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

PART 12

BD 31/01

THE DESIGN OF BURIED CONCRETE BOX AND PORTAL FRAME STRUCTURES

SUMMARY

This document sets the Standard requirements for and gives advice on the design of buried concrete box and portal frame structures of precast and cast in-situ construction up to 15 metres long from abutment to abutment and with up to 11m of fill above the roof slab.

In addition the Standard gives requirements for construction, installation and procurement of such structures.

INSTRUCTIONS FOR USE

This revised Standard is to be incorporated in the Manual.

1. This document supersedes BD 31/87 and SB 3/88, which are now withdrawn.

2. Remove existing contents page for Volume 2 and insert new contents page for Voluem 2 dated November 2001.

3. Remove BD 31/87 and SB 3/88, which is superseded by BD31/01, and archive as appropriate.

4. Insert BD 31/01 in Volume 2, Section 2, Part 12.

5. Archive this sheet as appropriate.

Note: A quarterly index with a full set of Volume Contents Pages is available separately from The Stationery Office Ltd.

THE HIGHWAYS AGENCY

SCOTTISH EXECUTIVE DEVELOPMENT DEPARTMENT

THE NATIONAL ASSEMBLY FOR WALES
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THE DEPARTMENT FOR REGIONAL DEVELOPMENT
NORTHERN IRELAND

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Contents

SYMBOLS AND DEFINITIONS

SYMBOLS

The symbols used in this Standard are defined as follows:

A_s (mm²)	Area of tension reinforcement.
b (mm)	Width or breadth of section.
C (m)	Contact width of a single wheel on the ground.
d (mm)	Effective depth to tension reinforcement.
d_{cnr} (mm)	Effective depth of corner reinforcement
D (m)	Depth from ground level to point on abutment walls under consideration.
E_D (m)	Distance of centre of wheel from inside face of headwall.
E_T (m)	Distance of centre of traction force from nearer edge of structure or longitudinal joint.
F_R (kN)	Frictional resistance of foundation base.
F_{R1}, F_{R2} (kN)	Frictional resistance of left and right foundation bases of a portal frame.
h (mm)	Thickness of roof slab.
h_{na}(m)	Depth from top of roof to the neutral axis of the roof.
h_w (mm)	Thickness of walls
H (m)	Height of cover from top surface of the roof to ground level.
K	Earth pressure coefficient to be used for a given load in the design.
K_a	Coefficient of active earth pressure.
K_{min}	Minimum credible coefficient for balanced earth pressures.
K_p	Coefficient of passive earth pressure.
K_r	Coefficient of partial passive earth pressure resisting sway and sliding
K_o	Coefficient of lateral earth pressure "at rest".
K_t	Traction factor.
L_j (m)	Length of precast segmental unit (parallel to walls). Distance between longitudinal joints in in-situ construction. Overall transverse length (or width) of structure (=L_t) if there are no joints.
L_L (m)	Overall length of structure perpendicular to the walls.
L_t (m)	Overall transverse length (or width) of structure parallel to the walls.
L_1 to L_5 (m)	Dispersion widths of wheels or axles.
M_{max} (kNm/m)	Maximum "free span" moment per metre in the roof of a structure due to vertical live loads only at the serviceability limit state.
M_f (kNm/m)	Differential temperature moment for encastre roof.
M_{roof}	"Relaxed" differential temperature moment in roof.
M_{base}	"Relaxed" differential temperature moment in base.
n	Number of wheels under consideration.
Q*	Design Loads (Refer to BS5400: Part 1).
p_{sc} (kN/m²)	Horizontal Live Load surcharge pressure.

R_1, R_2 (kN)	Horizontal reactions on the left and right bases of a portal frame.
R*	Design Resistance (Refer to BS5400: Part 1).
S (m)	Transverse centre-line spacing of wheels.
S*	Design Load Effects.
T_{max} (^oC)	The maximum effective temperature of the roof.
T_{min} (^oC)	The minimum effective temperature of the roof.
T_q (kNm)	Plan torque applied to foundation.
T_R (kNm)	Rotational sliding resistance of foundation base.
V (kN)	Wheel Load.
v_{sc} (kN/m^2)	Vertical Live Load Surcharge pressure.
V_{tot} (kN)	Applied Vertical Load.
X_{clear} (m)	The clear span of a single span structure. The maximum clear span in a multi span structure.
X (m)	Effective square span of roof measured between the centres of the walls.
Y (m)	Effective height of box side-wall measured between the centres of the roof and the base slab.
Z (m)	Depth below water level.
α (per ^oC)	Coefficient of thermal expansion of concrete.
β	Superimposed Dead Load Factor.
γ (kN/m^3)	Average bulk unit weight of compacted fill and road construction materials.
γ_{fL}	Partial safety factor that takes account of the possibility of unfavourable deviation of the loads from their nominal values and of the reduced probability that various loadings acting together will all attain their nominal values simultaneously.
γ_{f3}	Partial safety factor that takes account of inaccurate assessment of the effects of loading, unforeseen stress distribution in the structure, and variations of dimensional accuracy achieved in construction.
γ_m	Partial safety factor that takes account of variability in material strength and uncertainties in the assessment of component strength.
δ	Design angle of wall friction.
δ_b	Design angle of base friction.
Δ (m)	Midspan deflection of roof.
η	Differential Temperature Reduction Factor
θ	Skew angle. The acute angle between the edge of the structure and the normal to the abutment walls.
ϕ'	Effective angle of shearing resistance.
ϕ'_{crit}	Critical state angle of shearing resistance. (Refer to BS8002).
ϕ'_{max}	Maximum value of ϕ' determined from conventional triaxial test or shear box test (Refer to BS8002).

