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

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Foreword

Publishing information

This Published Document was published by BSI and came into effect on 31 May 2011. It was prepared under the authority of B/526, *Geotechnics*. A list of organizations represented on this committee can be obtained on request to its secretary.

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.

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.

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.

Contractual and legal considerations

This publication does not purport to include all the necessary provisions of a contract. Users are responsible for its correct application.

Compliance with a Published Document cannot confer immunity from legal obligations.

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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 referenced documents are indispensable for the application 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*

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, *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, *UK National Annex to Eurocode 7 – Geotechnical design – Part 1: General rules*

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 See BS EN 1997-1:2004, **9.3.2.2**.*

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 Overseeing Organization
client or other relevant technical authority

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_o 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_o	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}	critical angle of shearing resistance

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 or agreed with the Overseeing Organization.

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, unless otherwise agreed with the Overseeing Organization.

The geotechnical design should normally be carried out by calculation in accordance with BS EN 1997-1:2004, **2.4**, although prescriptive methods described in BS EN 1997-1:2004, **2.5**, and the Observational Method described in BS EN 1997-1:2004, **2.7**, may be used for appropriate structures, with the agreement of the Overseeing Organization.

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, **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 BS EN 1997-1:2004, **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, **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 resisting 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):

a) the soil either side of the embedded length of piles of an embedded abutment;

b) natural ground above excavation level on an embedded wall, if this is similar to the soil in the embedded section below excavation level;

c) earth pressure on different bays of a horizontal spanning counterfort or buttress wall;

d) 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

e) 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) [1)]] and the *Design Manual for Roads and Bridges (DMRB)* [2].

For example, according to BS 5400 and the *DMRB*, the partial factor combination $\gamma_{fL} . \gamma_{f3}$ specified for horizontal earth pressure in the ULS was equal to 1,5 × 1,1 = 1,65. According to BS EN 1990, 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 ≈ 1,2 should be applied to the horizontal earth pressure coefficients (i.e. to K_a or K_o) (For further discussion, see Denton, Christie, Shave and Kidd [3]).

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

The model factor $\gamma_{Sd;K}$ 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 values given in Table 4.

If samples of clay containing veins or seams 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 then, in Table 4, use the next value of the plasticity index higher than recorded in the tests.

Table 4 **φ'_{cv} for fine soils**

Plasticity index %	15	30	50	80
φ'_{cv} degrees	30	25	20	15

1) Withdrawn.

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).

The critical state angle of shearing resistance for siliceous sands and gravels may be estimated from the following equation:

$$\varphi'_{cv} = \varphi'_{cv,base} + \varphi'_{cv,A} + \varphi'_{cv,B}$$

where the values of the various components of φ'_{cv} are given in Table 5 depending on the particle size distribution and classifications according to BS EN ISO 14688 (all parts).

Table 5 **φ'_{cv} for coarse soils**

Base value	**Angularity of particles**		**Grading of coarse soil**	
$\varphi'_{cv,base}$ degrees		$\varphi'_{cv,A}$ degrees		$\varphi'_{cv,B}$ degrees
30	Well-rounded/rounded	0	Even-graded	0
	Sub-rounded/sub-angular	2	Medium graded	2
	Angular/very angular	4	Multi-graded	4

Bolton [4] has introduced empirical relations between φ', φ'_{cv}, initial soil relative density, and mean effective stress at failure to reflect the change in the secant value of peak angle of shearing resistance with the change in the mean effective stress in the ground.

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 **7.3.3** should be applied. At ultimate limit states, the selected value of the model factor, $\gamma_{sd;K}$, 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, 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, **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, **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, 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, **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, 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, **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, **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 $\gamma_{sat} - \gamma_w$, 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, **D.3**, or other recognized methods for undrained conditions, and verified for the drained condition.

5.4 Sliding

BS EN 1997-1:2004, **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 [5] and TRL Project Report 71 [6].

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 retaining structures and bridge abutments

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 should be stated in the contract. The backfill supplied to site should be verified to ensure that it conforms to the design assumptions (see also **9.10**) and is in accordance with the contract requirements.

7.2 Earth pressure on retaining walls and abutments

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, **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_o to be used in this situation should be taken from BS EN 1997-1:2004, **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**[2)]]. BS EN 1997-1:2004, **9.5.1**(6), allows a design value of wall friction of up to 2/3 $\varphi'_{d:cv}$ to be used for concrete walls and sheet piles. However, BS EN 1997-1:2004, **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 [because the presence of a heel could prevent the backfill from moving downwards relative to the wall or for other reasons such as those listed in BS 8002:1994, **3.2.6**[3)]] a cautious value of δ_w should be used.

2) Withdrawn.

3) Superseded, withdrawn. Replaced by BS EN 1997-1.

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 6.

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:

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

NOTE This equation is quoted (with a minor typographical error) in Clayton, Milititski and Wood [7] and its derivation is presented in Denton, Christie, Shave and Kidd [3].

Table 6 **Values of K_a for a vertical face when $\delta = \beta$**

	Values of K_a						
$\delta = \beta$	φ' = 20°	φ' = 25°	φ' = 30°	φ' = 35°	φ' = 40°	φ' = 45°	φ' = 50°
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_γ derived using the equations given in BS EN 1997-1:2004, Annex C.

7.2.4 Active pressure on gravity 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°.

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°.

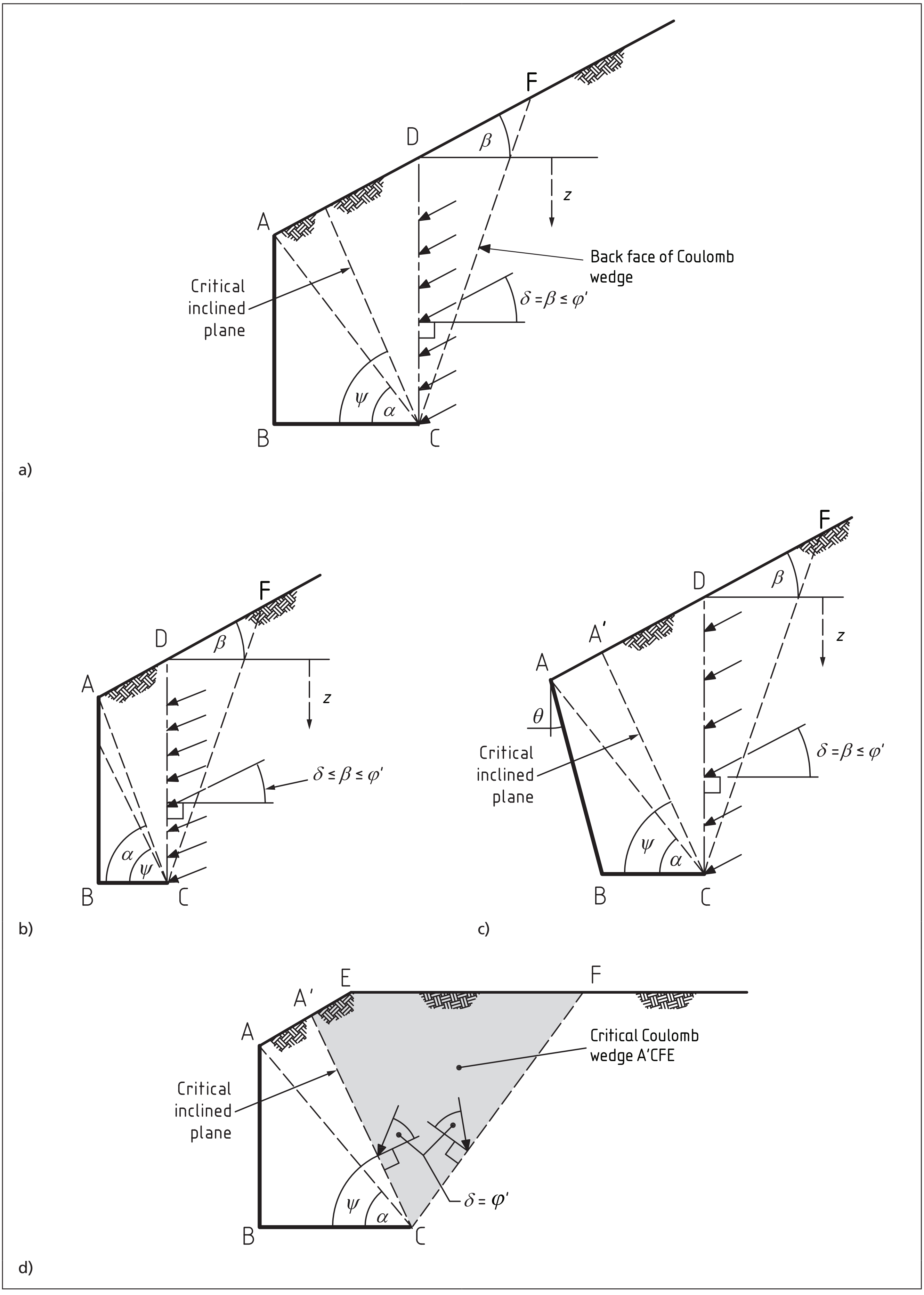
For walls inclined at θ to the vertical [see Figure 1c)], the same approach may be applied except that $K_{a,\delta w}$ should be taken as the horizontal component of K_a applied to the inclined face of the wall AB with $\delta = \delta_w$. When β is greater than δ_w, K_a on CD may be assumed to increase linearly from $K_{a;\beta}$ to $K_{a;\delta w}$ as α increases from ψ' to (90° – θ). 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**



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, 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)* [8] 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 $\psi_2 = 0$ for traffic surcharge), permanent compaction pressures should be assessed. The effect of compaction may be simulated by applying an additional horizontal pressure on the top section of the wall that reduces linearly from:

$$\sigma_{top} = \sqrt{(2V\gamma / \pi)} \text{ kN/m}^2$$

at ground level to zero at a depth of

$$\frac{\sigma_{top}}{\gamma K_d}$$

where:

- V is the design value of the weight of the compaction roller in kN per metre width;
- γ is the design value of the soil weight density in kN/m^3;
- K_d is the design value of the earth pressure coefficient including the model factor $\gamma_{Sd;K}$ at ULS where relevant (see **4.7**).

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* [8]. 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 **7.3.2**a) and **7.3.2**b) 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 **7.3.2**a) and **7.3.2**b) 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, 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 values of *v*/*h* needed to mobilize full passive pressure (v_p/h) and half passive pressure (termed here v_2/h), the latter being given in brackets beside the value of v_p/h for full K_p. For intermediate values, it may be assumed that the coefficient of earth pressure (*K*) increases linearly from K_o to $0{,}5K_p$ as the movement (*v*/*h*) increases from zero to v_2/h. For values of *K* greater than $0{,}5K_p$, the relationship between *K* and *v* can be found from the (curve-fitted) equation:

$$K = K_p \left[1 - 0{,}5 \left\{ \frac{(v_p/h) - (v/h)}{(v_p/h) - (v_2/h)} \right\}^3 \right]$$

It can be noted that the movements required to mobilize full passive pressure and half passive pressure given in BS EN 1997-1:2004, Table C.2, can be significantly greater than those recommended by Hambly [9] and traditionally used in UK bridge design practice.

7.6 Horizontal earth pressure due to traffic surcharge loading behind the abutments of highway bridges

7.6.1 Principles

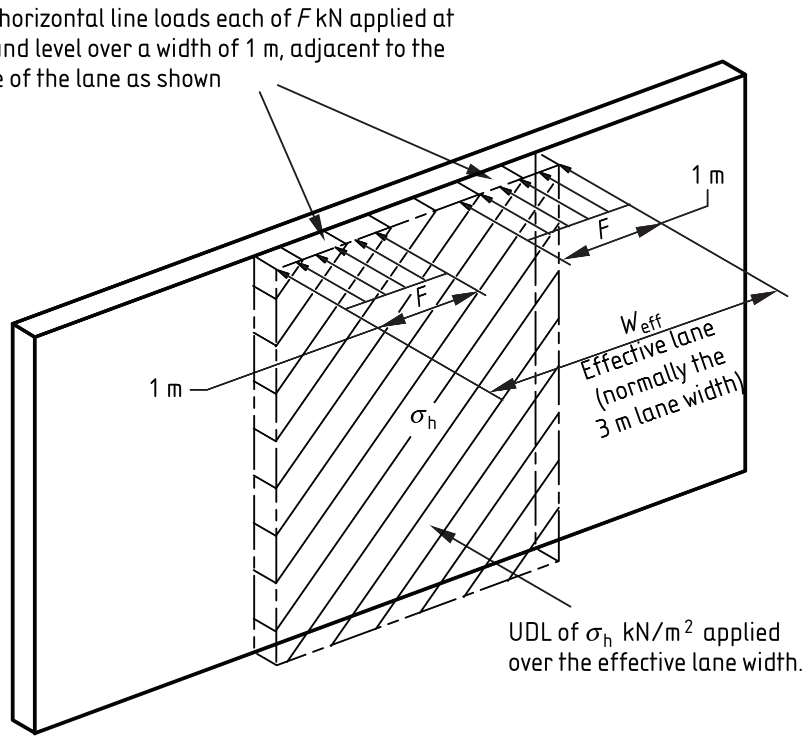
Details of the traffic loads to be used when evaluating the traffic surcharge pressures on an abutment wall or wing wall are given in the UK National Annex to BS EN 1991-2. Any rigorous analysis of the horizontal pressures generated by the traffic surcharge load should take into account the strength of the backfill, the movement of the wall, the proximity of the wheel loads to the wall and the high pressure concentrations that develop just below the surface when a heavy wheel load is applied immediately behind the wall.

In the absence of a rigorous analysis, the effect of the BS EN 1991-2 vertical traffic surcharge applied behind abutments, wing walls and retaining walls, may be found using the models given in **7.6.2** and **7.6.3** respectively. The background to these models is given by Denton, Christie, Shave and Kidd [3].

7.6.2 Horizontal surcharge model for vertical cantilever and vertically spanning abutments

For vertical cantilever and vertically spanning abutments, the effect of each lane of traffic surcharge may be modelled as two horizontal line loads applied at the top of the wall together with a uniformly distributed load applied over the full height of the wall and the effective lane width (W_{eff}). The values of the effective lane widths and horizontal surcharge loads for various situations are given in Figure 2 and Table 7 respectively.

Figure 2 **Horizontal surcharge model**



NOTE 1 The figure shows the surcharge model for one lane of effective width W_{eff}. The models for adjacent lanes are similar. For most situations, W_{eff} will be equal to 3,0 m. However, for carriageways between 5,4 m and 6,0 m in width subject solely to normal traffic (see National Annex to BS EN 1991-2:2003, ***NA.2.34.3****), the effective lane width, W_{eff}, will be equal to the notional lane width, see BS EN 1991-2:2003,* ***4.2.3****.*

NOTE 2 For carriageways subject to SV or SOV (LM3) loading (see National Annex to BS EN 1991-2:2003, ***NA.2.16****) which encroaches on the adjacent notional lane, the effective width of that lane available for carrying normal traffic should be taken as the distance from the edge of the LM3 vehicle to the far edge of the notional lane (see UK National Annex to BS EN 1991-2:2003,* ***NA.2.16.4****), except that if that width is less than 2,5 m, no normal traffic need be applied adjacent to the LM3 vehicle in that lane.*

NOTE 3 The effective width of a lane carrying LM3 loading used in applying the surcharge model is always 3,0 m.

Table 7 **Horizontal surcharge model for highway traffic loading**

	Loading from normal traffic		**SV/100 and SV/196** A)		**SOV model vehicles**	
	Horizontal line load F at top of structure B)	Horizontal uniformly distributed load (UDL) σ_h	Horizontal line load F at top of structure B)	Horizontal UDL σ_h	Horizontal line load F at top of structure B)	Horizontal UDL σ_h
Case A: Characteristic horizontal surcharge for each effective lane of traffic C), D)	Two 1 m wide line loads each $330K_dL_f$ (kN)	3 m wide UDL of $20K_dRL_f$ (kN/m^2)	Two 1 m wide line loads each $330K_d$ (kN)	3 m wide UDL of $30K_d$ (kN/m^2)	Two 1 m wide line loads each $330K_d$ (kN)	3 m wide UDL of $45K_d$ (kN/m^2)
Case B: Characteristic horizontal surcharge for a metre width design E)	$330K_dD_f$ (kN/m) width	$20K_dR$ (kN/m^2)	$330K_d\ D_f$ (kN/m) width	$30K_d$ (kN/m^2)	$330K_dD_f$ (kN/m) width	$45K_d$ (kN/m^2)
Case C: Characteristic horizontal surcharge for segmental structures F)	$330K_d$ (kN/m) width	$20K_d$R (kN/m^2)	$330K_d$ (kN/m) width	$30K_d$ (kN/m^2)	$330K_d$ (kN/m) width	$45K_d$ (kN/m^2)

where:

K_d is the design value of K_a or K_o (as appropriate) based on Φ'_d;

R is $3{,}0/W_{eff}$;

L_f is the lane factor specified in the UK National Annex to BS EN 1991-2:2003, **NA.2.34.2**; and

D_f is the dispersion factor applied to the effect of the horizontal line load in a metre-strip design E).

