 — Values of K_a for a vertical face when $\delta = \beta$

$\delta = \beta$	Values of K_a						
	$\varphi' = 20^\circ$	$\varphi' = 25^\circ$	$\varphi' = 30^\circ$	$\varphi' = 35^\circ$	$\varphi' = 40^\circ$	$\varphi' = 45^\circ$	$\varphi' = 50^\circ$
0,0	0,49	0,41	0,33	0,27	0,22	0,17	0,13
5,0	0,5	0,41	0,34	0,27	0,22	0,17	0,13
10,0	0,52	0,42	0,34	0,28	0,22	0,17	0,13
15,0	0,58	0,45	0,36	0,29	0,23	0,18	0,14
20,0	0,88	0,51	0,39	0,30	0,24	0,18	0,14
25,0	—	0,82	0,45	0,33	0,25	0,19	0,14
30,0	—	—	0,75	0,38	0,27	0,20	0,15
35,0	—	—	—	0,67	0,32	0,22	0,16
40,0	—	—	—	—	0,59	0,26	0,17
45,0	—	—	—	—	—	0,50	0,21
50,0	—	—	—	—	—	—	0,41

NOTE The values of K_a given in this table are derived using wedge analysis and agree closely with values of K_y derived using the equations given in BS EN 1997-1:2004+A1:2013, Annex C.

7.2.4 Active pressure on gravity $\langle A_1 \rangle$ cantilever $\langle A_1 \rangle$ walls with short heels

For vertical walls with short heels, where α in Figure 1b) is greater than ψ , the critical inclined plane will be curtailed by the face of the structural wall, and the values of K_a on the vertical virtual face CD will vary from $K_{a;\beta}$ to $K_{a;\delta_w}$ as α increases from ψ to 90° as follows:

When β is greater than δ_w , K_a on CD may be taken to increase linearly from $K_{a;\beta}$ to $K_{a;\delta_w}$ as α increases from ψ to 90° . $\langle A_1 \rangle$ Alternatively, K_a may be taken conservatively as $K_{a;\beta}$. $\langle A_1 \rangle$

When β is less than δ_w , K_a on CD may be taken as $K_{a;\beta}$.

where:

- $K_{a;\beta}$ is the value of K_a applied to the vertical virtual face CD if $\delta = \beta$ (as in 7.2.3);
- $K_{a;\delta_w}$ is the value of K_a applied to the vertical virtual face CD if $\delta = \delta_w$;
- δ_w is the value of wall friction assumed on the back of the structural wall;
- ψ is as calculated in 7.2.3, but here not taken as greater than 80° .

$\langle A_1 \rangle$ **NOTE** $K_{a;\delta_w}$ may be calculated using Coulomb wedge analysis, or BS EN 1997-1:2004+A1:2013, Annex C. $\langle A_1 \rangle$

For walls inclined at θ to the vertical [see Figure 1c)], the same approach may be applied except that $K_{a;\delta_w}$ should be taken as the horizontal component of K_a applied to the inclined face of the wall AB with $\delta = \delta_w$. When β is greater than δ_w , K_a on CD may be assumed to increase linearly from $K_{a;\beta}$ to $K_{a;\delta_w}$ as α increases from $\langle A_1 \rangle \psi \langle A_1 \rangle$ to $(90^\circ - \theta)$. If β is less than δ_w , K_a on CD may be taken equal to $K_{a;\beta}$ as for vertical walls.

7.2.5 Active pressure on gravity walls with irregular (non-planer) backfill surface

For walls where the surface of the backfill is inclined for a short distance and then becomes horizontal as in Figure 1d) or for other irregular surfaces where a representative value of β cannot be established, the inclination of the critical plane A'C (ψ) and the associated horizontal active thrust may be determined by trial and error using wedge analysis or by other means with δ taken as φ' on A'C [see Figure 1d)].

If, however, such an analysis indicates that the horizontal thrust on AC is greater than the thrust on any plane A'C (that is when $\psi = \alpha$ in Figure 1 and A'C lies along AC), then the short heel effect described in 7.2.4 should be applied. In these circumstances, the effective value of δ on AC lies between φ' and δ_w and in the absence of a more rigorous analysis, the horizontal thrust on the structure may be assumed to be equal to the thrust on the inclined plane AC with δ on that plane taken as δ_w .