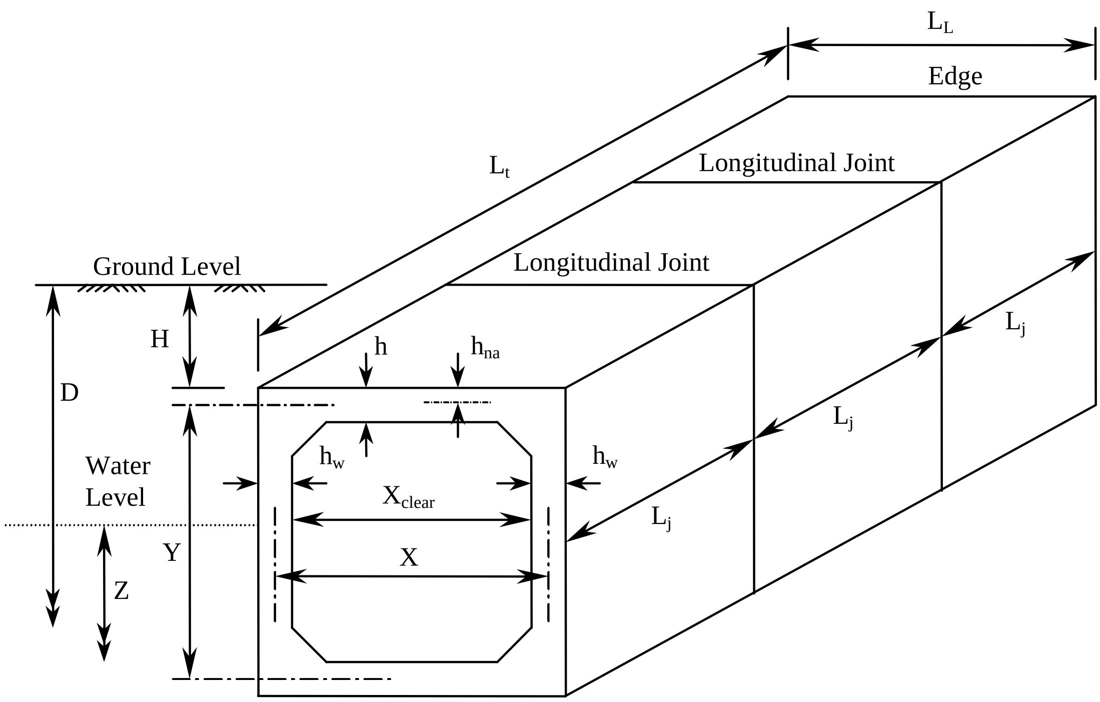


Figure 1 : Symbols for Typical Box Structure

DEFINITIONS

The following definitions and abbreviations are used in this standard:

"BS5400 Part 4" means BS5400 Part 4 as implemented by BD 24 (DMRB 1.3.1) and supplemented by BD 57 (DMRB 1.3.7).

"BD 37" means BS5400 Part 2 as implemented by BD 37 (DMRB 1.3).

"Longitudinal" means perpendicular to the walls.

"Transverse" means parallel to the walls.

"Ground Level" means finished carriageway level, or the temporary ground level on which traffic can run during construction.

"Hard material" means material which requires the use of blasting, breakers or splitters for its removal.

"Cover" means the depth of fill between ground level and the top of the roof.

"Abutment" means an end wall to which horizontal earth pressure loads are applied.

"Longitudinal Joint" means a break in the lateral structural continuity of the structure

"HAUDL/KEL Combination" means the combination of the Uniformly Distributed Load and Knife Edge Load described for HA loading in BD 37.

"Traction" means the longitudinal live load described in BD 37 arising from braking and acceleration of vehicles.

"Differential Temperature" means variations in temperature through a section at a given moment in time.

"Temperature Range" means the difference between the highest and lowest mean temperatures that an element is likely to sustain during its life or the specified return period.

"Self Equilibrating Stress" means the stresses occurring in a simply supported or continuous member as a result of a non-linear strain diagram being imposed on a section in which plane sections remain plane.

"SDL" is an abbreviation of "Superimposed Dead Load".

"SLS" is an abbreviation of "Serviceability Limit State".

"ULS" is an abbreviation of "Ultimate Limit State".

1. INTRODUCTION

1.1 SUMMARY

This document brings up to date the design requirements for buried concrete box and portal frame structures of precast segmental and in-situ construction. It sets the Standard requirements for these structures and gives advice on their design. The design rules are more comprehensive than those in BD 31/87 and have been written taking into account comments received over a period of years. Structures with depths of cover up to 11m are included in the scope of this Standard which also gives the requirements for construction and installation and the procedures to be followed when procuring these structures which permit the Contractor to choose a proprietary structure that meets the Overseeing Organisation's requirements.

1.2 EQUIVALENCE

The construction of buried concrete box and portal frame structures will normally be carried out under contracts incorporating the Specification for Highway Works (MCHW1). In such cases products conforming to equivalent standards or technical specifications of other states of the European Economic Area and tests undertaken in other states of the European Economic Area will be acceptable in accordance with the terms of Clauses 104 and 105 in Series 100 of MCHW1. Any contract not containing these Clauses must contain suitable clauses of mutual recognition having the same effect regarding which advice should be sought.

1.3 SCOPE

(a) This document sets the Standard for buried box structures and portal frames for which:

   (i) the depth of cover measured from the finished ground level to the roof of the structure is up to 11.0m

   (ii) the length of the structure between the inside faces of the outermost walls (measured perpendicular to the walls) is greater than 0.9m and up to 15.0m.

(b) The structures covered by this document are precast or in-situ boxes or portal frames, constructed of reinforced or prestressed concrete. The structures can be single or multi-span with roof slabs that are either integral with, or pinned to, the abutments.

(c) Structures *not* covered by this document include:

   (i) Structures with inclined abutment walls.

   (ii) Structures with abutment walls that are pinned at both top and bottom.

   (iii) Any structure that would behave as a mechanism when not backfilled.

   (iv) Structures with moving bearings at either abutment.

   (v) Proprietary precast arch structures.

   (vi) Structures with piled foundations.

   (vii) Structures with walls constructed of contiguous or similar piling.

   (viii) Structures with reinforced earth abutments.

(d) This document gives guidance on the installed structure but does not address the loads imposed during construction by thrust boring or jacking structures into place.

1.4 ENVIRONMENTAL CONSIDERATIONS

The Overseeing Organisation's requirements for environmental design shall be taken into account in designing buried box and portal structures. Volume 10 of DMRB (Environmental Design) gives advice on the use of underpasses by multiple species of small mammals and fish. It illustrates various forms of culvert design to facilitate free passage of these species. Often these considerations are fundamental to the determination of the span, headroom, cross-section invert and gradient of the structure.