A) The SV80 loading may be assumed to be less critical than loading from normal traffic (see National Annex to BS EN 1991-2:2003, **NA.2.34**).

B) For buried structures with less than 2 m of fill above the top of the structure, the horizontal line loads should be applied at the top of the structure and reduced by the factor $(1 - H_c/2)^2$, where H_c is the depth of fill measured in metres from the surface of the carriageway to the top of the structure. For cover depths in excess of 2 m, the horizontal line loads may be ignored.

C) For the design of the foundations (sliding, overturning and bearing pressure) of a transversely stiff structure, and other global effects, the total surcharge load on the wall may be taken as the sum of the horizontal surcharge loads applied by each individual lane of traffic given in Case A and Figure 2 and distributed over the whole width of the abutment.

D) Where it is necessary to find the moments and shears in a vertically spanning, transversely stiff abutment wall using a computer analysis, the individual horizontal lane loads given in Case A and Figure 2 may be applied directly onto the computer model.

E) For a metre-strip analysis of a transversely stiff, vertically spanning wall, the loads per metre width given in Case B may be used. The dispersion factor D_f allows for the dispersion of the effect of the horizontal line load with depth and may be taken as $(1 + z/2)/(1 + z)$ where z is the distance below the top of the wall at which the analysis is being carried out. D_f should not be taken as less than 0,67.

F) For segmental walls and other walls with little or no transverse strength, the loads per metre width given in Case C may be used in the design.

NOTE The loads given in this table include the overload and dynamic amplification factors specified in the National Annex to BS EN 1991-2, but exclude all relevant ψ factors and γ_F partial factors.

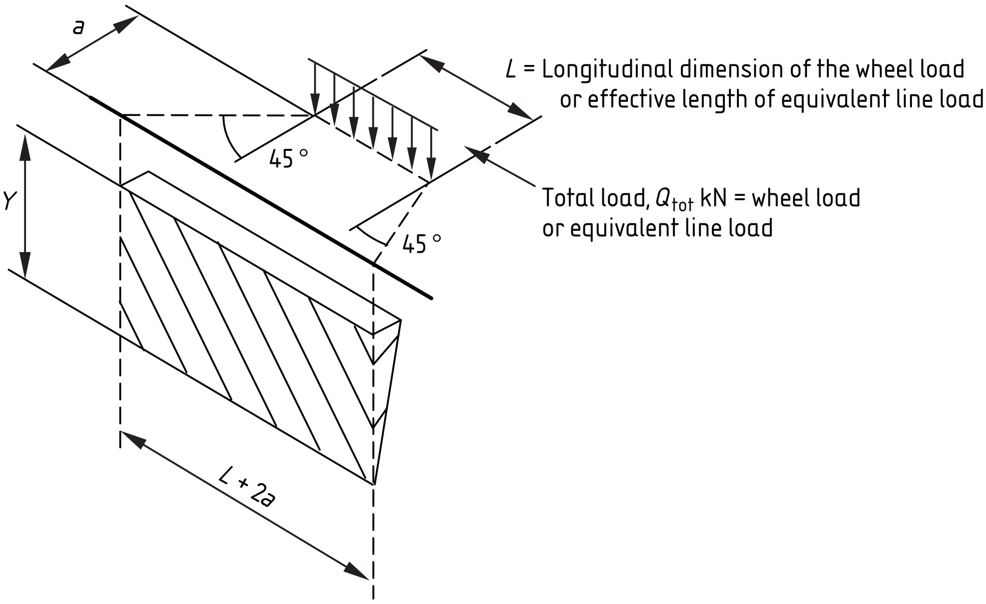
7.6.3 Surcharge model for wing walls and other earth retaining structures

For other earth retaining structures such as wing walls, walls parallel to or inclined to the direction of the traffic, transversely spanning counterfort walls and walls remote from the carriageway, the effect of the vertical surcharge due to each wheel load may be found using the model shown in Figure 3.

For short walls, and for determining local concentrated pressures on a wall, the pressures arising from all the wheels affecting the structure or relevant part of the structure should be superimposed. However, for global effects on walls, each lane of traffic can be modelled as two parallel, uniformly-distributed line loads except for the SOV model vehicle which can be modelled as four parallel, uniformly-distributed line loads. A load of Q_{tot} kN may be assumed to exert a total horizontal force of Q_{tot} tan ($\alpha - \varphi'_d$) on the wall where the angle α is taken as 45 + (φ'_d/2) or $\tan^{-1}(H/a)$, whichever is the smaller [see Figure 3b) and Figure 3c)].

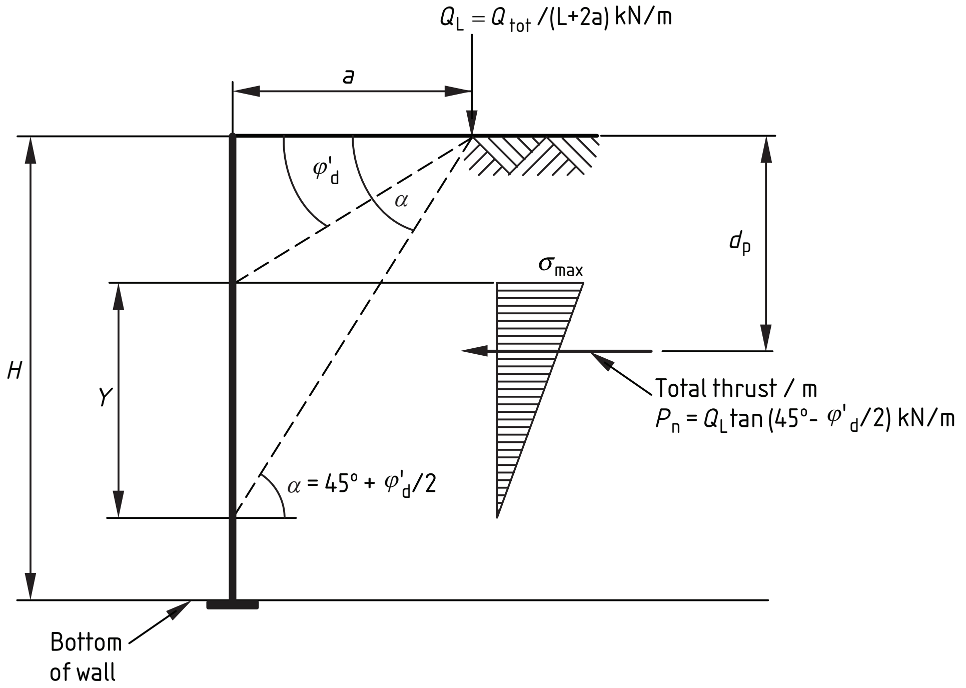
This model may also be used for determining the horizontal effects of vertical loads other than traffic loads. Patch loads or strip loads of significant width can be simulated by superimposing the effects of a number of parallel line loads.

Figure 3 **Lateral and vertical dispersion of finite line loads for calculating horizontal surcharge pressure**

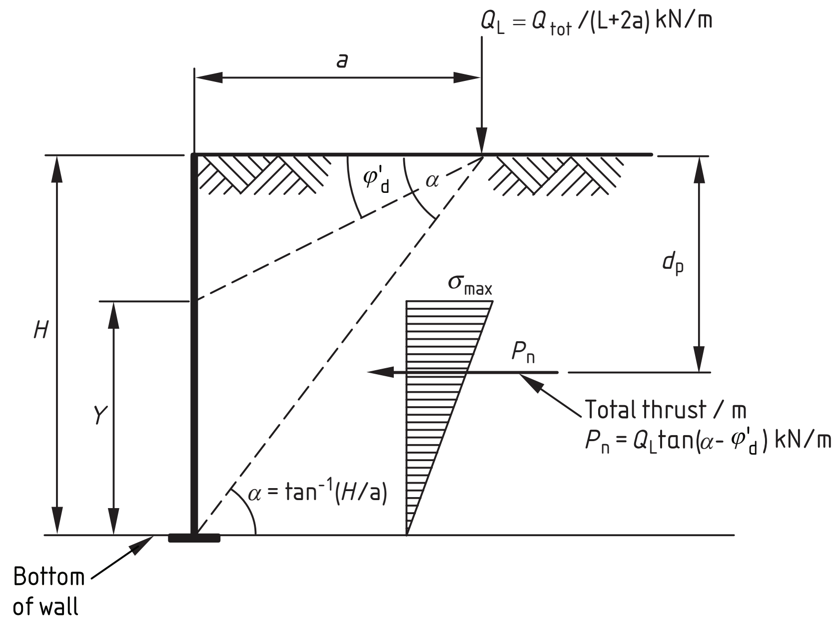


a)

Figure 3 **Lateral and vertical dispersion of finite line loads for calculating horizontal surcharge pressure** *(continued)*



b)



c)

7.7 Hydrostatic pressure

As discussed by Denton, Kidd, Simpson and Bond [10], BS EN 1997-1 allows several different approaches to be used to account for ground-water pressure. For example, the design value of ground-water pressure may be directly assessed in accordance with BS EN 1997-1:2004, **2.4.6.1**(2)P and **2.4.6.1**(6)P, or determined by applying a safety margin to the characteristic water level in accordance with BS EN 1997-1:2004, **2.4.6.1**(8), or by applying a partial factor to the characteristic water pressure in accordance with BS EN 1997-1:2004, **2.4.6.1**(8).

Because of the site-specific nature of uncertainty in water levels and the associated difficulties in calibration, no partial factor is given for ground-water pressure in the UK National Annex to BS EN 1990 for the design of bridges. If a partial factor is applied to the characteristic water pressure, the appropriate value should be established on a project-specific basis taking account of the particular circumstances in which it is used.

In cases where the effects of water pressure are small (if they are present at all), the design is insensitive to the approach used. Past practice has generally been to use "directly determined" cautious values without applying a partial factor, and this remains an available option.

However, if the hydrostatic effects are predominant and it is unrealistic to apply a significant safety margin to the water level (because, for example, the characteristic water level is close to the top of the retaining structure), it is advisable to apply a partial model-uncertainty factor to the effect of hydrostatic pressure even when the level and weight density of water are known with a high degree of certainty. This factor is required to take account of inaccurate assessment of the effects of loading, unforeseen stress distribution in the structure, construction tolerances and other secondary effects normally covered by the model factor incorporated in γ_F (see BS EN 1990:2002+A1, **6.3.2**, and UK National Annex to BS EN 1990:2002+A1, Table NA.A2.4(B), Note 9, and Table NA.A2.4(C), Note 9).

8 Embedded walls

Embedded walls other than those that are integral with bridge decks, may be designed using the non-contradictory complementary information in CIRIA Report C580 [11]. The earth pressures on the retained face of embedded walls may be calculated using the information in Clause **7** where appropriate. Guidance on the design of embedded walls in soft clays is given by Karlsrud and Andresen [12].

9 Integral bridges

9.1 General

Integral bridges are bridges without joints in the deck that accommodate expansion and contraction by movement of the abutments in and out of the backfill. In granular soils, this repeated contraction and expansion causes particle realignment and an associated increase in the soil stiffness and the mobilized passive resistance. This effect results in a progressive year-on-year increase in soil pressure which is termed "soil ratcheting".

The Eurocodes do not provide specific guidance on the design of integral bridges. Clause **9** is therefore provided to help designers make a realistic estimate of the pressures that will develop with time behind integral abutments. The recommendations are based on the results of a number of laboratory model tests carried out over recent years, see England, et al. [13], Hambly, et al. [14], Springman, et al. [15], Tapper, et al. [16] and Tan, et al. [17].

The expressions for earth pressure given in Clause **9** do not include the effects of ground-water. The effects of ground-water should be taken into account in the design, if necessary (see BS EN 1997-1:2004, Annex C).

*NOTE Background to aspects of Clause **9** and Annex A is given by Denton, Riches, Christie and Kidd [18].*

9.2 Methods of analysis

9.2.1 Limit equilibrium methods

The limit equilibrium analysis methods described in **9.4.3** and **9.4.4** are applicable to abutments where:

a) the characteristic thermal movement of the end of the deck does not exceed 40 mm;

b) the skew does not exceed 30°; and

c) the depth of soil affected by the abutment movement can be identified without recourse to a soil–structure interaction analysis, for example, abutments founded on spread footings (**9.4.3**) and end screen abutments (**9.4.4**). Abutments seated on pile caps consisting of more than one row of piles may also be designed using the methods described in **9.4.3** provided the sway at pile cap level is sufficiently small for at rest pressure to be considered as acting at pile cap level.

The limit equilibrium methods described in **9.4.3** and **9.4.4** are not appropriate for the following:

1) abutments founded on a single row of piles;

2) embedded wall abutments;

3) over-consolidated backfill material;

4) cohesive soils; and

5) layered soils.

9.2.2 Soil–structure interaction

As an alternative to the limit equilibrium methods described in **9.2.1** and for bridges excluded by the requirements of **9.2.1** such as piled abutments and embedded wall abutments, the horizontal earth pressures on integral abutments may be evaluated using recognized soil–structure interaction methods incorporating an appropriate numerical model of the soil properties.

Where numerical modelling is carried out, it is important that the approach used has been calibrated against comparable experience, laboratory modelling and/or case history data experience.

9.2.3 Short bridges

Integral bridges with an expansion length L_x exceeding 10 m which conform to the requirements in **9.2.1** or **9.2.2** should be designed in accordance with this Clause **9**. Integral bridges with an expansion length of less than 7,5 m may be designed as described in Clause **10** on buried concrete structures.

Integral bridges with an expansion length between 7,5 m and 10 m may conservatively be designed in accordance with Clause **9** or alternatively (for structures designed using the limit equilibrium method) by assuming that the earth pressure at any depth varies linearly between the values for a buried structure of 15 m overall length to the values for an integral bridge with a 10 m expansion length designed in accordance with Clause **9**.

9.3 Types of abutment for integral construction

9.3.1 General

The information in this document is relevant to the types of integral and semi-integral abutments illustrated in Figure 4 and described as follows. There are three main categories of abutment used with integral bridges:

a) full height frame abutments shown in Figure 4a), Figure 4b) and Figure 4c) and described in **9.3.2**;

b) embedded wall abutments shown in Figure 4d) and described in **9.3.3**; and

c) end screen abutments shown in Figure 4e) to Figure 4m) and described in **9.3.4** to **9.3.7**.

NOTE End screen abutments include bank pad abutments (supported on the ground or on piles), flexible support abutments and semi-integral abutments, where the earth pressure resisting expansion is applied to the end screen at the end of the deck only.

9.3.2 Frame abutments

These support the bridge deck and act as retaining walls for the backfill. They are connected structurally to the deck and are supported on spread footings or piled foundations as shown in Figure 4a), Figure 4b) and Figure 4c).

9.3.3 Embedded wall abutments

These include bored pile, sheet pile or diaphragm wall abutments, which extend to a depth below the retained fill as shown in Figure 4d). The walls are integral with the bridge deck.

9.3.4 Bank pad abutments

These are effectively extensions to the deck that are seated on the backfill and act as end supports for the bridge as shown in Figure 4e) and Figure 4f). They slide on the foundation in response to thermal expansion and contraction on the deck and can rotate under live loading. The bank pad should have adequate weight to provide stability to the structure, and in multi-span structures, the end span should have adequate flexibility to accommodate differential settlement and to avoid uplift when remote spans are subject to traffic loading.

9.3.5 Bank pad abutments on piles

These are bank pad abutments founded on a single row of discrete vertical piles which are driven or bored through an embankment or cutting slope. The top of the piles are integral with the deck. When the deck expands, the pile cap and the end of the deck move into the backfill without significant rotation, whereas the piles flex backward into the fill.

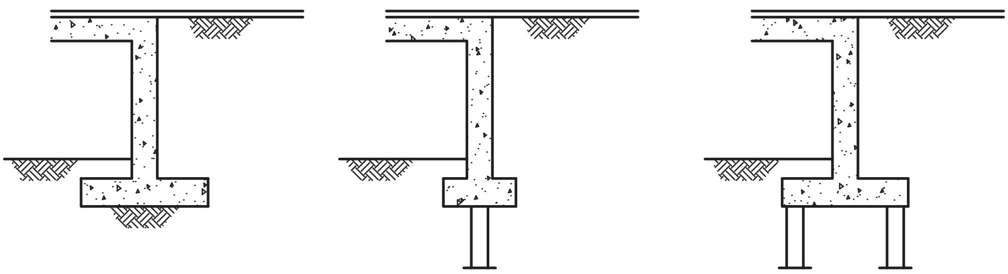
9.3.6 Flexible support abutments

These are abutments in which the deck is supported on flexible piles or columns. The piles or columns are either enclosed in sleeves to allow them to flex without displacing the surrounding soil as shown in Figure 4h) and Figure 4i), or they are located in front of a reinforced soil or similar abutment as shown in Figure 4j). In these types of abutment, only the end screen, which is attached to the end of the deck, moves into the fill.