Figure 1 — Earth pressure on retaining structures

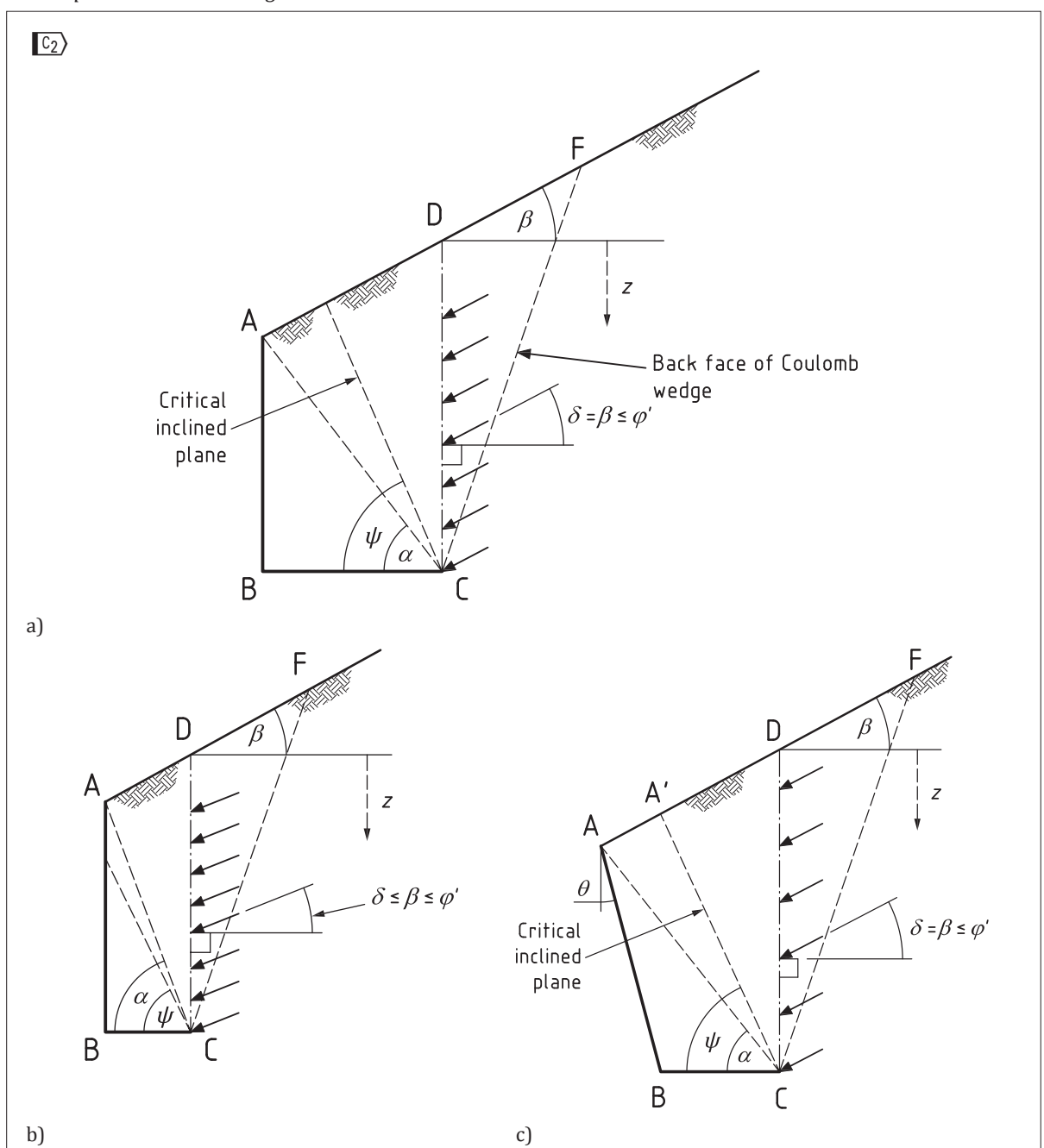
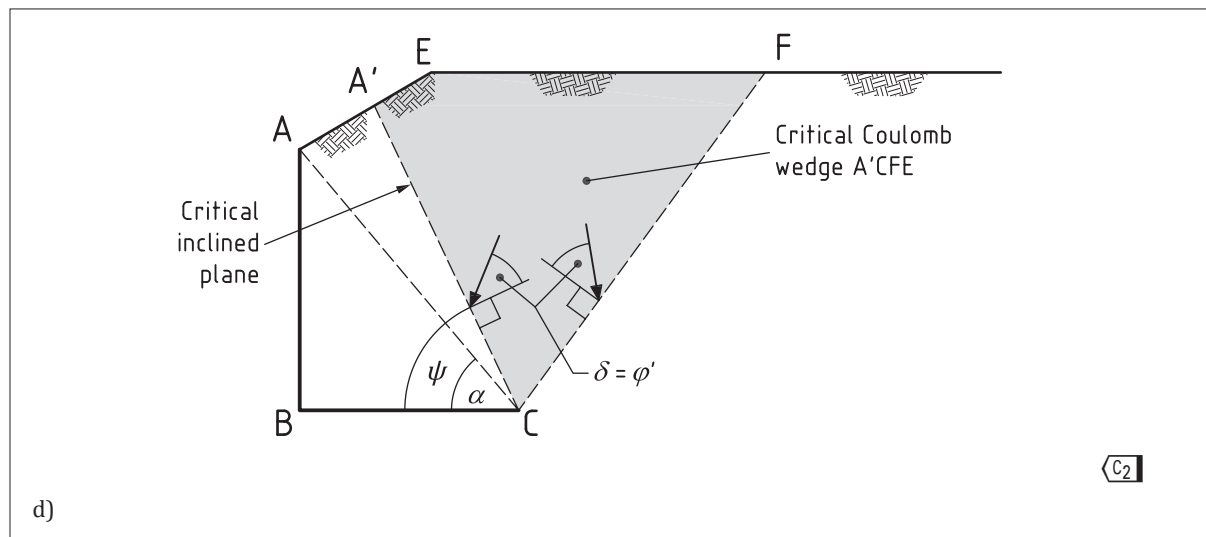


Figure 1 — *Earth pressure on retaining structures (continued)*

7.3 Earth pressures for structural analysis

7.3.1 Earth pressure at SLS

At SLS, at rest pressure should be applied to the wall and other earth retaining elements unless it can be demonstrated that the wall moves sufficiently to allow a lower pressure (between at rest and active) to be developed (see 7.2.1). Calculated deflections used for this purpose should be based on uncracked concrete sections and upper characteristic ground stiffness.