1.5 IMPLEMENTATION

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

1.6 MANDATORY REQUIREMENTS

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

2. DESIGN PRINCIPLES

2.1 LIMIT STATES

(a) In this Standard, limit state principles have been adopted for the design of the structural elements and the foundations. Both an Ultimate Limit State and a Serviceability Limit State are considered.

(b) The Ultimate Limit State (ULS) is that represented by the collapse of the structural element concerned.

(c) The Serviceability Limit State (SLS) is that represented by the condition beyond which a loss of utility or cause for public concern may be expected and remedial action required. In particular, crack width shall be limited as described in Clause 4.2.1 and there shall not be excessive movement at the joints capable of seriously damaging the carriageway above. (See Clauses 4.2.3 and 4.2.4)

(d) Design loads (Q*) are expressed as the product of nominal loads and the partial safety factor γ_{fL}.

(e) Design load effects (S*) are expressed as the effects of the product of the design loads (Q*) and the partial safety factor γ_{f3}.

(f) Design resistance (R*) is expressed as the nominal strength of the component divided by the partial safety factor γ_m.

(g) The design load effects (S*) at ULS shall not be greater than the design resistance (R*).

(h) In addition, for precast segments at SLS, the vertical deflection of the roof slab under live loads (including foundation settlement) shall not be greater than the limiting value given in the Standard.

(i) The design life of buried concrete box and portal frame structures shall be 120 years.

2.2 DESIGN PRINCIPLES OF STRUCTURAL ELEMENTS

The Overseeing Organisation's requirements for the design of the concrete structural elements are contained in BS5400: Part 4 as implemented by BD 24 (DMRB 1.3.1) and supplemented by BD 57 (DMRB 1.3.7) except as specified otherwise by this Standard. These documents are hereafter referred to collectively as "BS5400 Part 4".

2.3 DESIGN PRINCIPLES OF FOUNDATIONS

Even when the structural elements are designed to their required strength, the structure as a whole can fail due to overloading of the soil-structure interface or excessive soil deformations. In order to prevent such failures occurring, two situations shall be investigated prior to carrying out the final structural design, to confirm whether or not the proposed geometry and structural form are suitable.

(a) Sliding

The possibility of failure of the structure by sliding on its base shall be investigated at ULS.

(b) Bearing Failure and Settlement of the Foundations.

The maximum net bearing pressure under the base of the structure under nominal loads shall be checked against the safe bearing pressure of the foundations to ensure that there is an adequate factor of safety against bearing failure of the foundation and to prevent excessive settlement and differential settlement.

2.4 LOADS

(a) The Overseeing Organisation's requirements for Loading are contained in BS5400: Part 2 as implemented by BD 37 (DMRB 1.3), except as specified otherwise by this Standard. These documents are hereafter referred to collectively as "BD 37"

(b) The following loads (which are described more fully in Chapter 3) shall be used in the design:

(i) Permanent Loads

Dead Loads
Superimposed Dead Loads
Horizontal Earth Pressure
Hydrostatic Pressure and Buoyancy
Differential Settlement Effects

(ii) Vertical Live Loads

HA or HB loads on the carriageway
Footway and Cycle Track Loading
Accidental Wheel Loading
Construction Traffic

(iii) Horizontal Live Loads

Live Load Surcharge
Traction
Temperature Effects
Parapet Collision
Accidental Skidding
Centrifugal Load

2.5 LOAD COMBINATIONS

The load combinations to be used in the design shall be as given in BD 37. Only combinations 1, 3 and 4 apply to this standard as follows:

(a) Combination 1 — Permanent loads, Vertical live loads and Horizontal live load surcharge.

(b) Combination 3 — Combination 1 plus temperature effects.

(c) Combination 4 — Permanent loads and Horizontal live load surcharge plus one of the following:

(i) Traction

(ii) Accidental load due to skidding

(iii) Centrifugal loads

(iv) Loads due to collision with parapets

and the associated vertical (primary) live loads in accordance with BD 37.

Details of the load combinations and the associated partial safety factors to be used in the design of the structural elements are given in Table 3.2 at the end of Clause 3.3

Details of the load combinations and associated partial load factors to be used in the design of the foundations are given in Clause 3.4.

3. LOADING

3.1 PERMANENT LOADS

3.1.1 Dead Load

The nominal dead load consists of the weight of the materials and parts of the structure that are structural elements excluding superimposed materials described below.

3.1.2 Superimposed Dead Load

(a) The nominal superimposed dead load consists of the weight of the soil cover and the road construction materials above the structure. It shall be applied to the roof of the structure as a uniformly distributed load.

(b) The possible effects of positive arching reducing this load shall be ignored.

(c) Where consolidation or settlement of the fill adjacent to a buried structure will cause negative arching of the fill above the roof, increased loading will be generated on the roof slab. These effects can be greater if the foundation is on hard material (see definitions). In the absence of reliable estimates of the effects of differential settlement between the structure and the adjacent ground, the superimposed dead load intensities to be applied to the roof of a structure with cover H shall be as follows:

(i) The minimum superimposed dead load intensity shall be taken as γH.

(ii) The maximum superimposed dead load intensity shall be taken as $\beta\gamma H$ where:

γ is the average nominal bulk density of the fill and surfacing and β is taken from Figure 3.1