9.3.7 Semi-integral abutments

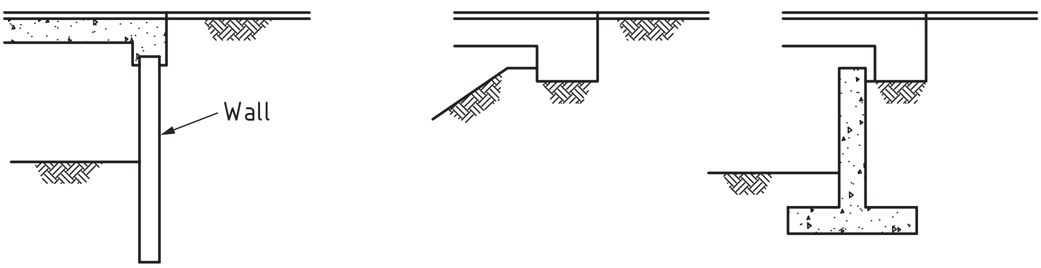
In these types of structure the vertical support to the end of the deck is provided by movement bearings seated on conventional or embedded walls, or reinforced soil abutments which do not move into the fill when the bridge expands [see Figure 4k) to Figure 4m)]. Semi-integral abutments therefore act in a similar manner to flexible support abutments in relation to the horizontal earth pressures on the end screen wall.

Figure 4 **Types of abutment for integral bridge construction**



a) b) c)

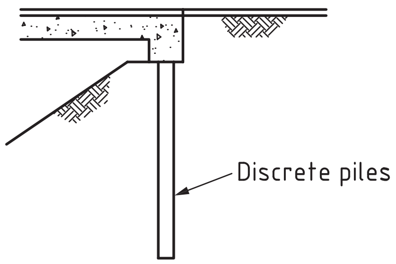
Frame abutment



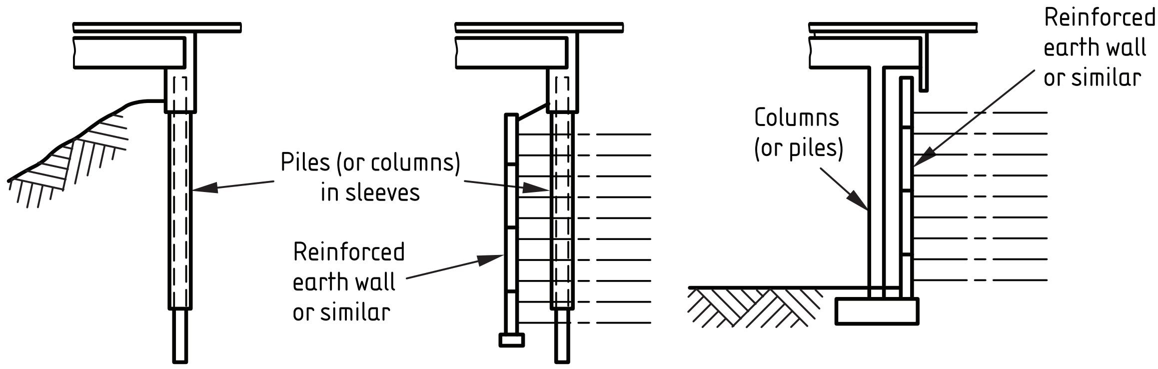
d) Embedded wall abutment

e) f) Bank pad abutments

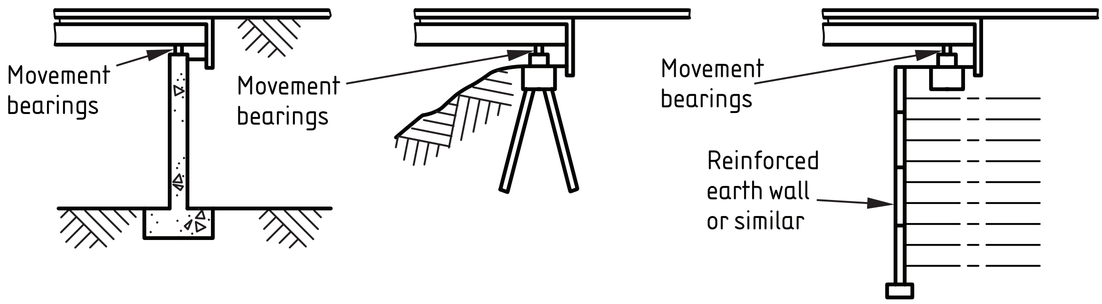
Figure 4 **Types of abutment for integral bridge construction** *(continued)*



g)
Bank pad abutments on piles



h) i) j)
Flexible support abutments



k) l) m)
Semi-integral end screen abutments

9.3.8 Replacement and inspection of bearings on semi-integral bridges

The longitudinal earth pressure on the ends of integral bridge decks are likely to be large and therefore provision should be made for the replacement of any bearings on semi-integral bridges without significant vertical movement of the deck. This can be done, for example, by providing a plinth or thick plate under the bearings which can be removed as soon as the load on the bearings is transferred to the jacks. For skewed semi-integral bridges where plan rotation is resisted by transverse forces on the bearings, provision should be made for resisting these transverse forces when the bearings are removed. Details of the proposed replacement method and jacking points provided should be given on the drawings. Adequate space should be provided around the bearings to facilitate their inspection, monitoring and replacement.

9.4 Earth pressures behind integral abutments and end screen walls

9.4.1 Values of $K_{p;t}$

The coefficient of earth pressure that develops behind integral abutments and end screen walls during expansion (K^*) is proportional to the design value of $K_{p;t}$. For unfavourable passive pressure, the value of $K_{p;t}$ may be interpolated from Table 8. The values of φ'_{triax} given in Table 8 are the design values of the triaxial φ'. In the absence of test results for φ'_{triax}, its design value may be found from the equation given in **9.10.1**.

Table 8 **Maximum (unfavourable) values of $K_{p;t}$**

$\varphi'_{triax;d}$	Values of $K_{p;t}$				
	Inclination of abutment face				
	Vertical	**Forwards**		**Backwards**	
		+10°	**+20°**	**–10°**	**–20°**
30°	4,29	3,67	3,15	5,00	5,79
35°	5,88	4,86	4,02	7,09	8,49
40°	8,38	6,65	5,28	10,51	13,06
45°	12,57	9,51	7,20	16,52	21,45
50°	20,20	14,24	10,28	28,10	38,55

NOTE Values of $K_{p;t}$ are the horizontal component of K_p with $\delta_d/\varphi' = 0,5$ calculated from the equations given in BS EN 1997-1:2004, Annex C using the design values of the triaxial φ'.

Values of K_p for favourable passive pressure for vertical walls may be taken from BS EN 1997-1:2004, Figure **C.2.1**, with $\delta_d/\varphi'_d = 0$.

9.4.2 Strain ratcheting

For integral bridges which are subject to many thermal cycles, the repeated backward and forward movement of the abutment generates pressures when the bridge is expanding which are significantly higher than those that would occur with a single thermal cycle. After many cycles, this pressure tends to a maximum value with a pressure coefficient of K^*. K^* is dependent on the total movement of the end of the deck from its maximum contraction position to its maximum expansion position. The characteristic value of the movement (d_k) is given by the equation:

$$d_k = \alpha L_x \left(T_{e;max} - T_{e;min}\right)$$

where:

α is the coefficient of thermal expansion of the deck;

L_x is the expansion length measured from the end of the bridge to the position on the deck that remains stationary when the bridge expands;

$T_{e;max}$ and $T_{e;min}$ are the characteristic maximum and minimum uniform bridge temperature components for a 50-year return period given in the UK National Annex to BS EN 1991-1-5.

d_k is not affected by the temperature at which the deck is attached to the abutments.

The design value of d may be found from the equation:

$$d_d = \frac{1}{2} d_k \left(1 + \psi \gamma_Q\right)$$

where, for the combination of actions under consideration:

γ_Q is the partial factor for thermal actions;

ψ is 1,00, ψ_0, ψ_1 or ψ_2 for thermal actions as appropriate (see BS EN 1990:2002+A1, **6.3.1**).

Where the earth pressure due to a combination of long term thermal cycling and longitudinal traffic action (braking and acceleration) is being assessed, the value of d_d given above should be increased by the sway displacement caused by the design value of the longitudinal traffic action (see also **9.5.2**).

d_d may be reduced for bridges with axially flexible decks. Where the elastic shortening under K^* pressure is significant, the reduction in d_d may be taken as the elastic shortening of the expansion length of the deck when the deck expands.

9.4.3 Horizontal pressures on abutments accommodating thermal movements by rotation and/or flexure

For full height abutments on spread footings which accommodate thermal movements by rotation and/or flexure, the design value of the earth pressure coefficient for expansion K_d^* may be calculated from the equation, but should not be taken as greater than $K_{p;t}$:

$$K_d^* = K_o + \left(\frac{C d'_d}{H}\right)^{0,6} K_{p;t}$$

where:

H is the vertical distance from ground level to the level at which the abutment is assumed to rotate; that is, the underside of the base slab for rotationally flexible foundations and the top of the base slab for rotationally rigid foundations [see Figure 5a) and Figure 5b)]. For an abutment wall that is pinned at or near its base, H is measured from ground level to the level of the pin;

d'_d is the wall deflection at a depth $H/2$ below ground level when the end of the deck expands a distance d_d as determined in accordance with **9.4.2**. d'_d may conservatively be taken as $0{,}5d_d$ for abutments with rotationally rigid foundations and $0{,}7d_d$ for abutments with pinned walls or rotationally flexible foundations;

C is 20 for foundations on flexible (unconfined) soils with $E \leq 100$ MPa;

C is 66 for foundations on rock or soils with $E \geq 1\,000$ MPa;

C may be found by linear interpolation for values of E between 100 MPa and 1 000 MPa;

$K_{p;t}$ is taken from Table 8.

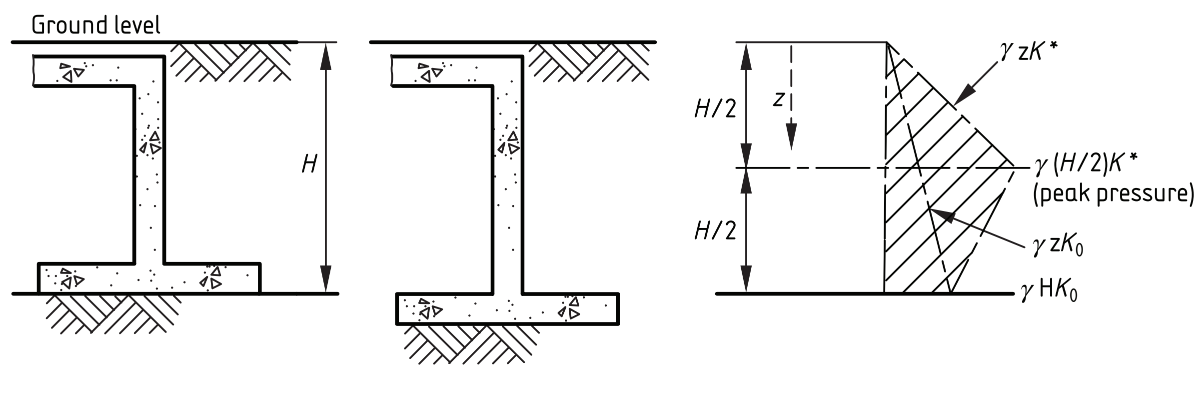
NOTE In determining C, the Young's modulus of the ground, E, may be taken as the secant drained vertical Young's modulus established at 50% of the maximum stress at failure in a standard drained triaxial compression test.

The pressure distribution on the retained face can be simulated as shown in Figure 5c); namely:

a) a triangular pressure diagram from ground level to $H/2$ based on the pressure coefficient K^*_d;

b) a trapezoidal pressure diagram between $H/2$ to H with the pressure coefficient reducing linearly from K^*_d at mid-height to K_o at depth H.

Figure 5 **Earth pressure distributions for abutments which accommodate thermal expansion by rotation and/or flexure**



a) Abutment on flexible foundations

b) Abutment on rigid foundation or hinged at base of stem

c) Characteristic earth pressure distribution for full height abutments a) and b)

9.4.4 Horizontal earth pressures on end screen and abutments that accommodate thermal movements by translation without rotation

For abutments that accommodate thermal movements by translation without rotation, such as bank pad and semi-integral end screen abutments, the earth pressure coefficient for expansion, K^*_d, may be calculated from the equation, but should not be taken as greater than $K_{p;t}$:

$$K_d^* = K_o + \left(\frac{40d'_d}{H}\right)^{0,4} K_{p;t}$$

where:

H is the height of the end screen;

d'_d is the movement of the end screen calculated at $H/2$ below ground level in accordance with **9.4.2**.

For this type of abutment, the pressure diagram may be assumed to be triangular with the design pressure at depth z equal to $\gamma z K_d^* \gamma_G$.

9.4.5 Horizontal earth pressures on full height frame abutments on piles and embedded wall abutments (soil–structure interaction analysis)

9.4.5.1 Abutments with granular soils

For full height integral abutments founded on a single row of vertical piles and integral embedded wall abutments, the horizontal earth pressure at various depths below ground level should be found using a soil–structure interaction analysis which takes account of:

- the non-linear response of the soil to deck expansion and contraction;
- the effect of strain ratcheting on soil properties, which may be based on 120 cycles with an amplitude of d_k;
- variations of soil properties at different depths;
- the degree of compaction of the soil;
- the rotational and axial stiffness of the deck;
- horizontal soil arching between the piles;
- the staged application of the thermal effects;
- the early and later life of the structure; and
- the envelope of possible combinations of minimum earth pressures with maximum expansion and maximum earth pressures with minimum expansion (see **9.4.8**).

The accuracy of any soil–structure interaction software and numerical model used in the analysis should be demonstrated by calibration against comparable laboratory and/or field monitoring to demonstrate compatibility in deflection and soil pressures down the depth of the abutment after 120 thermal cycles between $T_{e;min}$ and $T_{e;max}$.

NOTE An example of a soil–structure interaction analysis is given in Annex A.

9.4.5.2 Abutments with cohesive soils

For a piled or embedded integral abutment with cohesive soils, the effects of strain ratcheting in the cohesive soils may be ignored and the pressure on the wall and piles when the end of the deck expands, calculated using a conventional soil–structure interaction analysis and an appropriate value of E for the soil (see **A.3**).

9.4.6 Horizontal earth pressures on bank pad abutments founded on a single row of embedded piles

A bank pad abutment supported on a single row of piles [see Figure 4g)] may be designed as a piled abutment in accordance with **9.4.5**. Account should be taken in the design of any inclined slope in front of the piles. This may be done by using software that can accommodate a berm, and/or by taking the value of K_a and K_p on the front face of the piles as that applicable to the inclined slope (see also Clause 7.2 in CIRIA Report C580 [11]). The effect of horizontal soil arching between the piles should also be assessed.

However, unless it can be shown by a rigorous analysis that the strains across the whole width of fill behind the piles are not significantly different from the strains that would occur with a continuous embedded wall, the horizontal pressure behind the end screen itself should be calculated as for a bank pad abutment supported on fill, as in **9.4.4**.

9.4.7 ψ factors and partial factors for earth pressures behind integral abutments

The vertical earth pressure behind an integral abutment due to the weight of soil is considered to be a permanent action and is therefore factored by γ_G (where γ_G is the partial factor for the weight of soil). Because it is a permanent action, no ψ factor is applied to it in accordance with BS EN 1990.

Horizontal earth pressures applied to integral abutments are dependent on the thermal expansion of the bridge deck, with this thermal expansion determined by applying relevant values of ψ and γ_Q. Account of this may be taken by using the values of d_d specified in **9.4.2** when calculating the design value of the earth pressure coefficient, K_d^*.

From this, it follows that the design value of the horizontal earth pressure at depth z when K_d^* governs is $\gamma z K_d^* \gamma_G$.

For γ_M, the National Annex to BS EN 1997-1 states that the value of γ_M should be taken as either the value given in the tables in the National Annex or the reciprocal of that value (denoted γ_M^*), whichever results in the more onerous effects. The superior (upper) value of tan φ'_K should therefore be divided by γ_M^* for unfavourable passive pressures and favourable active and at rest pressures, and the inferior (lower) value of tan φ'_k divided by γ_M for favourable passive pressures and unfavourable active and at rest pressures.