7.3.2 Active and enhanced earth pressures for structural ultimate limit state (STR) verification

For the verification of the STR limit state, active pressure can be applied to the retained face except for the following situations, where the enhanced pressures specified should be used.

a) Recurring traffic surcharge and longitudinal load.

For an earth retaining structure where traffic loading is recurrent, the repeated forward and backward movement of the wall causes permanent deformation of the backfill with a resultant increase in pressure in the fill and locked-in moments in the structural members. For this situation, at rest pressure should be used, except for ductile structures at ULS as described in 7.4, where active pressure may be assumed.

b) Counterfort and propped walls.

Where, due to counterforts, propping or other causes, the wall movements are less than those needed to allow fully active pressure to be mobilized, at rest pressure should be applied, except for ductile structures at ULS as described in 7.4. The movement required to mobilize active pressure can be based on the values given in BS EN 1997-1:2004+A1:2013, Annex C.

c) Reverse rotation.

Where, under permanent loading with active pressure, the pressure under the heel of the base is greater than the pressure under the toe, or where for other reasons a retaining wall tends to rotate backwards, the horizontal design pressure should be taken as the pressure necessary to counteract the backward rotational effect, both at SLS and at ULS, but not less than active pressure and not more than passive pressure.

d) Clay backfills.

Increase in pressure in clay backfills can occur for a number of reasons including pressures prior to construction, the effects of wall installation and soil excavation, the effects of compaction

in layers and other effects causing swelling. Reference should be made to relevant published literature from reputable sources to obtain an appropriate value for the earth pressure when clay backfills are used.

7.3.3 Compaction pressures

Abutments and retaining walls which are backfilled in accordance with the Highways Agency *Manual of Contract Documents for Highway Works (MCHW)* [7] may be deemed adequate to resist compaction pressures applied during construction, provided they are designed to accommodate the traffic surcharge described in 7.6 located adjacent to the wall. Where, however, the surcharge traffic is located remote from the back of the wall (and also for the quasi-permanent combination of actions where $\langle C_2 \rangle \psi_2 \langle C_2 \rangle = 0$ for traffic surcharge), permanent compaction pressures should be assessed.

A1 The effect of compaction may be simulated using the expressions in BS 8002:2015, 4.5.2.5.

Conservatively, the effects of compaction may be assumed to be a constant magnitude from ground level with no allowance for the reduced compaction pressure near the ground surface. This assumption is only slightly conservative as the maximum compaction pressure usually occurs only a short depth below the surface. **A1**

Where compaction stresses are critical, the value of the weight of the compaction roller assumed in the design should be specified in the contract. Recommended values for the restricted weight of rollers used to backfill highway structures are given in Clause 610 of the *MCHW* [7]. The partial factor γ_Q to be applied to the roller weight should be the same as for road traffic actions.

7.4 Ductile structures and brittle failure modes

If in the situations described in 7.3.2a) and 7.3.2b) the structure is capable of undergoing sufficiently large deformations to mobilize active pressure prior to any loss of structural resistance, the structure can be considered to be ductile and the applied pressure at STR can be taken to be active pressure.

However, the following modes of failure are generally not sufficiently ductile for fully active pressure to be mobilized and in such cases, the enhanced earth pressure described in 7.3.2a) and 7.3.2b) should be used at STR and SLS:

- a) shear failure in concrete or masonry;
- b) bending failure of over-reinforced concrete in which compression failure of the concrete occurs before the tension reinforcement yields;
- c) tension failure in mass concrete or masonry;
- d) buckling of slender props located near the top of the wall;
- e) failure of strut/waler and similar connections;
- f) tensile failure of non-ductile anchors;
- g) bond failure (lap length and anchorage length) in reinforced concrete; and
- h) bearing stress on the inside of a bend in reinforced concrete.

In determining whether ductile failure would precede brittle failure, upper-bound values of the strengths of the ductile materials should be used. For reinforcement, the upper-bound yield strength should be taken as 30% higher than the characteristic strength [see BS EN 1992-1-1:2004, C.2(1)P].

7.5 Movement required to generate passive pressure

The mobilization of full passive pressure can involve unacceptably large horizontal movements, especially at SLS. BS EN 1997-1:2004+A1:2013, Table C.2, defines these movements in terms of the ratio v/h , where v is the wall displacement and h is the height of the wall. The table gives the range of