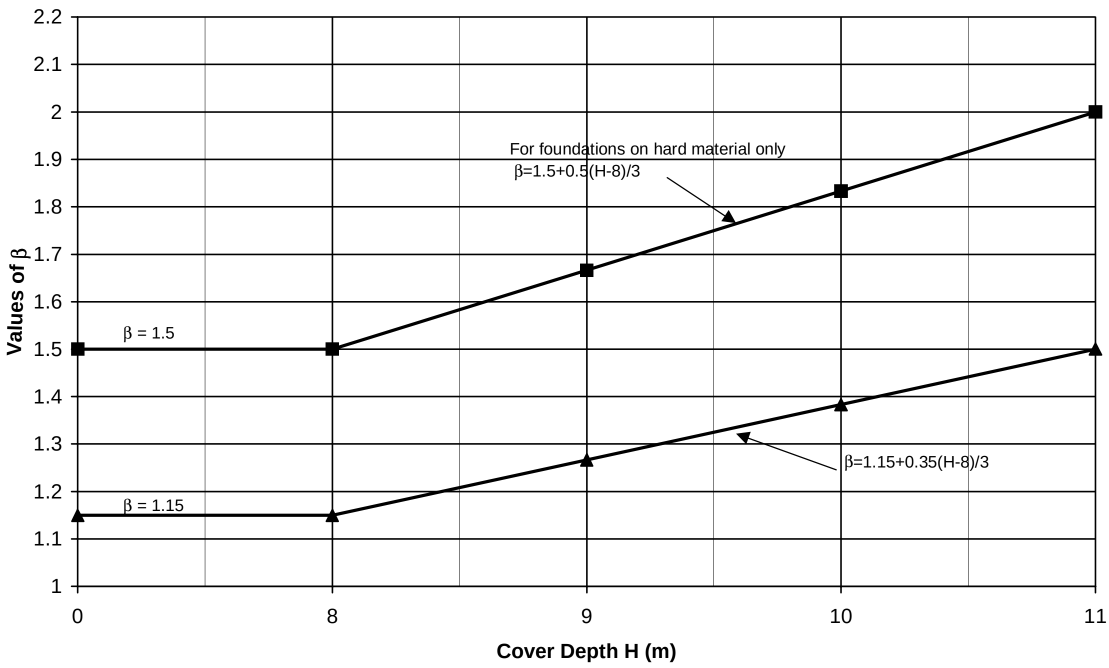


Figure 3.1

3.1.3 Horizontal Earth Pressure (permanent)

(a) The nominal permanent horizontal earth pressures applied to the side walls of the structure at a depth D below ground level shall be taken as follows:

(i) For Combination 1 and 3 loads:

A maximum earth pressure equal to $K_o\gamma D$ applied simultaneously on both side walls, or

A minimum earth pressure equal to $0.2\gamma D$ applied simultaneously on both side walls

(ii) For Combination 4 loads with traction:

A "disturbing" earth pressure equal to $K_a\gamma D$ acting in the same direction as the horizontal live load, and

A "restoring" earth pressure equal to $0.6\gamma D$ acting in the opposite direction to the horizontal live load

If under the above loading the structure sways in the opposite direction to the applied horizontal live load, this load case need not be considered.

(iii) For Combination 4 loads with skidding, centrifugal load or parapet collision, when relevant:

As (i) or (ii) above to give the most onerous effect.

(b) Values of Earth Pressure Coefficients

(i) If the backfill properties are not known but the backfill materials comply with the requirements of Chapter 5 the following nominal default values may be used:

K_{min} = 0.2
K_a = 0.33
K_o = 0.6
K_p = 3.0

(ii) If the backfill properties are known, the nominal values of K_a, K_o and K_p may be calculated from BS8002: 1994, as described in Appendix B. However, as the value of K_o determined using BS8002: 1994 does not account directly for effects such as compaction pressure, thermal expansion and cyclical loading (strain ratcheting) which can lead to a significant increase in earth pressure, the default value of 0.6 should be used for K_o (with γ_{fL} = 1.5) unless such effects are taken into account.

A minimum earth pressure coefficient of not more than 0.2 should be used where earth pressures are beneficial.

(iii) Pressures in excess of K_o but not exceeding $0.5K_p$ may be used to resist sliding (see Clause 4.1.2 (d) (ii) and Clause 4.4.2 (c)).

(c) Where the structure is constructed in a steep trench, the critical horizontal earth pressures may be applied by the native ground rather than by the backfill (see Clause 5.1.1(d)). This shall be investigated by considering potential failure planes in the native ground close to the edge of the trench.

3.1.4 Hydrostatic Pressure

When appropriate, the effect of hydrostatic pressure and buoyancy shall be taken into account. The increase in pressure on the back of the walls due to hydrostatic pressure at a depth Z metres below water level shall be taken as 10Z(1-K) kN/m^2.

3.1.5 Settlement

The settlement and differential settlement of the sub-soil under unfactored nominal permanent loads shall be calculated from BS8004 using the site investigation data. Any differential settlement of the soil that is likely to affect the structure shall be taken into account.

3.2 LIVE LOADS

Vertical Live Loading

3.2.1 HA and HB Carriageway Loading

The nominal carriageway loading shall be HA or HB Loading as described in BD 37, whichever is the more onerous.

(a) HA Loading

(i) Where the depth of cover (H) is 0.6m or less, HA loading shall consist of the HAUDL/KEL combination. No dispersion through the fill of either the HAUDL or the HA knife edge load shall be applied.

(ii) For cover depths exceeding 0.6m, the HAUDL/KEL combination does not adequately model traffic loading. In these circumstances the HAUDL/KEL combination shall be replaced by 30 Units of HB loading, dispersed through the fill as described in paragraph (c) below.

(iii) Account shall also be taken of the single 100kN HA wheel load, (dispersed through the fill as described in paragraph (c) below), where this has a more severe effect on the member under consideration than the loads described in (i) or (ii) above.

(b) HB Loading

(i) 45 Units of HB loading shall be applied on structures on Trunk Roads and Motorways. On structures on other Public Highways, 30 Units shall be applied unless a higher value is specified by the Overseeing Organisation.

(ii) A minimum of 30 Units of HB loading shall be applied to all structures including those that are designated to carry HA loading only.

(c) Dispersal of Wheel and Axle loads through the Fill

(i) All wheel loads shall be assumed to be uniformly distributed at ground level over a contact area, circular or square in shape, based on an effective pressure of 1.1N/mm^2.

(ii) Dispersion of a wheel load through the fill may be assumed to occur both longitudinally and transversely from the limits of the contact area at ground level to the level of the top of the roof at a slope of 2 vertically to 1 horizontally as shown in Figure 3.2a. Where the dispersion zones of the individual wheels overlap, they may be combined and distributed jointly as shown in Figure 3.2a (Zone 2). This applies to adjacent wheels on the same axle and to wheels on succeeding axles.

(iii) As an alternative to the method described in (ii) the effects of a wheel load on the structure may be derived using Boussinesq's theory of load dispersion as given in standard text books on soil mechanics. The Boussinesq Theory states that for an infinite elastic half space the vertical pressure at a horizontal distance *R* and a depth *z* from a vertical point load P applied to the surface is given by:

$$\frac{\dfrac{3P}{2\pi z^2}}{\left[1+\left(\dfrac{R}{z}\right)^2\right]^{2.5}}$$

(iv) Where however any individual wheel is located close to the edge of the structure such that its 2:1 dispersal zone is curtailed by a headwall, the increase in pressure near to the headwall shall be taken into account. This may be done by assuming that the load is dispersed transversely over the curtailed width of the 2:1 dispersal zone, as shown in Figure 3.2b.