*NOTE The use of γ_M^* is required because BS EN 1997-1:2004, **2.4.6.2** states that geotechnical parameters are always divided by γ_M. Dividing a geotechnical parameter by γ_M^* is numerically equivalent to multiplying it by γ_M. Therefore, the approach in the National Annex to BS EN 1997-1 effectively requires geotechnical parameters to be divided by or multiplied by γ_M, whichever results in the more onerous effects.*

9.4.8 Pressure envelope

The earth pressure on the retained face of an integral abutment is dependent on the following:

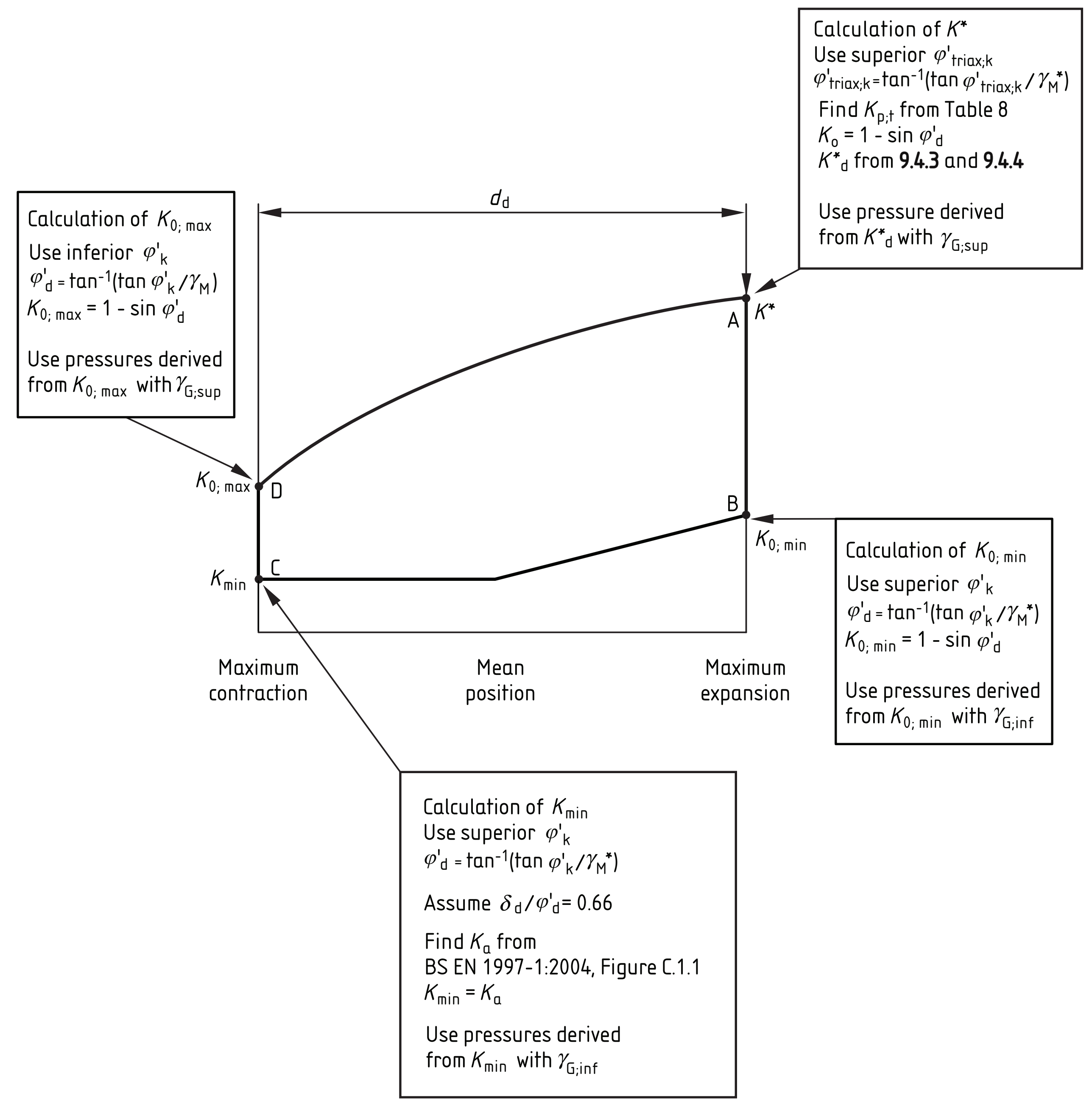
1) the thermal movement range based on a 50-year return period (see **9.4.2**);
2) the direction of movement (expanding or contracting); and
3) the actual amount of expansion or contraction for the combination of actions or design situation under consideration.

The design values of the movements to be identified in the relevant design situation are given in **9.4.2**.

In some circumstances, minimum earth pressures are more unfavourable than maximum earth pressures, so both expansion and contraction have to be assessed.

Figure 6 gives an envelope of the pressure coefficients that should be used with expansion and contraction for limit equilibrium calculations.

Figure 6 **Pressure coefficient envelope**



*NOTE 1 Figure ABCD shows the maximum and minimum pressure coefficients to be considered with expansion and contraction when the design value of the thermal movement is d_d as described in **9.4.2** and **9.4.7**.*

*NOTE 2 The superior characteristic and inferior characteristic values of φ′ represent the limiting values between which the shear strength of the backfill have to lie (see **9.10**).*

NOTE 3 $\gamma_{G;inf}$ and $\gamma_{G;sup}$ are the favourable and unfavourable values of γ_G for the weight of soil.

*NOTE 4 γ_M^* is the reciprocal value of γ_M given in the tables in the UK National Annex to BS EN 1997-1 (see **9.4.7**).*

9.5 Longitudinal loads

9.5.1 Traffic surcharge loading

Traffic surcharge loading need not be applied in conjunction with *K** pressure. However, it should be applied to one abutment in conjunction with active pressure when the structure is being designed for longitudinal load as described in **9.5.2**.

If the model given in **7.6.2** is used for the design of semi-integral bridges, the surcharge line load (*F*) need not be applied to the abutment wall if the end screen is greater than 2 m in depth. If the end screen is less than 2 m in depth, the surcharge line load (*F*) applied to the abutment wall should be treated as for buried structures, see Table 7, Note B.

9.5.2 Resistance to longitudinal loads

Where earth pressure behind an abutment wall is used to resist variable longitudinal loads such as braking, acceleration and longitudinal wind, it is important to estimate the associated abutment displacement that would occur early in the life of the bridge before strain ratcheting of the backfill has occurred. Methods of estimating the movement required to mobilize a given proportion of passive resistance are given in **7.5**.

As illustrated by the range of movements given in BS EN 1997-1:2004, Table C.2, the movements required to generate a specific proportion of passive resistance cannot be accurately predicted. It is therefore important, especially where part of the longitudinal force is resisted by earth pressure and part by flexure of the abutments and piers, to be conservative regarding the displacements needed to mobilize a specific proportion of passive resistance. Lower-bound values of the proportion of passive pressure, ignoring wall friction, should therefore be used in determining the resistance to longitudinal loads.

9.6 Thermal distortions

Although *K** pressures are unaffected by the temperature at which the deck is attached to the abutment, the thermal distortions caused by expansion and contraction are dependent upon the temperature at which the deck is restrained. The design value of the displacement for calculating thermal distortions should be based on a temperature increase of $(T_{e;max} - T_{0;min})\ \psi\gamma_Q$ for expansion and $(T_{0;max} - T_{e;min})\ \psi\gamma_Q$ for contraction where $T_{0;max}$ and $T_{0;min}$ are the upper-bound and lower-bound values of T_0 defined in the UK National Annex to BS EN 1991-1-5 and γ_Q and ψ are as defined in **9.4.2**.

9.7 Foundations

9.7.1 General

Foundations for integral bridges should be designed in accordance with BS EN 1997-1 and Clause **5** and Clause **6** of this document.

9.7.2 Spread footings for full height frame abutments

Spread footings of full height frame abutments can tend to slide forward as a result of deck contraction or *K** pressures when the deck expands. If the sliding resistance of the base is not large enough to resist this tendency, the base slab slides forwards and the hogging moments at the top of the wall increase until equilibrium is achieved. Although the sliding movement can tend to reduce the earth pressure behind the wall, the movement is likely to be irreversible and, with time, K* pressures build up again. Unless the base slab is proportioned to provide full sliding resistance at SLS, the structure should be designed at SLS for the combined effects of K* pressures and the effects of the forward displacement of the base slab.

At ULS the situation is different because, provided ductile bending failure occurs before brittle shear failure (see **7.4**), forward sliding of the foundation would be accompanied by the development of plastic hinges in the wall with no loss of capacity, and the K* pressures would reduce to active pressure. Load cases for the design of a ductile structure at ULS should therefore include the following load cases.

- Full K* pressure (and other effects) assuming the base slab is fully restrained against sliding.
- Active pressure and base friction with the abutment wall assumed to be cantilevered from the deck.

The shear forces in the abutment wall should be assumed to be those arising from full K* pressures minus the friction force on the base.

Checks should also be made to ascertain whether backward sliding of a base of a frame abutment could occur as a result of horizontal earth pressure due to traffic surcharge on the remote abutment in conjunction with variable longitudinal load and/or thermal expansion, especially early in the life of the bridge. Account should be taken of the effects of any such displacement of the base, both at SLS and at ULS and the possibility that the base remains in the displaced position after the disturbing forces are removed.

9.7.3 Spread footings for bank pad abutments

Foundations which slide while supporting vertical loads are prone to greater settlements than those that do not. This effect should be taken into account in the design. This may be done by limiting the SLS pressure applied to the ground to half the value that would be acceptable in verifying settlement for a foundation that did not slide.

9.7.4 Piled foundations

Pile foundations for full height frame abutments should normally consist of one or more rows of vertical piles, but for abutments that do not rely on the sway of their foundations to accommodate thermal movements, raked piles may be used, provided the pile/pile cap configuration does not form a mechanism if the piles are considered to be pinned top and bottom.

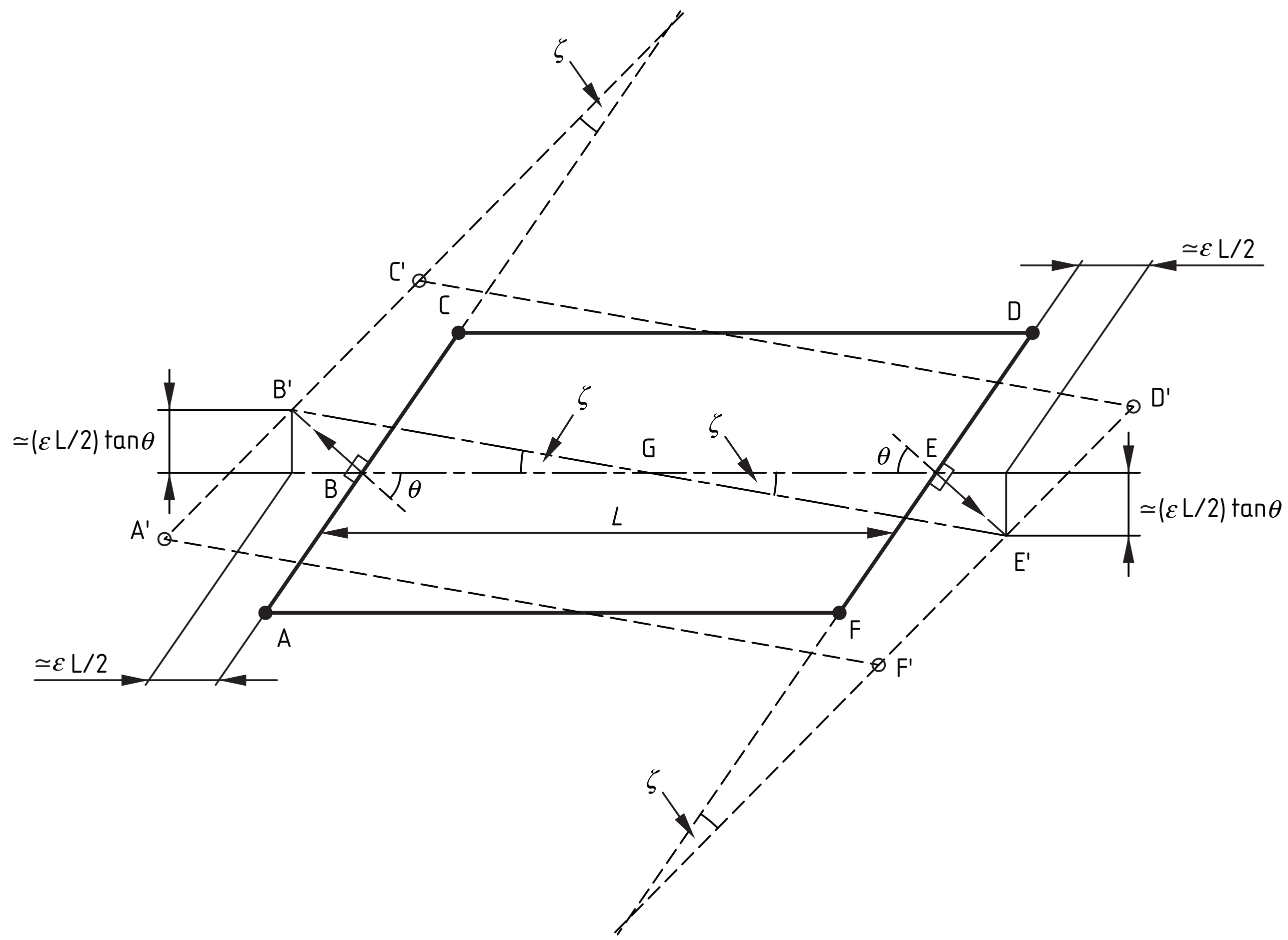
9.8 Skew effects

9.8.1 Twisting of frame abutment and integral piers due to expansion and contraction of deck

For skewed integral bridges which accommodate expansion and contraction by flexing or rocking of the abutments about axes parallel to their planes, longitudinal thermal movements are accompanied by a plan rotation of the deck and twisting of the tops of the abutments and piers relative to their bases. This effect should be assessed in the design and accounted for.

As illustrated in Figure 7, the angle of twist, ζ, is approximately equal to $\varepsilon \tan \theta$, where ε is the thermal strain and θ is the skew angle. The angle of twist, ζ, is the same for all piers and abutments and is independent of the length of the bridge.

Figure 7 **Twisting of skewed structure**



NOTE The figure shows a plan view of the deck of a skewed portal frame where the abutments accommodate expansion and contraction by flexing about axes parallel to AC and DF and twisting about a vertical axis.

When the centreline of the deck BE expands, B has to move perpendicular to AC, to B′, and E has to move perpendicular to FD, to E′. BE therefore twists through an angle ζ (equal to angle BGB′ and EGE′).

As the deck is effectively rigid in plan, the whole deck also twists through an angle ζ, and as the tops of the abutments AC and DF are integral with the deck, they also have to twist through an angle ζ. This twisting movement is superimposed on the bending displacement.

9.8.2 Plan rotation of deck due to earth pressure and distortional effects

In skewed integral bridges, the earth pressure is applied normal to the abutment walls so that the earth pressure actions on the two abutments are not collinear and will cause a couple. This tends to cause the bridge to rotate in plan and the tendency to rotate usually has to be resisted at the abutments.

For skewed bank pad abutments, which accommodate expansion by sliding over foundations, transverse friction on the base should be ignored when calculating the resistance to rotation. This is because this force cannot be mobilized without incurring a significant rotational displacement which would increase every time the bridge expanded or contracted.

9.9 Wing walls

9.9.1 Geometrical details

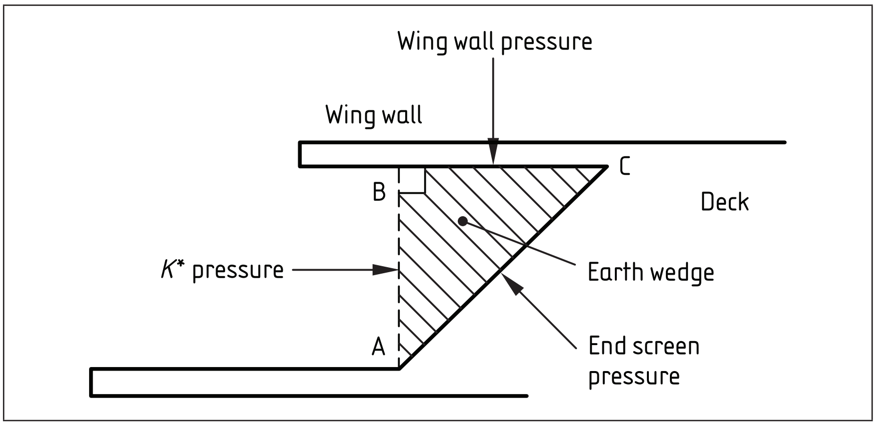
Wing walls for frame abutments should normally be designed to be structurally independent of the abutments.

9.9.2 Wing walls supporting K^* earth pressures

Wing walls which provide lateral restraint to backfill that is subject to strain ratcheting are themselves subject to enhanced earth pressures. Such effects should be taken into account in the design. This can be done by using a pressure coefficient of K_a multiplied by K^*, but not less than K_o, in their design.

Additionally, the pressures on a wing wall forming an acute corner with the end screen of a skewed abutment, as shown in Figure 8, should be determined considering the equilibrium of the earth wedge ABC with K^* pressure applied to AB.

Figure 8 **Equilibrium of horizontal earth wedge behind skew abutment**



9.10 Backfill

9.10.1 Backfill material

The design methods given in this document assume that the backfill material for integral bridges is free draining granular material with properties and grading conforming to Classes 6N or 6P, specified, installed and compacted in accordance with the *MCHW* [8] or as given in the project specification or as otherwise agreed with the Overseeing Organization.

It is important that the backfill is neither too weak nor too strong. The design should be based on a range of soil strengths and the upper (superior characteristic) and lower (inferior characteristic) values assumed should be stated in the contract. Verification to ensure that the backfill provided on site lies within the strength limits assumed in the design should be carried out.

To achieve accurate predictions of soil pressure, it is essential that the soil parameter $K_{p;t}$ is derived from appropriate long term $\varphi'_{\text{max triaxial}}$ values that reflect the long term weight density and mean effective stresses in the soil.

In the absence of triaxial tests, where plane strain testing or shear box testing has been carried out, the equivalent $\varphi'_{\text{max triaxial}}$ may be obtained using the following equation, see Bolton [4]:

$$\varphi'_{\text{max triaxial}} = 0,6\varphi'_{\text{max plane strain}} + 0,4\varphi'_{\text{cv}}$$

9.10.2 Backfill wedge

The underside of the zone of backfill behind an integral abutment should slope up from the bottom of the abutment at an angle of inclination not exceeding 45°.

9.10.3 Rock cuttings

In rock cuttings where there is no backfill wedge and the abutment is cast directly against the rock, it should be assumed that all thermal expansion is prevented by the rock, and the structure should be designed for the resulting pressures and deck forces, unless the stiffness of the rock is established at the design stage.

9.10.4 Run-on slabs

Run-on slabs should normally be avoided because they create two road joints rather then one and might rock if not properly bedded. Where it is considered necessary to install a run-on slab to limit settlement adjacent to the structure, a buried run-on slab below the level of the road construction should be installed.

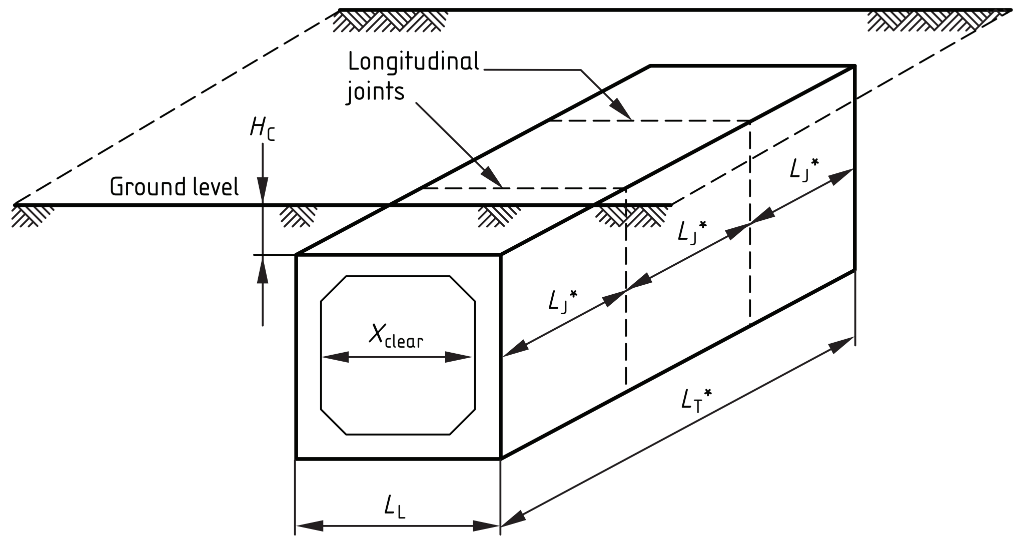
10 Buried concrete structures

10.1 General

This clause provides non-contradictory complementary information which may be used in the design of buried concrete structures. The symbols used for a typical buried box structure are shown in Figure 9.

The recommendations given in this clause relate to concrete boxes and portal frame structures for which the overall longitudinal length (L_L) (**3.1.9**) does not exceed 15 m in length and the depth of earth cover (H_c) (**3.1.2**) does not exceed 11 m.

Figure 9 **Symbols for typical buried box structure**



10.2 Actions applied to buried concrete structures

10.2.1 Design principles for actions

A buried concrete structure with a depth of earth cover (H_c) less than 0,6 m should be treated as a normal bridge structure and designed for traffic and thermal actions specified in BS EN 1991-2 and BS EN 1991-1-5 together with the earth pressures described in BS EN 1997-1 and this document.

A buried concrete structure with more than 0,6 m of earth cover may be designed as described in the previous paragraph except that:

a) the vertical traffic actions may be considered to be dispersed through the fill as described in **10.2.7**;

b) the braking and acceleration forces applied to the structures may be reduced as described in **10.2.8.2**; and

c) the thermal effects may be modified as described in the National Annex to BS EN 1991-1-5 and **10.2.11** and **10.2.12**.

10.2.2 Superimposed permanent load

The effect of superimposed permanent load consisting of the weight of the earth cover and the road construction materials above the structure can be influenced by positive and negative arching action.

The possible effects of positive arching reducing this load should be ignored.

Where consolidation or settlement of the fill adjacent to a buried structure causes negative arching of the fill above the roof, increased loading will be generated on the roof slab. These downdrag effects can be greater if the structure is founded on hard material or piles so that the backfill settles more than the structure. In the absence of a more rigorous analysis, the effects of differential settlement between the structure and the adjacent ground should be taken into account by applying a supplementary model factor $\gamma_{Sd;ec}$ to the effects of the weight of the earth cover and the road construction materials where:

$\gamma_{Sd;ec}$ = 1,5 + 0,5(H_c – 8)/3 but not less than 1,5 for structural foundations on hard material or piles;

$\gamma_{Sd;ec}$ = 1,15 + 0,35(H_c – 8)/3 but not less than 1,15 for other foundation material.

10.2.3 Horizontal earth pressure

The effects of strain ratcheting, wall friction and thermal expansion and contraction should be incorporated into calculating the horizontal earth pressures applied to the walls of buried structures.

NOTE A method to calculate horizontal earth pressures applied to the walls of buried structures that are backfilled with 6N or 6P fill in accordance with the MCHW [8], which may be used where there is uniformity of ground conditions, is given in Annex B. This method gives directly determined values of earth pressure coefficients that take into account the effects of strain ratcheting, wall friction and thermal expansion and contraction.

10.2.4 Vertical road traffic actions

Vertical traffic actions, footway and cycle track actions and accidental traffic loading on edge members should be applied in accordance with BS EN 1991-2 and may be dispersed through the fill in accordance with the principles described in **10.2.7**.

10.2.5 Loading on central reservations

On dual carriageways, the portion of structure supporting the central reservation should be designed for full carriageway loading.

10.2.6 Construction traffic

Under the low earth cover conditions which prevail during construction, the structure can be subjected to load conditions that are more severe than those experienced in normal service. During the design stage therefore, the type of construction traffic likely to be relevant at different stages should be identified and details of the traffic load capacities of the structure under various depths of earth cover should be recorded on the drawings to ensure that these are not exceeded during construction.

10.2.7 Dispersal of loads through the fill

In the absence of a more rigorous analysis, a wheel load or other vertical load may be dispersed transversely and longitudinally through the fill at an angle of 30° to the vertical as shown in Figure 10a).

Where the dispersal zone of a wheel load is curtailed by a wing wall or similar, the pressure distribution may be based on the assumption that the wheel load is transversely dispersed over the curtailed dispersal zone as shown in Figure 10b).

Where the dispersion zones of different wheels overlap, the following assumptions may be made.

a) For in-situ structures which are to be analysed using a three-dimensional (3-D) model, the dispersed patch loads may be superimposed.

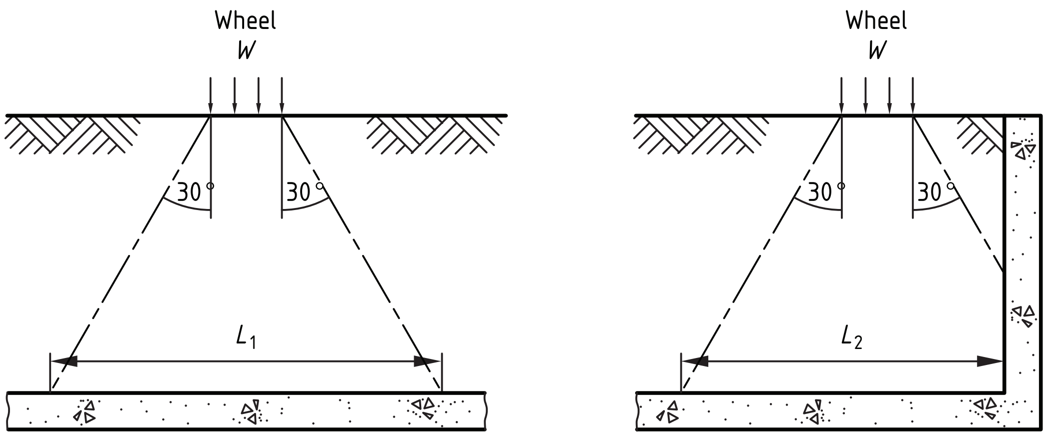
b) For in-situ structures which are to be analysed using a two-dimensional (2-D) model or on a metre-strip analysis basis, the load per metre width in the transverse direction may be based on the total load on the most heavily loaded metre-strip as shown in Figure 11.

c) For segmental units, the total load on the most heavily loaded segment may also be determined as in Figure 11, but where the earth cover is small, the effect of applying concentrated wheel loads close to the edge of a segment should also be taken into account.

d) A uniformly distributed traffic load may be considered to be dispersed at 30° to the vertical from each edge. Where the dispersal zone is curtailed by a wing wall or similar, the load may be considered to be dispersed over the curtailed dispersal zone.

e) Where the transverse moments in an in-situ structure are not analysed using a 3-D computer model or the "Pucher Charts" in *Influence surfaces of elastic plates* [19] or similar, or where the width of a segmental unit (L_j) is more than twice as wide as the depth of earth cover (H_c), the transverse bending moment per metre may be taken as one half of the longitudinal moment per metre due to the vertical traffic load (see also **4.4**).

Figure 10 **Transverse load dispersion**

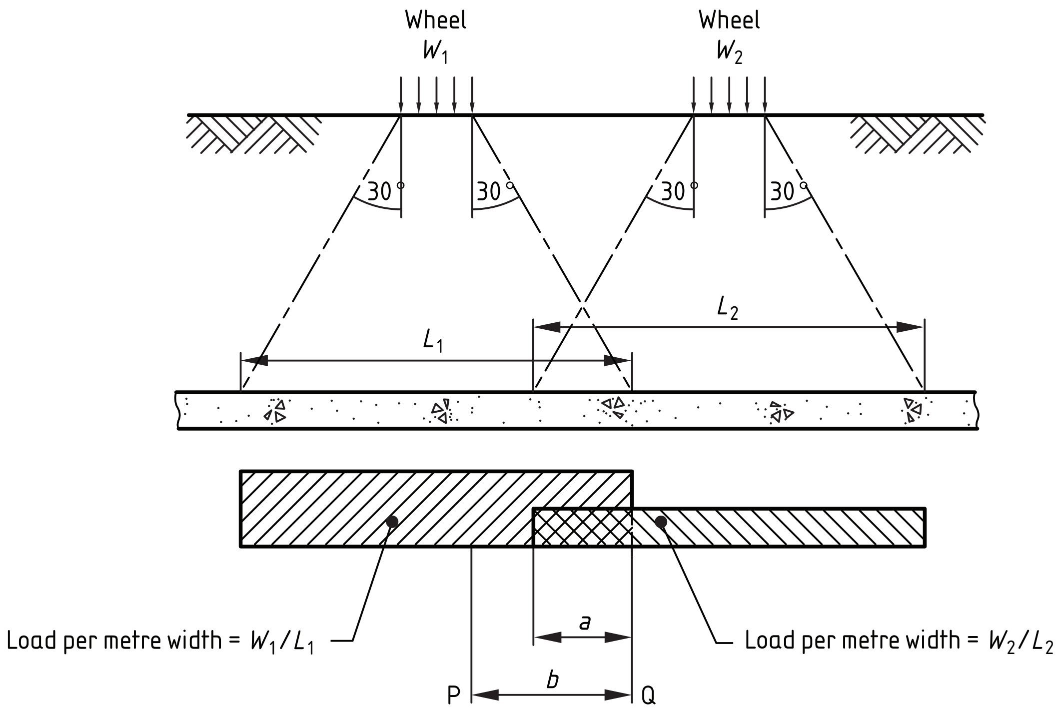


a) Wheel load remote from wing wall

b) Wheel load close to wing wall

NOTE In a), the load is dispersed transversely and longitudinally over a length L_1. In b), the transverse dispersal length is L_2 (curtailed by the wing wall). The longitudinal dispersal length equals L_1 unless curtailed by a transverse wall or similar.

Figure 11 **Transverse load/metre where two dispersion zones overlap**



NOTE 1 PQ in the figure is the most heavily loaded strip of width b. For a two-wheel overlap, the load on the strip can be taken as: $bW_1/L_1 + aW_2/L_2$ where W_1 is the larger wheel load and a is the length overlap. For a metre-width design, b is to be taken as 1 m. For narrow segmental units b may be taken as the segment width (L_j).

NOTE 2 When the dispersal zones of more than two wheels overlap, the local pressure may be based on the most heavily loaded segment or metre-strip, as appropriate.

10.2.8 Longitudinal road traffic actions

10.2.8.1 Traffic surcharge

The areas of carriageway on the backfill beyond the ends of the structure should be loaded with the traffic surcharge load specified in the UK National Annex to BS EN 1991-2. The resulting horizontal pressures on the abutments can be calculated in accordance with **7.6** and Table 7, Note B.

10.2.8.2 Braking and acceleration forces

For buried structures with less than 0,6 m depth of earth cover (H_c), the full value of the braking or acceleration action described in BS EN 1991-2:2003, **4.4.1**, should be applied. Where the structure is skew to the direction of traffic, the combined effect of the longitudinal and transverse components of the horizontal actions should be assessed.

For buried structures where the depth of earth cover (H_c) is greater than 0,6 m and less than the overall length of the structure (L_L), the horizontal force may be assumed to reduce linearly to zero, as H_c increases from 0,6 m to L_L. The reduction factor, η, to be applied to the horizontal force in this range is therefore given by $\eta = (L_L - H_c)/(L_L - 0{,}6)$.

For a buried structure with $H_c > L_L$, the effects of braking and acceleration on the structure may be ignored.

The braking or acceleration force should be applied directly to the top of the roof of the structure over a notional lane width without lateral dispersion. However, the in-plane rigidity of the roof and walls of in-situ box and portal structures provides a robust resistance to distortional warping, and provided account is taken of the tendency for plan rotation, the effects of the braking and acceleration forces may be assumed to be distributed structurally over a length of L_j.

The total braking or acceleration force applied to the top of the roof of a buried structure need not be taken as greater than the friction force that can be generated between the earth and the roof, taking the weight of the vehicle into account.

10.2.9 Centrifugal and other transverse forces

Precautions should be taken to prevent the separation of narrow precast units at the edge of a segmental structure under the effects of centrifugal force, skew braking and skidding, unless the structure is provided with an independently supported headwall that can resist the transverse load.

10.2.10 Parapet impact

Where a vehicle restraint system is attached to or seated on a precast segment, the design should prevent the unit supporting the restraint system separating from the rest of the structure in the event of a parapet collision.

10.2.11 Thermal expansion and contraction

The effects of thermal expansion and contraction in buried concrete structures should be identified in accordance with BS EN 1991-1-5, unless the overall length of the structure L_L is less than 3 m, in which case such effects may be ignored.

The effect of the thermal expansion and contraction of the roof slab, ignoring any thermal movements of the base of the structure, should be accounted for in the analysis of the structure. Interaction between the backfill and structure due to thermal effects may be ignored if the directly determined earth pressure coefficients in Annex B of this document are used, because any increase in earth pressure due to expansion is deemed to be covered by the specified K_{max} pressure coefficient.

10.2.12 Temperature difference

The effects of the temperature difference specified in BS EN 1991-1-5 should be assessed. Temperature gradients within the walls and base slabs of buried structures may be ignored, but the effect on these members of roof flexure induced by temperature difference should be accounted for in the design.

10.3 Design of foundations

10.3.1 Rotational stiffness of foundations

The bearing capacity of buried structures (particularly buried box structures) is not usually critical and site investigation is therefore often limited. In the absence of detailed testing and a rigorous soil–structure interaction analysis, the following procedures may be used.

a) Where a box or portal structure is founded on hard material (**3.1.5**), the footings may be assumed to be rotationally rigid.

b) Where a box structure is founded on softer material, it may be assumed that the pressure across the base slab is trapezoidal.

c) Where a portal frame is founded on material other than hard material, it may be assumed that the base slab is rotationally flexible and the portal frame can therefore be considered to be pinned at the base of the legs.

d) For portal frames, where the properties of the foundations are not known, or where the structures or elements of the structure are particularly sensitive to the rotational stiffness of the base, an analysis of both the rigid and flexible (fixed and pinned) conditions should be carried out and the more onerous effects used in the design.

10.3.2 Bearing pressure

For verifying the bearing pressure under the base slab of a buried structure which is subject to braking or acceleration forces and traffic surcharge, it may initially be assumed that the pressure on the passive wall is the K_{max} pressure as shown in Table B.4. If this loading results in excessive bearing pressure, the assumed pressure on the passive wall may be increased, provided that the component members of the structures are designed for this increased pressure and the rotations required to mobilize this pressure are acceptable at the relevant limit states.