(v) A wheel load not directly over the part of the structure being considered shall be included if its dispersion zone falls over the part of the structure.

(d) Dispersal of the Wheel and Axle Loads through the Roof Slab.

Where the dispersed width of the wheel or axle at roof level is less than the spacing between adjacent joints (L_j), a further lateral dispersal of the load may be made at 45^o down to the neutral axis of the roof slab (at depth h_{na}) so that:

A single wheel is dispersed over a total width of $C+H+2h_{na}$

An axle is dispersed over a total width of $C+(n-1)S+H+2h_{na}$

where:

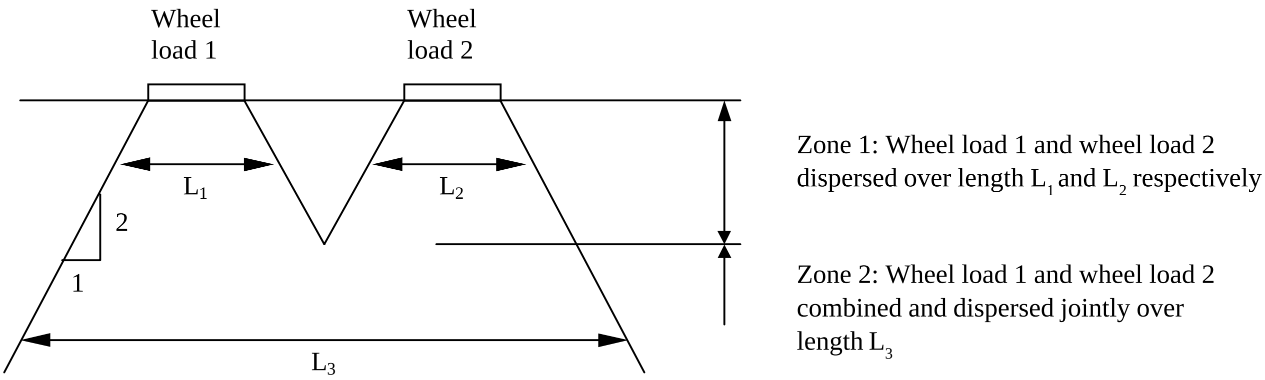
n is the number of wheels on an axle

S is the wheel spacing

and

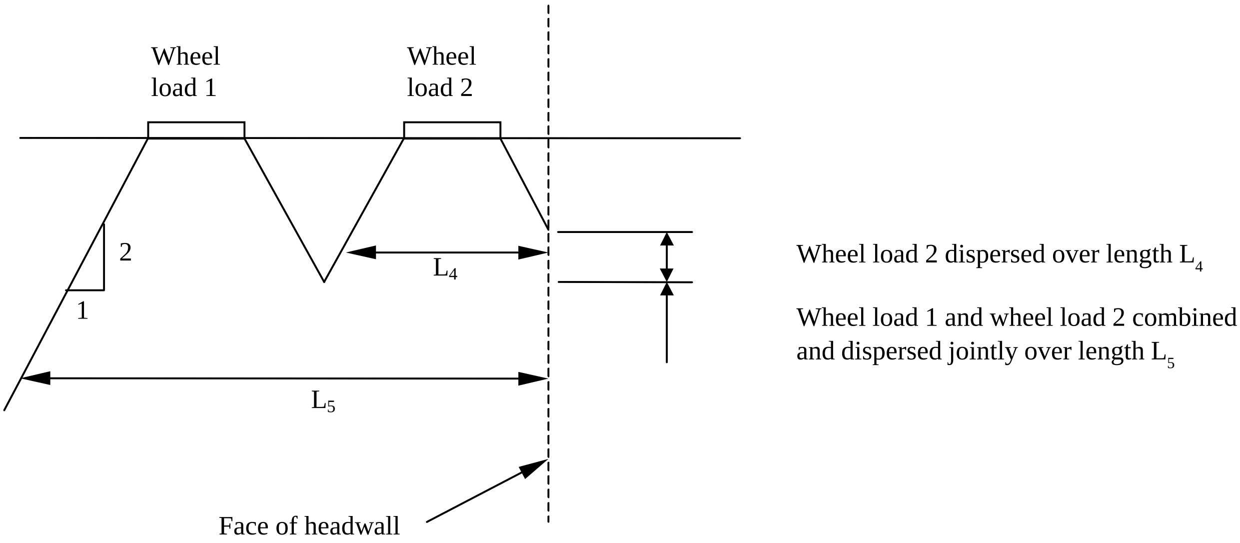
h_{na} is the depth from the top of the roof to the neutral axis which may for convenience be approximated to half the overall roof depth.

Dispersion through the slab at 45^o cannot occur through a longitudinal joint. The above approach does not account for the distribution properties of the structure itself (see Clause 4.1.1(e)).



Dispersion of wheel loads through fill (lateral and longitudinal)

Figure 3.2a



Example of lateral dispersion through fill adjacent to a side wall (longtudinal dispersion is as in Figure 3.2a)

Figure 3.2b

Figure 3.2

3.2.2 Footway and Cycle Track Loading

(a) Footway and cycle track loading shall consist of a load of 5kN/m^2 applied over the total area of the footway or cycle track except that this load may be reduced, by a factor of 0.8, to 4kN/m^2 for elements that carry both footway/cycle track loading and carriageway loading.

(b) The loading may be assumed to be dispersed at a slope of two vertically to one horizontally from the edge of the load to a total width not greater than twice the distance from the centre of the footway to the nearer headwall unless a more rigorous dispersion analysis is undertaken.

3.2.3 Accidental Wheel Loading on Edge Members

(a) Where the elements of a structure supporting outer verges, footways or cycle tracks are not protected from vehicular traffic by an effective barrier, they shall be designed to sustain the local effects of the accidental wheel loading described in BD 37. Each of the accidental wheel loads shall be dispersed through the fill using the principles described in Clause 3.2.1 (c) and (d) and Figures 3.2a and 3.2b.

(b) No other vertical live load nor dispersed load from the adjacent carriageway need be considered in combination with the accidental wheel loading.