Rotations required to mobilize a proportion of passive pressure can be found in BS EN 1997-1:2004, Table C.2 (case a), see also **7.5**.

10.3.3 Sliding and overturning

Where sliding or overturning of a box or portal structure is initiated by braking or acceleration as shown in Table B.6, the earth pressure on the passive face may initially be based on the pressure coefficient K_{max}. If, however, this pressure is insufficient to prevent sliding or overturning the earth pressure on the passive face may be increased until equilibrium is achieved, provided that the component members of the structures are designed for this increased pressure and the movements required to mobilize this pressure are acceptable at the relevant limit states.

Sliding displacements required to mobilize a proportion of passive pressure can be found in BS EN 1997-1:2004, Table C.2 (case b), see also **7.5**.

10.3.4 Sliding of portal legs

Under symmetrical earth pressure loading, both legs of a portal structure tend to slide inwards. If this tendency can be fully resisted by friction under the base slabs, then the portal can be considered to be restrained against horizontal movement at the base of the legs. If, however, the tendency to slide cannot be fully resisted by base friction, the legs flex inwards until equilibrium is achieved, producing additional flexural moments and shears in the structure which should be assessed in the structural design. If this leg displacement is likely to be irreversible, the consequences of the legs being permanently displaced inwards should be identified and accounted for in all relevant design situations.

10.4 Skew

10.4.1 Analysis methods

Skewed in-situ structures which are long (transversely) relative to their span may be designed on the basis of either the square or the skew span. For structures designed on the basis of the square span, however, structural elements within a width of $X_{clear}\sin\theta$ from the edge of the structure should be designed on the basis of the skew span.

Alternatively, skewed in-situ structures may be analysed by more rigorous methods such as a 3-D computer analysis.

10.4.2 Effect of offset pressures

With skewed in-situ structures, the line of thrust of the horizontal earth pressure forces on one abutment is offset laterally from the line of thrust of the earth pressure forces on the opposite abutment. This results in a plan torque which for box structures is resisted by torsional friction on the base, and, if this is insufficient (and the skew is not too great), by a build up of passive resistance on the abutment walls towards the obtuse corners of the structure. For portal structures, see also **9.8**.

If the line of thrust of the earth pressure forces on one wall passes within the middle third of the other wall (that is, if $L_L\tan\theta < L_j/6$), this plan twisting effect may be ignored.

If $L_L\tan\theta \geq L_j/6$ and the applied torque is greater than the rotational frictional resistance of the base, the unbalanced torque may be resisted by increasing the horizontal earth pressure on the walls towards the obtuse corner provided the structural elements are designed to resist the increased pressure.

The value of passive pressure that can be mobilized near the obtuse corners should be limited by the value of movement and plan rotation that is acceptable at the relevant limit state (see **7.5**), but it should not exceed the lower-bound value of passive pressure based on zero wall friction.

On skewed structures where the line of thrust from one abutment passes close to, or outside, the obtuse corner of the opposite abutment, passive pressure might tend to increase, rather than to resist the tendency of the structure to rotate in plan, and the danger of failure due to rotational sliding should be carefully examined.

10.4.3 Skewed precast units

Skewed precast units should generally be avoided because they can be very sensitive to plan rotations.

10.5 Longitudinal joints

Segmental structures should be designed to accommodate the movements that might result from differential settlement of the foundation and differential deflection between units due to traffic.

To prevent the occurrence of excessive movements at longitudinal joints in structures with low earth cover, the net vertical midspan deflection of a segmental unit under traffic loading at SLS under the characteristic combination of actions should be limited to $0{,}15H_c$. This deflection should include the short-term elastic settlement of the foundation due to traffic loading.

These recommendations also apply to longitudinal joints in in-situ structures unless these joints are designed to transmit the effects of the permanent loads and traffic actions.

10.6 Stages to be analysed

Three stages should be analysed in the design:

a) the completed structure backfilled up to the top of the roof;

b) the structure backfilled to an intermediate level between roof level and finished surface level, at which it is proposed to use the structure for construction traffic; and

c) the structure, fully backfilled, in service.

Each element of the structure should be designed for stages a) and b) accounting for the following:

1) permanent loads with maximum or minimum superimposed dead loads (see **10.2.2**) (excluding differential settlement);

2) relevant traffic or construction loads (see **10.2.6**);

3) maximum or minimum horizontal earth pressures (see for example Table B.1 to Table B.6); and

4) thermal actions.

For the in-service stage c), the structure should be designed for the load cases illustrated diagrammatically in Table B.1 to Table B.6, in conjunction with the requirements in the relevant parts of BS EN 1991 and BS EN 1990.

Annex A (informative) Method for designing integral abutments using a soil–structure interaction analysis

A.1 General

This annex gives a method for designing integral abutments using a soil–structure interaction analysis. It has been calibrated against a number of experimental results, given in England, et al. [13], Hambly, et al. [14], Springman, et al. [15], Tapper, et al. [16] and Tan, et al. [17], and satisfies the recommendations of **9.4.5**. The method offers a way of determining the pressures behind embedded wall abutments, frame abutments founded on a single row of piles and other structures where the depth of soil affected by the repeated deck expansion and contraction cannot readily be determined. It may also be used as an alternative to the limit equilibrium methods for frame abutments on spread footings described in **9.4.3**.

*NOTE The phrase "needs to" is used in this annex to express recommendations applicable to the method presented in this annex. Other soil–structure interaction analysis methods may be used provided they satisfy the requirements of **9.2.2** and **9.4.5**.*

The soil–structure interaction analysis needs to be carried out using a numerical model with structure and soil properties satisfying the recommendations of **A.2** and **A.3**. The analysis approach needs to be capable of:

- generating the soil stiffnesses and quasi-passive resistance of the soils as a function of the soil strain, and modelling the variation of soil stiffness with depth (see **A.3**);
- accounting for the staged and repeated application of deck expansion and contraction (see **A.4**); and
- taking account of vertical and horizontal soil arching effects.

A.2 Structural properties used in model

The soil–structure interaction model needs to include all vertical elements of the abutments and piles explicitly. The rotational and lateral restraint provided by the deck needs to be modelled. This may be done explicitly or by using elastic horizontal and rotational springs. The stiffness of the horizontal spring, S_H, may be taken as AE/L_x where A and E are the area and elastic modulus of the deck respectively.

A.3 Soil properties used in model

A.3.1 General

The soil needs to be modelled as an elastic continuum (see **A.3.2**) with earth pressures restricted to lie between active pressure and the quasi-passive limit (see **A.3.3**). The variation in soil properties with depth needs to be modelled over the depth H' (see **A.4**).

A.3.2 The elastic modulus of the soil

The relationship between the soil stiffness, which varies with depth, and the soil strain needs to either be derived directly from triaxial testing of the soils, or by using published data on small strain stiffness values (for example Seed and Idriss [20]). A method of deriving suitable short term stiffness values for granular soils based on the findings of Seed and Idriss [20] is as follows.

The depth of soil behind and in front of the abutment with properties influenced by the abutment movement needs to be modelled as a series of thin strips to which design values of weight density (γ), angle of shearing resistance (φ'_d) and elastic modulus (E_d) are assigned. The value of E for granular soils is derived as a function of the applied soil strain (d'_d/H') and the mean effective stress of the soil (σ_m). The value of E at a depth z below ground level is calculated as follows:

$$\sigma_v = \gamma z - u$$

$$\sigma_m = \sigma_v \frac{(1+2K_o)}{3}$$

$R_{F,G}$ is taken from Figure A.1 for the value of d'_d/H' given in **A.4**;

G = 220 $R_{F,G}$ $\sqrt{\sigma_m}$ (where G and σ_m are both in kPa);

E = $2G (1 + \nu)$

where:

σ_v is the vertical stress;

σ_m is the mean stress;

u is the pore pressure;

G is the shear modulus of the soil;

υ is Poisson's ratio (which may be taken as 0,3);

$R_{F,G}$ depends on the average rotational strain d'_d/H' (see Figure A.1);

H' is the depth of soil influenced by the abutment movement (see **A.4**);

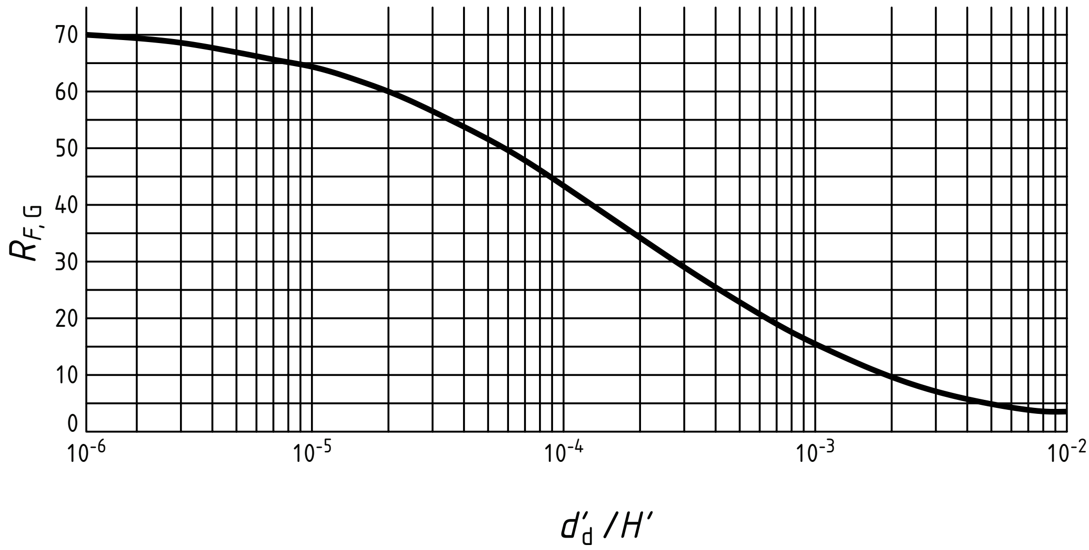
d'_d is the movement of the structure at a depth $H'/2$ due to an expansion or contraction of the end of the deck (see **A.4**).

For granular soils, account needs to be taken of the repeated thermal cycling. In the absence of a more detailed analysis, this may be addressed by assuming densification to at least 90% (see Figure A.1) and by multiplying the soil stiffness values by a factor of 1,5 in line with the findings of Clayton, Xu and Bloodworth [21]. In this case, the design value of E (E_d) is given by:

$$E_d = 1{,}5E$$

For cohesive fills, reference may be made to Koutsoftas and Fischer [22] and the effects of strain ratcheting may be assumed to be negligible.

Figure A.1 **Variation in soil shear modulus factor, $R_{F,G}$, with d'_d/H' assuming densification to 90%**



NOTE These variations have been derived from Seed and Idriss [20].

A.3.3 The quasi-passive limit

For granular soils, the quasi-passive limiting pressure coefficient K_d^* on the back and front of the abutments may be derived from the equation in **9.4.3**, taking $H = H'$ and with d'_d and H' for the front and back of the abutments derived in accordance with **A.4**.

For cohesive soils, reference needs to be made to published data or testing of the soils. For such soils, the effects of strain ratcheting may be assumed to be negligible.

A.4 Soil strain

A.4.1 General

The properties of the soil used in the soil–structure interaction model depend on the average rotational strain in the soil d'_d/H'. The values of d'_d/H', and therefore the soil properties, differ for the soil behind and in front of the abutment. Typically, an iterative approach is required to determine H' and d'_d.

The soil behaviour under cyclic expansion and contraction exhibits hysteresis and therefore the results of the soil–structure interaction analysis are affected by the sequence of expansion and contraction. Such sensitivity needs to be taken into account in the analysis. This may be done by applying an initial contraction giving a movement of $d_d/2$ at the end of the deck followed by an expansion and a contraction giving movements of d_d at the end of the deck as described in **A.4.3**.

A.4.2 Definition of H' and d'_d for back and front of abutment

H' and d'_d for the back and front of abutment are defined as follows.

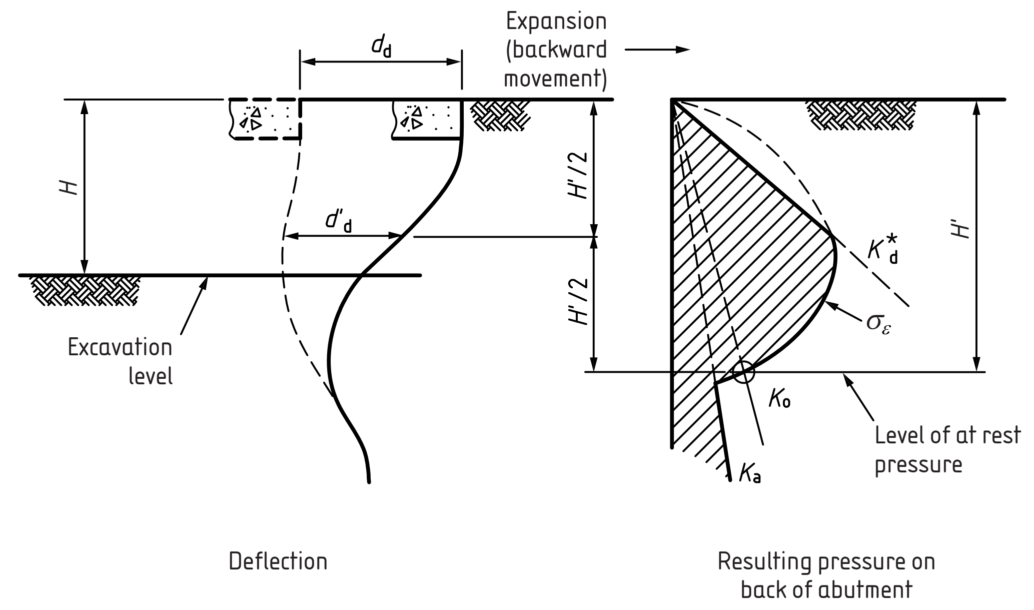
a) For the back of the abutment [see Figure A.2a)]:

- H' is the depth of soil behind the abutment affected by the repeated deck expansion, which may be taken as the depth from ground level to the level at which the earth pressure reduces to its at rest value when the deck is at its maximum expansion for the combination of actions under consideration; and
- d'_d is the corresponding horizontal deflection of the abutment wall at a depth $H'/2$ below ground level.

b) For the front of the abutment [see Figure A.2b)]:

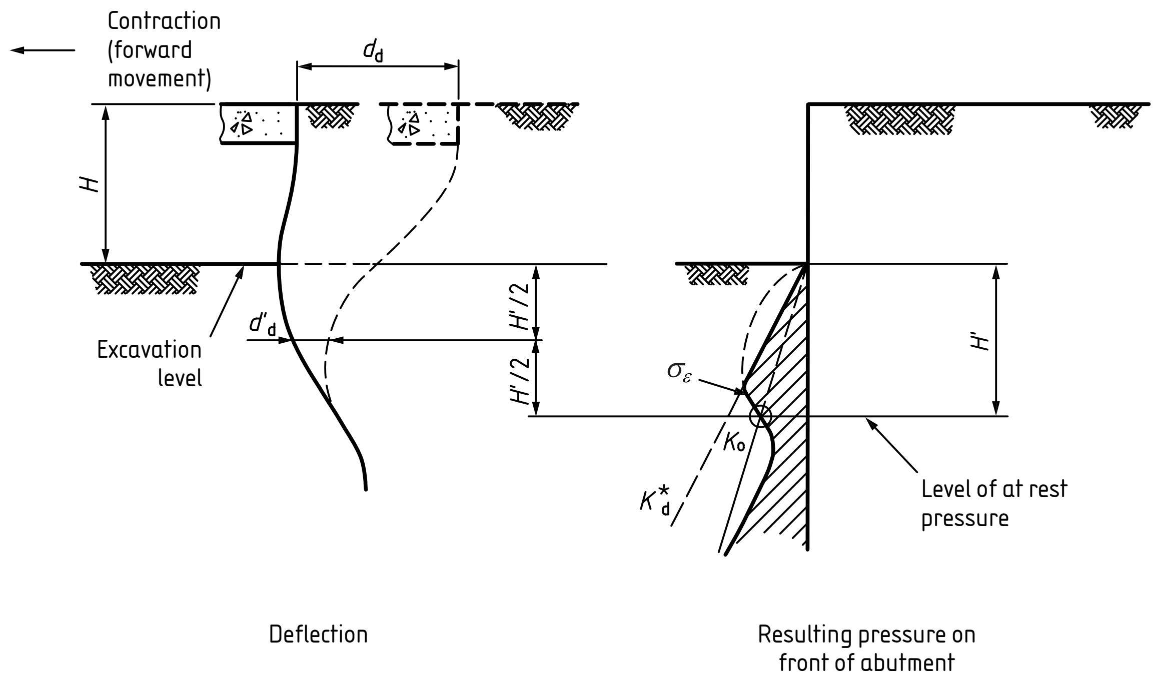
- H' is the depth of soil in front of the abutment affected by the repeated deck contraction, which may be taken as the depth from excavation level to the depth at which the earth pressure on the front of the wall or piles reduces to its at rest value when the deck is at its maximum contraction for the combination of actions under consideration; and
- d'_d is the corresponding horizontal deflection of the abutment wall at a depth $H'/2$ below excavation level.

Figure A.2 **Values of H' and d'$_d$ and illustration of earth pressures**



a) Earth pressures applied to the back of abutment (expansion case)

Figure A.2 **Values of H′ and d′$_d$ and illustration of earth pressures** *(continued)*



b) Earth pressures applied to the front of abutment (contraction case)

A.4.3 Iterative procedure for deriving d'_d and H'

The following iterative procedure may be used to determine H' and d'_d for the back and front of abutment.

- Step 1: Determine the design value of the thermal movement of the end of the bridge deck, d_d, for the combination of actions under consideration (see **9.4.2**).
- Step 2: Assume initial values of H' and d'_d for the back and front of the abutment. In the absence of previous experience of the behaviour of the type of bridge being analysed, suggested initial values of d'_d and H' are:
  a) for the back of abutment:
    - $H' = H$ (*i.e.* the depth from ground level to excavation level);
    - $d'_d = 0{,}5d_d$ for walls that are rotationally rigid at H;
    - $d'_d = 0{,}7d_d$ for walls that are rotationally flexible at H; and
  b) for the front of the abutment or piles:
    - $H' = H$ (as above);
    - $d'_d = 0{,}1d_d$.
- Step 3: Derive soil properties for the back and front of the abutment (see **A.3**) using the assumed or calculated values of d'_d and H', and input into model.
- Step 4: Apply contraction giving a movement of $d_d/2$ at the end of the deck.

- Step 5: Apply expansion giving a movement of d_d at the end of the deck and determine earth pressures applied to the abutment for this expansion case.
- Step 6: Identify the depth below ground level at which the earth pressure behind the abutment reduces to at rest pressure (H') and the resulting deflection d'_d at $H'/2$.
- Step 7: Compare the values of H' and d'_d for the back of the abutment with those used in Step 3. If these are significantly different, repeat Steps 4 to 7 using updated values of H' and d'_d.
- Step 8: Apply contraction giving a movement of d_d to the end of the deck and determine earth pressures applied to the abutment for this contraction case.
- Step 9: Identify the depth below excavation level at which the earth pressure in front of the abutment reduces to at rest pressure (H') and the resulting deflection d'_d at $H'/2$.
- Step 10: Compare the values of H' and d'_d for the front of the abutment with those used in Step 3. If these are significantly different, repeat Steps 8 to 10 using updated values of H' and d'_d.

A.5 Loading

A.5.1 Thermal action

The application of the thermal action to the model may be done either by applying the relevant deck expansion or contraction explicitly in the model or by applying an equivalent horizontal force. The equivalent horizontal force may be taken as the product of the deck stiffness, S_H (see **A.2**), and the relevant movement of the end of the deck (see **A.4**).

If the restraint provided by the deck is modelled using springs (see **A.2**), applying the equivalent horizontal force through the horizontal spring enables account to be taken directly of elastic shortening of the deck, as permitted in **9.4.2**.

A.5.2 Other variable actions

In the evaluation of earth pressures, the effect of variable actions defined in BS EN 1991, with the exception of thermal actions and groups of traffic loads that include braking and acceleration forces, may be neglected (but see **A.5.3**).

A.5.3 Subsequent frame analysis

The earth pressures determined using the analysis in this annex are the earth pressures that occur with various combinations of actions after many thermal cycles. These earth pressures need to subsequently be included in a model of the whole bridge in combination with an imposed simultaneous expansion or contraction of each end of the bridge of $\alpha L_x (T_{e;max} - T_{o;min}) \psi\gamma_Q$ or $\alpha L_x (T_{o;max} - T_{e;min}) \psi\gamma_Q$ respectively, and the most unfavourable combination of traffic and other variable actions, where $T_{o;min}$ and $T_{o;max}$ are the upper-bound and lower-bound values of T_o given in the National Annex to BS EN 1991-1-5 (see **9.6**).

A.5.4 Minimum expansion pressures with maximum contraction pressures

When the bridge stops expanding and starts contracting, the pressures behind the abutment are likely to reduce rapidly with very small associated contraction movements. Similarly, when the bridge stops contracting and starts expanding, the pressures behind the abutment are likely to increase rapidly with very small expansion movements. Such behaviour needs to be taken into account in the soil–structure interaction analysis, particularly for the design of members which are critical for maximum expansion and minimum earth pressures, and maximum contraction with maximum earth pressures. Alternatively, in these situations, at rest pressure ($K_{o;min}$ and $K_{o;max}$ respectively) may conservatively be assumed to be applied behind the abutments (see **9.4.8** and Figure 6).

Annex B (informative) Cases to be considered for buried concrete structures design

B.1 Cases to be considered

Table B.1 to Table B.6 show cases to be considered and the directly determined values of earth pressure coefficients that may be used with various loading combinations. These values only apply to backfill classes 6N and 6P, specified, installed and compacted in accordance with the *MCHW* [8], and to structures where there is uniformity of ground conditions.

B.2 Maximum pressure

The value of K_{max} represents the maximum pressure coefficients to be applied. It includes the effects of strain ratcheting and may be considered to be independent of the strength of the backfill.

B.3 Minimum pressure

The value of K_{min} represents the minimum pressure coefficient to be applied. It includes the effect of wall friction and thermal contraction and may be considered to be independent of the strength of the backfill.

B.4 Active pressure

The value of K_a given in Table B.4, Table B.5 and Table B.6 is a default value that may be used on the active face when braking or acceleration forces are applied to the structure.

B.5 At rest pressure

The value of K_o given in the tables is the at rest pressure coefficient applied to the vertical traffic surcharge in conjunction with K_{max}.

B.6 Sliding/overturning resistance

Where K_{max} pressure on the passive face is sufficient to resist sliding, overturning or excessive bearing pressures in conjunction with active pressure on the active face, the structure needs to be analysed for the moments and shears caused by these pressures. Where, however, K_{max} pressure is insufficient to prevent sliding or overturning, the assumed pressure on the passive face may be increased provided the structure is designed for the increased pressure and the movements required to mobilize these pressures are acceptable at the relevant limit state (see **10.3.2** and **10.3.3**).

B.7 Portal legs

Inwards and outwards sliding of portal legs need to be accounted for (see **10.3.4**).

B.8 Earth pressure coefficients

The values of the earth pressure coefficients (K) given in the tables include the partial factor γ_M in accordance with the UK National Annex to BS EN 1997-1 and a supplementary model factor $\gamma_{Sd;K}$ of 1,2 applied to permanent earth pressures at ULS.

B.9 Resistances R_1 and R_2

The resistances R_1 and R_2 shown underneath the bases of the portal structures are either the horizontal reactions required to prevent the bases sliding, or the sliding resistance of the bases, whichever is smaller.

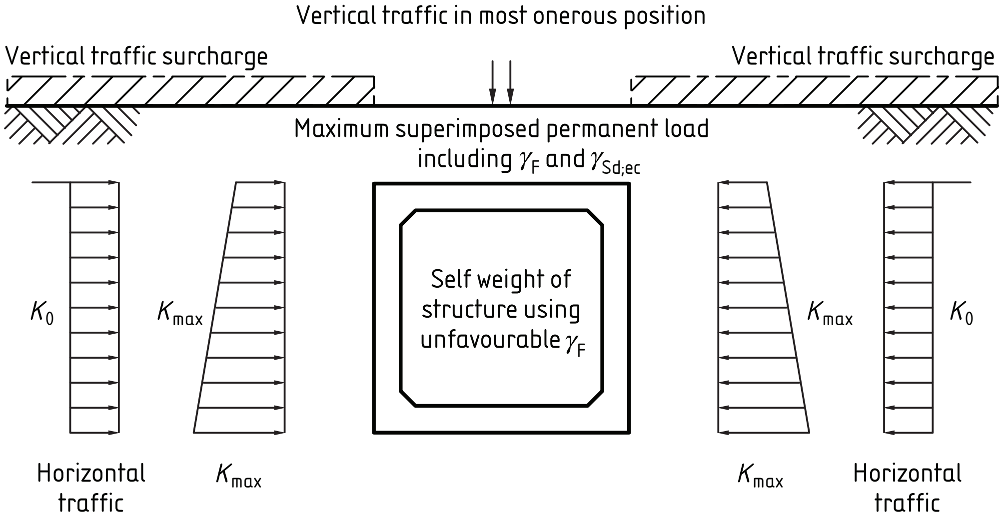
B.10 Combinations of actions

Combinations of actions need to be formed in accordance with the requirements of BS EN 1990 and relevant parts of BS EN 1991.

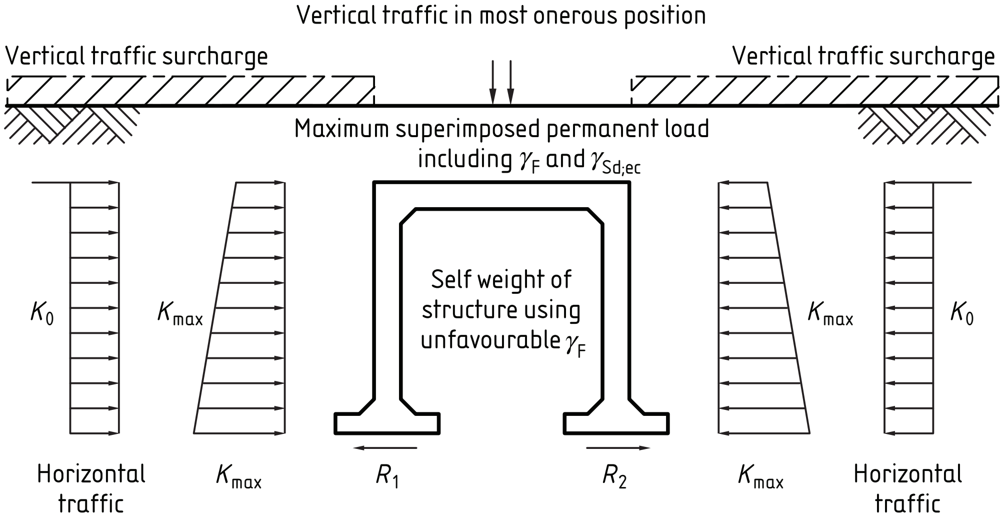
B.11 Superimposed permanent load

Where the maximum vertical load is being considered, the superimposed permanent load needs to include the model factor $\gamma_{Sd;ec}$ defined in **10.2.2** where applicable. Where the minimum vertical loading is being considered, $\gamma_{Sd;ec}$ is taken as 1,0.

Table B.1 **Maximum vertical load with maximum horizontal load**



Box structures



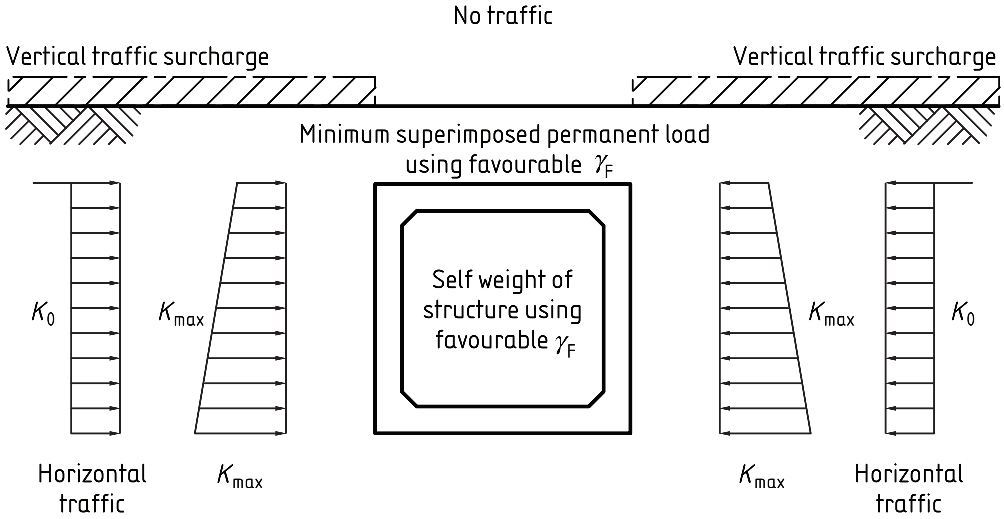
Portal structures

Directly determined design values of the earth pressure coefficient *K* that may be applied at various limit states

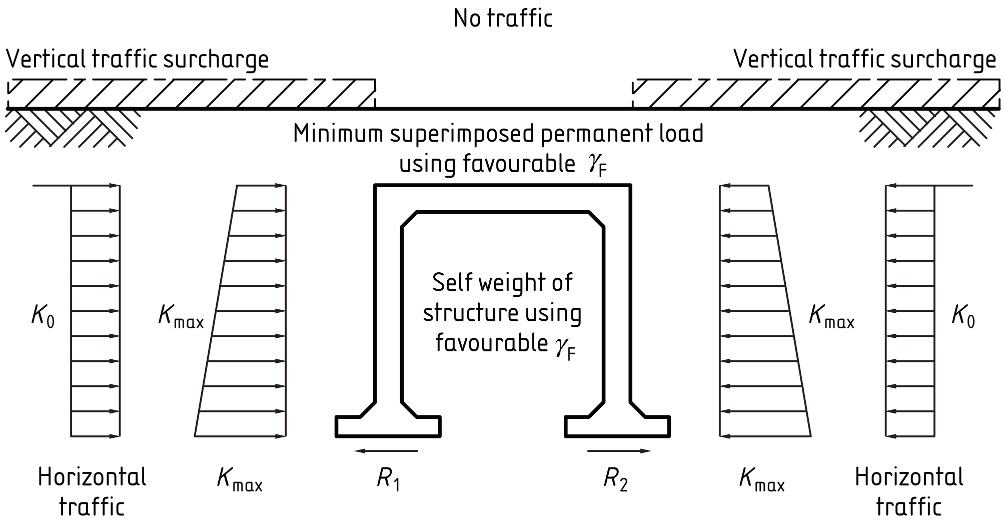
Horizontal traffic surcharge K_o	Earth pressure K_{max}	Limit state	Earth pressure K_{max}	Horizontal traffic surcharge K_o
0,5	0,60	SLS (characteristic)	0,60	0,5
0,54	0,77	EQU	0,77	0,54
0,50	0,72	STR/GEO (Comb 1)	0,72	0,50
0,58	0,84	STR/GEO (Comb 2)	0,84	0,58

NOTE The values of K include the partial factor γ_M, and (for permanent earth pressure at ULS) the model factor $\gamma_{Sd;K}$.

Table B.2 **Minimum vertical load with maximum horizontal load**



Box structures

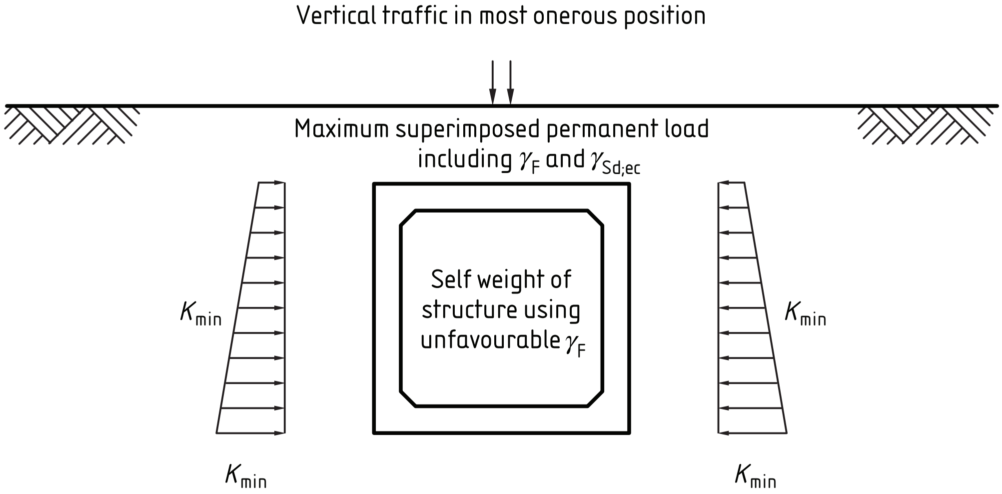


Portal structures

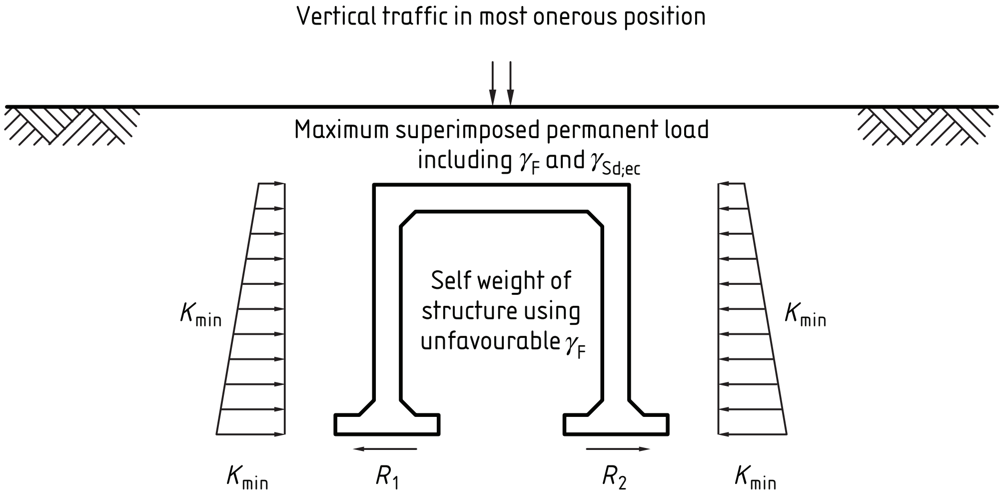
Directly determined design values of the earth pressure coefficient *K* that may be applied at various limit states				
Horizontal traffic surcharge K_o	Earth pressure K_{max}	Limit state	Earth pressure K_{max}	Horizontal traffic surcharge K_o
0,5	0,60	SLS (characteristic)	0,60	0,5
0,54	0,77	EQU	0,77	0,54
0,50	0,72	STR/GEO (Comb 1)	0,72	0,50
0,58	0,84	STR/GEO (Comb 2)	0,84	0,58

NOTE The values of K include the partial factor γ_M, and (for permanent earth pressure at ULS) the model factor $\gamma_{Sd;K}$.