3.2.4 Loading on Central Reserves

On dual carriageways the portion of structure supporting the central reservation shall be designed for full HA or HB carriageway loading.

3.2.5 Construction Traffic

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

Horizontal Live Loads

3.2.6 Live Load Surcharge

(a) A horizontal live load surcharge shall be applied in conjunction with all vertical live loads. The nominal uniform horizontal pressure (p_{sc}) to be applied to the external walls of the structure shall be determined from the equation:

$$p_{sc} = K.v_{sc}$$

where K is the value of the nominal earth pressure coefficient from Clause 3.1.3 for the wall under consideration and v_{sc} is the *vertical* surcharge pressure applied behind the abutments as follows:

Vertical LL	v_{sc}
HA Loading	10 kN/m^2
45 Units of HB	20 kN/m^2
30 Units of HB	12 kN/m^2
Footpath & Cycle Track	5 kN/m^2
Accidental Wheel	10 kN/m^2
Construction	10 kN/m^2 or as otherwise determined

For between 30 and 45 units of HB the value of v_{sc} shall be linearly interpolated.

(b) The same value of nominal live load surcharge with the same partial safety factors γ_{fL} and γ_{f3} shall be applied simultaneously to both external walls except as follows:

In conjunction with Combination 4 horizontal live loading (for traction see Diagrams A/4, A/5 and A/6 in Appendix A).

For calculating the maximum bearing pressure (see Diagram A/7 in Appendix A).

In these cases the live load surcharge pressure shall be applied on one face only to maximise the effect under consideration.

(c) It should be noted that when the minimum permanent earth pressure is applied on both sides of the structure (Clause 3.1.3(a)(i)) no live load surcharge shall be applied to either wall (see Diagram A/3a and 3b).

3.2.7 Traction

(a) The structure shall be designed to resist the traction forces (longitudinal live loads) described in BD 37.

(b) The traction force shall be applied perpendicular to the walls of the structure for precast construction and parallel to the direction of traffic for in-situ construction.

(c) HA Traction

(i) For structures with cover not exceeding 0.6m, the HA traction force shall be applied in accordance with BD 37. The loaded length for calculating the traction force shall be overall length of the structure in the direction of the force, except where the most onerous effect on the member under consideration occurs with the structure loaded over only part of its length.

(ii) For structures with cover greater than 0.6m, no traction force need be considered in conjunction with the 100kN HA wheel load, but where, as in Clause 3.2.1(a)(ii), HA loading is replaced by 30 units of HB vertical loading, this shall be applied in conjunction with 30 units of HB traction, with γ_{fL} at ULS taken as 1.1.

(d) HB Traction

HB traction shall be applied in accordance with BD 37 when one or more axles of the HB vehicle are on the structure.

(e) All traction forces shall be multiplied by K_t before they are applied directly to the roof of the structure where:

$$K_t = (L_L – H)/(L_L – 0.6) \text{ but } 1 \geq K_t \geq 0$$

(f) The traction force shall be applied directly to the roof of the structure over the following widths measured perpendicular to the direction of the traction force.

HA traction - a width equal to the notional carriageway lane width given in BD 37

HB traction - a width equal to 3 + C metres.

(g) In-situ boxes and portal frames are very effective in the lateral distribution of traction because of the in-plane rigidity of their roof slabs. For in-situ structures designed on a metre width basis, the traction force may therefore be considered to be distributed transversely through the structure over a width of $2E_T$, where E_T is the distance of the centre of the traction force from the nearer edge of the structure (or from the nearest longitudinal joint) but not less than half the traction width given in (f) above. Alternatively, if consideration is given to the lateral eccentricity of the traction force and the resistance to plan rotation of the foundations and walls, the traction may be considered to be distributed over the full width L_j.

3.2.8 Load Effects Due to Temperature

(a) Temperature effects may be neglected where:

(i) the cover (H) > 2m and $X_{clear} < 0.2L_t$, or,

(ii) the overall length of the structure $L_L \leq 3$m.

(b) In all other buried structures the variations in mean temperature (Temperature Range) and temperature gradients within a section (Differential Temperature) shall be applied as given below. The coefficient of thermal expansion (α) shall be taken as 12 x 10^{-6} per °C for concrete except for concrete with limestone aggregates where α may be taken as 9 x 10^{-6} per °C. Interaction between the backfill and the structure due to temperature effects may be neglected (but see Clause 3.1.3(b)(ii))

(i) Temperature Range

The temperature range to be used for both the in-service stage and the construction stages shall be in accordance with BD 37, except that the expansion and contraction of the roof relative to the base of the walls due to thermal effects may be based on the temperature range given below:

	Temperature Range	
	For Expansion	**For Contraction**
Box structures	10°C to T_{max}	10°C to T_{min}
Precast Portal frames	0°C to T_{max}	20°C to T_{min}
In-situ Portal frames	10°C to T_{max}	30°C to T_{min}

where T_{max} and T_{min} are the maximum and minimum effective temperatures of the roof given in Table 3.1.

The datum temperatures for box structures given above are based on the assumption that the temperature of the base will not be greater than 10°C when the roof is at its coldest and not be less than 10°C when the roof is at its hottest.

The datum temperatures for precast portal frames assume that when a precast segment is installed its effective temperature will not be less than 0°C nor more than 20°C. For insitu portal frames these figures are both increased by 10 degrees to compensate for effects due to the heat of hydration in the roof and subsequent shrinkage.

The datum temperatures for portal frames may be modified if limits on the temperature at the time of construction or installation are specified.

The effects of temperature range shall be considered at SLS. They need not be considered at ULS for buried structures because expansion or contraction beyond the serviceability limit state up to the ultimate range will not cause collapse of the structure.

(ii) Differential Temperature

The effects of temperature gradients within a section (differential temperature) shall be applied to the roof slab only, but the effects of the resulting flexure on other members shall be considered.

The differential temperature gradients within the roof section for both the in-service and construction stages shall be taken from Table 3.1. For structures with both $X_{clear} < 0.2L_t$ and $H > 0.6$m, the moments and shears calculated using the temperature differences and dimensions given in BD 37, Figure 9, Group 4 should be multiplied by the reduction factor η given in Table 3.1.