Table B.3 **Maximum vertical load with minimum horizontal load**



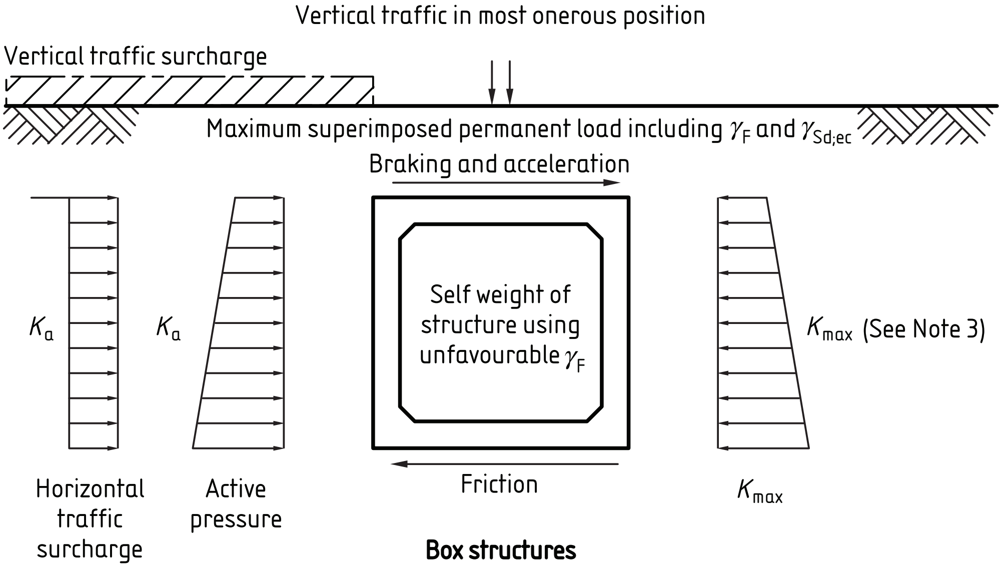
Box structures

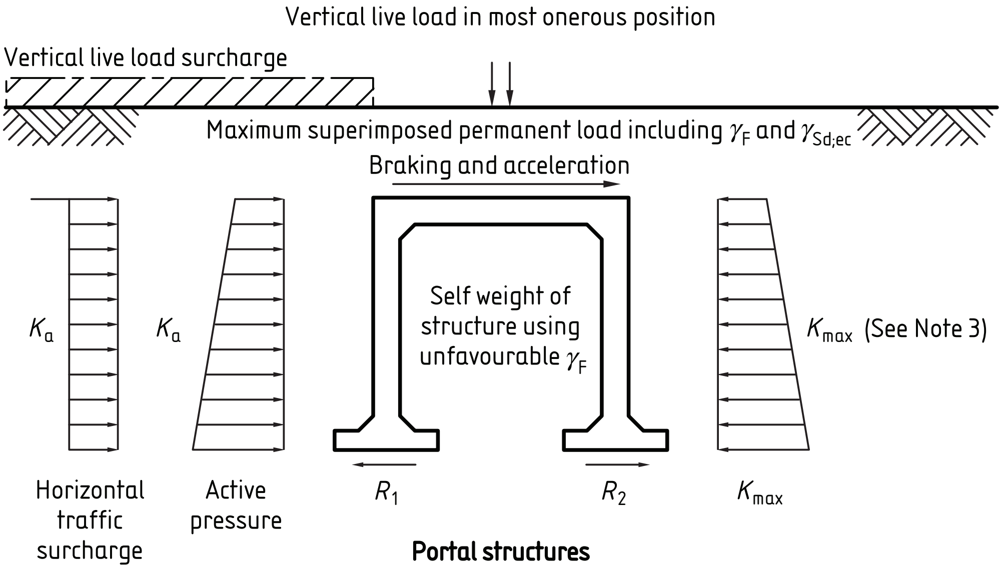


Portal structures

Design values of the earth pressure coefficient *K* that may be applied at various limit states		
Earth pressure K_{min}	Limit state	Earth pressure K_{min}
0,20	SLS (characteristic)	0,20
0,18	EQU	0,18
0,20	STR/GEO (Comb 1)	0,20
0,16	STR/GEO (Comb 2)	0,16
NOTE The values of K include the partial factor γ_M, and (for permanent earth pressure at ULS) the model factor $\gamma_{Sd;K}$.		

Table B.4 **Braking and acceleration with maximum vertical load and active pressure**





Directly determined design values of the earth pressure coefficient *K* that may be applied at various limit states

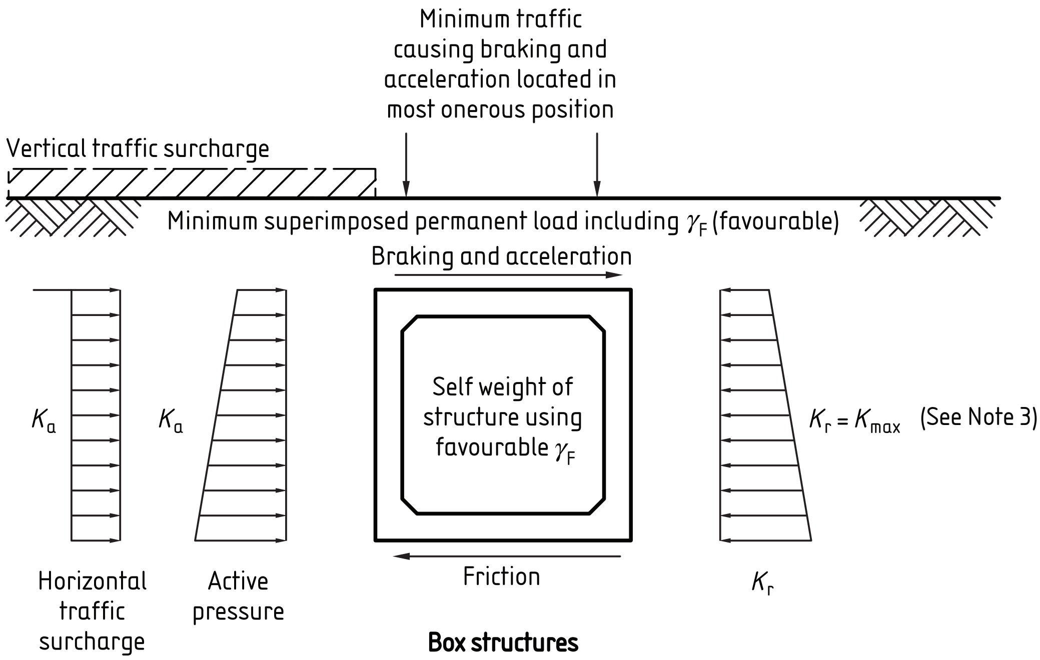
Horizontal traffic surcharge K_a	Earth pressure K_a	Limit state	Earth pressure K_{max}
0,33	0,33	SLS (characteristic)	0,6
0,37	0,44	EQU	0,6
0,33	0,40	STR/GEO (Comb 1)	0,72
0,41	0,49	STR/GEO (Comb 2)	0,84

NOTE 1 The values of K include the partial factor γ_M, and (for permanent earth pressure at ULS) the model factor $\gamma_{Sd;K}$.

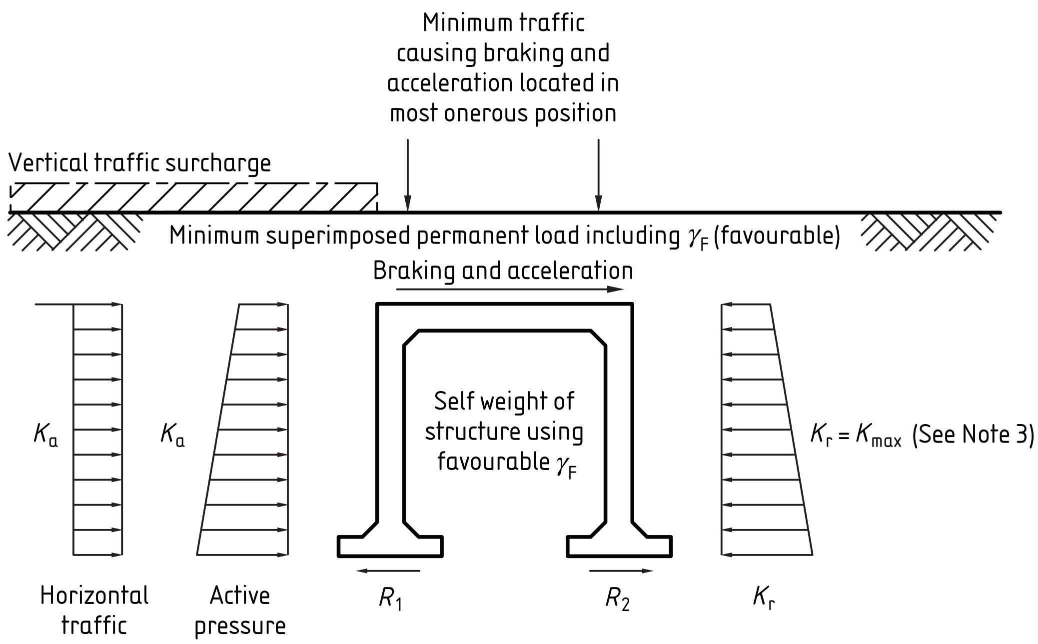
NOTE 2 If the structure sways towards the active side, this load case may be ignored.

*NOTE 3 The earth pressure coefficient for the passive wall may be taken as greater than K_{max} for bearing, sliding and overturning (see **10.3.2** and **10.3.3**) provided the associated displacements are acceptable at the relevant limit state.*

Table B.5 **Braking and acceleration with minimum vertical load and active pressures**



Box structures



Portal structures

Table B.5 **Braking and acceleration with minimum vertical load and active pressures** *(continued)*

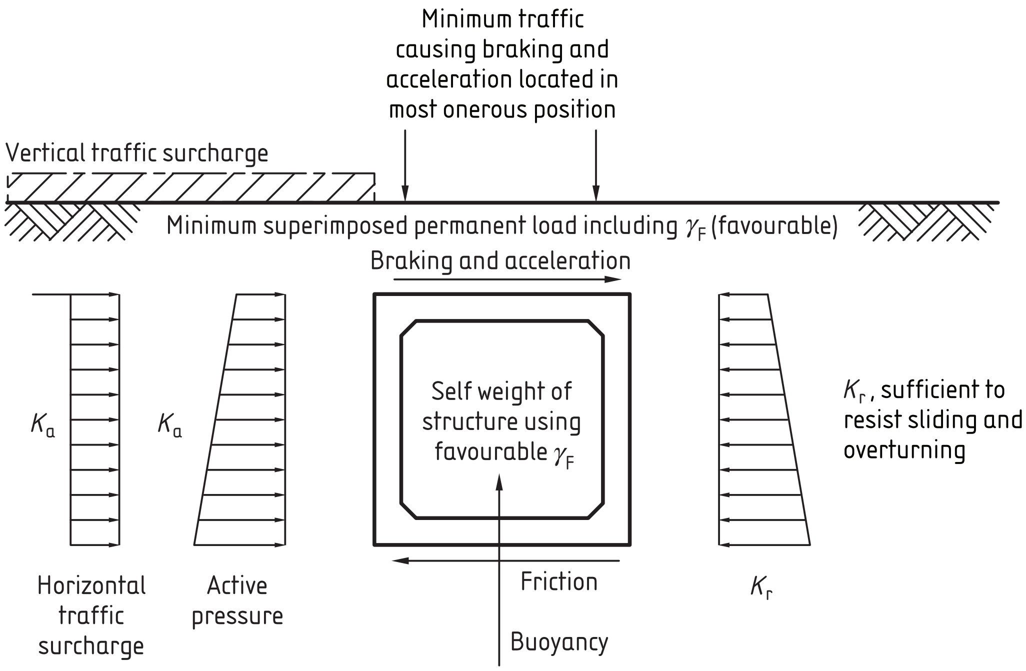
Directly determined design values of the earth pressure coefficient *K* that may be applied at various limit states			
Horizontal traffic surcharge K_a	Earth pressure K_a	Limit state	Earth pressure K_{max}
0,33	0,33	SLS (characteristic)	0,6
0,37	0,44	EQU	0,6
0,33	0,40	STR/GEO (Comb 1)	0,72
0,41	0,49	STR/GEO (Comb 2)	0,84

NOTE 1 The values of K include the partial factor γ_M, and (for permanent earth pressure at ULS) the model factor $\gamma_{Sd;K}$.

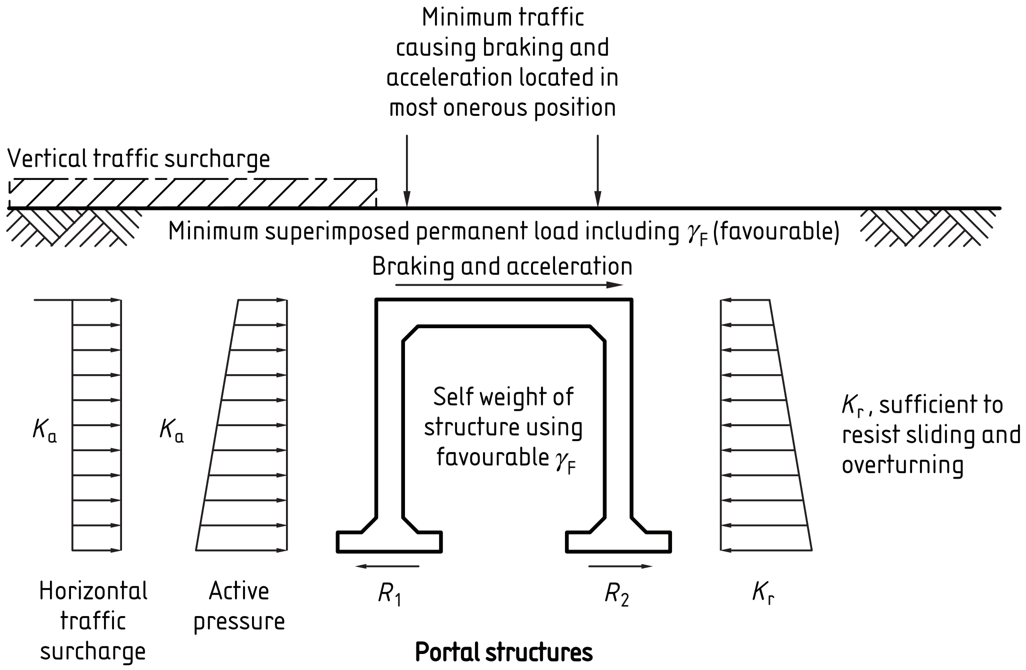
NOTE 2 If the structure sways towards the active side, this load case may be ignored.

NOTE 3 The earth pressure coefficient for the passive wall may be taken as greater than K_{max} for bearing, sliding and overturning (see ***10.3.2*** *and* ***10.3.3****) provided the associated displacements are acceptable at the relevant limit state.*

Table B.6 **Sliding**



Box structures



Portal structures

Table B.6 **Sliding** *(continued)*

Directly determined design values of the earth pressure coefficient *K* that may be applied at various limit states			
Horizontal traffic surcharge K_a	Earth pressure K_a	Limit state	Earth pressure K_r
0,33	0,33	SLS (characteristic)	K_r
0,37	0,44	EQU	K_r
0,33	0,40	STR/GEO (Comb 1)	K_r
0,41	0,49	STR/GEO (Comb 2)	K_r

NOTE The values of K include the partial factor γ_M, and (for permanent earth pressure at ULS) the model factor $\gamma_{Sd;K}$.

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