Simple methods for calculating the nominal moments and shears around a structure arising from the temperature differences in the roof slab taken from BD 37, Figure 9, Group 4 are given in Appendix C.

Self equilibrating stresses due to differential temperature (see “Definitions”) need only be considered for prestressed concrete roof slabs at SLS.

<table>
<tr><th rowspan="2">Span to Width Ratio</th><th rowspan="2">Cover</th><th rowspan="2" colspan="2">Minimum and maximum effective temperature</th><th colspan="2">Differential temperature</th></tr>
<tr><th>Temperature difference</th><th>Reduction factor</th></tr>
<tr><td>X_{clear}/L_t</td><td>H (m)</td><td>T_{min}</td><td>T_{max}</td><td></td><td>η</td></tr>
<tr><td>≥ 0.2*</td><td>All depths</td><td colspan="2">In accordance with BD 37</td><td>In accordance with BD 37**</td><td>N/A</td></tr>
<tr><td rowspan="5">< 0.2</td><td>H ≤ 0.6</td><td colspan="2">In accordance with BD 37</td><td>In accordance with BD 37**</td><td>N/A</td></tr>
<tr><td>0.6 < H ≤ 0.75</td><td>0^oC</td><td>20^oC</td><td>From BD 37, Figure 9, Group 4</td><td>0.5</td></tr>
<tr><td>0.75 < H ≤ 1.0</td><td>4^oC</td><td>16^oC</td><td>From BD 37, Figure 9, Group 4</td><td>0.33</td></tr>
<tr><td>1.0 < H ≤ 2.0</td><td>7^oC</td><td>13^oC</td><td>From BD 37, Figure 9, Group 4</td><td>Zero</td></tr>
<tr><td>H > 2.0m</td><td colspan="4">Temperature effects may be neglected</td></tr>
</table>

Load effects due to Temperature

TABLE 3.1

* Structures for which the maximum clear span X_{clear} is more than 20% of the width L_t are considered to be open to the atmosphere and the effects of temperature are therefore taken into account in accordance with BD 37

** For fill depths greater than 0.2m the temperature differences given in Table 24 of BD 37 for 0.2m of surfacing may be used. For roof slabs less than 600mm thick the resulting “fixed end moments” may be approximated as 0.5 times the values given in Table C1 of Appendix C.

3.2.9 Parapet Collision

The effects of parapet collision loading shall be considered in accordance with BD 37 where such loading is transmitted to the buried structure. Particular care should be taken in the design of segmental structures to ensure that such loading does not lead to the opening of joints between segments. In most cases with segmental construction it will be necessary to design headwalls and parapets which carry large transverse loads, such as vehicle impact or earth pressure, as independent structures which do not transmit these loads to precast units.

3.2.10 Skidding Loads

For structures with cover not exceeding 0.6m, loading due to skidding forces shall be considered in accordance with BD 37. Skidding loads do not need to be considered for structures with cover exceeding 0.6m.

3.2.11 Centrifugal Loads

For structures with cover not exceeding 0.6m, loading due to centrifugal forces shall be considered in accordance with BD 37. Centrifugal loads do not need to be considered for structures with cover exceeding 0.6m.

3.3 LOAD COMBINATIONS AND PARTIAL SAFETY FACTORS FOR THE DESIGN OF THE STRUCTURAL ELEMENTS

3.3.1 Load Combinations to be used for the Design of Structural Elements

The loads to be applied simultaneously in any load combination for the design of the structural elements are shown in Table 3.2.

3.3.2 Values of γ_{fL} to be used for the Design of Structural Elements

To obtain the design loads for a given load combination, the relevant nominal loads described in Clause 3.1 and 3.2 shall be multiplied by the corresponding value of γ_{fL} given in Table 3.2, except that, for the applied loads causing a relieving effect on the element under consideration, the value of γ_{fL} shall be taken as 1.0.

Where the same nominal values of horizontal earth pressure or live load surcharge are applied simultaneously on *both* sides of the structure, the same values of γ_{fL} (from Table 3.2) shall also be applied to the relevant loads on each side of the structure. (See Diagrams A/1 to A/3 in Appendix A).

3.3.3 Values of γ_{f3} to be used in the Design of Structural Elements

(a) The value of γ_{f3} at SLS shall be taken as 1.0

(b) The value to γ_{f3} at ULS shall be taken as 1.1 except:

(i) for all relieving effects γ_{f3} shall be taken as 1.0

(ii) For disturbing effects at ULS, where plastic methods are used in the analysis, γ_{f3} shall be taken as 1.15, as in BS5400 Part 4.

LOADS		**Limit State**	**γ_{fL} for Combinations**		
			1	**3**	**4**
PERMANENT LOADS					
Weight of concrete	3.1.1	ULS* SLS	1.15* 1.00	1.15* 1.00	1.15* 1.00
Superimposed pavement construction (top 200mm)	3.1.2	ULS** SLS	1.75 1.20	1.75 1.20	1.75 1.20
Superimposed fill including pavement construction in excess of 200mm	3.1.2	ULS SLS	1.20 1.00	1.20 1.00	1.20 1.00
Horizontal earth pressure (using default earth pressure coefficients)	3.1.3	ULS SLS	1.50 1.00	1.50 1.00	1.50 1.00
Horizontal earth pressure (using earth pressure coefficients calculated in accordance with BS8002)	3.1.3	ULS SLS	1.20 1.00	1.20 1.00	1.20 1.00
Hydrostatic pressure and buoyancy	3.1.4	ULS SLS	1.10 1.10	1.10 1.10	1.10 1.10
Settlement	3.1.5	ULS SLS	1.20 1.00	1.20 1.00	1.20 1.00
LIVE LOADS					
Vertical Live Loads HA carriageway loading	3.2.1	ULS SLS	1.50 1.20	1.25 1.00	
HB carriageway loading	3.2.1	ULS SLS	1.30 1.10	1.10 1.00	
Footway and cycle track loads	3.2.2	ULS SLS	1.50 1.00	1.25 1.00	
Accidental wheel loading	3.2.3	ULS SLS	1.50 1.20		
Construction traffic	3.2.5	ULS SLS	1.15 1.00	1.15 1.00	
Horizontal pressure due to live load surcharge	3.2.6	ULS SLS	1.50 1.00	1.50 1.00	1.50 1.00
HA traction and associated vertical	3.2.7	ULS SLS			1.25 1.00
HB traction live loads	3.2.7	ULS SLS			1.10 1.00
Temperature range	3.2.8	ULS SLS		N/A 1.00	
Differential temperature	3.2.8	ULS SLS		1.00 0.80	
Parapet collision and	3.2.9	In accordance with BD 37			
Skidding associated vertical	3.2.10	ULS SLS			1.25 1.00
Centrifugal load live loads	3.2.11	ULS SLS			1.50 1.00

(1)* γ_{fL} shall be increased to at least 1.20 to compensate for inaccuracies when dead loads are not accurately assessed.
(2)** γ_{fL} may be reduced to 1.2 and 1.1 for ULS and SLS respectively subject to the approval of the appropriate authority.

Loads, Load Combinations and Values of γ_{fL} for the Design of Structural Members

TABLE 3.2

3.4 LOAD COMBINATIONS AND PARTIAL SAFETY FACTORS FOR THE DESIGN OF THE FOUNDATIONS

3.4.1 Sliding

The loads to be applied simultaneously for checking the foundations against sliding shall be as follows:

	γ_{fL} (ULS)	γ_{f3}
Dead Load	1.0	1.0
Minimum Superimposed Dead Load	1.0	1.0
Buoyancy	1.1	1.1
Traction	1.25(HA)/ 1.1(HB)	1.1
Vertical Live Load associated with Traction	1.0	1.0
Disturbing Earth pressure (active)	1.5/1.2*	1.1
Disturbing Live Load Surcharge (active)	1.5/	1.1
Relieving earth pressure (see Clause 4.4.2 (c))	1.0	1.0

*The value of 1.5 is to be used with the default value of K and 1.2 for the value of K determined in accordance with BS8002.

These loads shall be applied at ULS only, using the values of γ_{fL} and γ_{f3} given above, (see also Diagram A/6 in Appendix A).

If the net horizontal force is in the opposite direction to the traction force, sliding need not be considered (except for rotational sliding: see Clause 4.1.2 (d) and Clause 4.4.2 (d))

3.4.2 Bearing Pressure and Settlement

The bearing pressures and settlements under the foundations shall be calculated for the following *nominal* loads as shown on Diagram A/7 in Appendix A:

Dead Load
Maximum superimposed dead load
Maximum horizontal earth pressure on both sides of the box
Hydrostatic Pressure and Buoyancy
Vertical Live Loading
Live Load surcharge on one side of the box only

4. DESIGN

4.1 DESIGN OF STRUCTURAL ELEMENTS

4.1.1 Structural Analysis

(a) The structure shall be analysed as a continuous frame, with pin joints where the walls are not continuous or fully integral with the roof slab or base. The stiffness of any corner fillets may be taken into account. Both ULS and SLS shall be considered.

(b) For boxes, an elastic compressible support may be assumed below the base slab except for structures founded on hard material (see definitions). In the former case the foundation shall be considered to be “flexible” and in the latter case the foundation shall be considered to be “rigid”.

(c) For portal structures, where the moments in the frame are sensitive to the rotational stiffness of the foundations, separate analyses shall be carried out for footings where the foundation shall be considered to be (a) “rigid” and (b) “flexible”, (see 4.1.1(b)), to ensure that the effects of the full range of possible foundation stiffnesses are considered.

(d) Moments and shears shall be obtained from the analysis at critical positions around the structure. The most critical positions for shear will normally be at a distance “d” from the inside edge of the fillets (or from the internal corners if there are no fillets) and both shear and coexisting moment shall be calculated at these and other critical positions, see also Clause 4.3.3.

(e) Analysis by the Unit Width Method

(i) In most situations it will be adequate to analyse the structure on a “metre strip” basis using a two-dimensional frame or similar. For structures where the dispersal zones of adjacent wheels overlap as shown in Zone 2 in Figure 3.2a, it will normally be adequate to base the load per metre width due to vertical live load on the dispersion widths determined in accordance with Clauses 3.2.1(c) and (d).

(ii) For structures with shallow fill, where the dispersal zones of adjacent wheels do not overlap (and for the single HA wheel load), the above method may lead to unacceptably conservative results. This is because no account is taken of the lateral load-distribution properties of the structure itself. A more realistic distribution width for calculating the live load effect per metre in this situation may be found by using the Pucher Charts, or the method for the Distribution of Concentrated Loads on Slabs given in BS8110, or by other rigorous methods. It should be noted however that in an elastic analysis, an individual HB wheel cannot be distributed over a width significantly greater than the wheel spacing because of the effects of adjacent wheels. Also, a single dispersed wheel load which is narrower than the segment width (L_j) cannot be distributed over a width wider than the segment width.

(f) If a three dimensional model is used consideration shall be given to the interaction of live loads in adjacent lanes as described in BD 37.

(g) Portal frames shall be designed for the more onerous effects resulting from assuming that:

(i) the base of each wall is fully restrained against horizontal movement,

(ii) the base of each wall is restrained longitudinally by a horizontal force not exceeding the frictional resistance of the footing under that wall (see Clause 4.4.2).

Where the frictional resistance of the footing is adequate to restrain the wall against horizontal movement then only case (i) need be considered.

4.1.2 Skew

(a) Precast units shall be rectangular in plan and skew effects can therefore be neglected for precast construction except that any special skewed edge units shall be designed separately.

(b) In-situ skewed 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 $X\sin\theta$ from the edge of the structure shall be designed on the basis of the skew span.

(c) Alternatively skewed in-situ structures may be analysed by more rigorous methods such as a three dimensional computer analysis.

(d) In 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 will be 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 pressure towards the obtuse corners of the abutment walls.

(i) 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.

(ii) If $L_L \tan\theta \geq L_j/6$ and the applied torque (T_q) is greater than the frictional resistance torque of the base (T_R), consideration shall be given to resisting the unbalanced torque by increasing the horizontal earth pressure on the walls towards the obtuse corner. In this case, the maximum earth pressure anywhere on the wall at ULS at depth D (excluding live load surcharge) shall not exceed $0.5\gamma DK_p$ and the structural elements must be designed to resist this increased pressure.

(iii) It should be noted that 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 will tend to increase, rather than resist the tendency of the structure to rotate in plan and the danger of failure due to rotational sliding needs to be carefully examined.

4.1.3 Stages to be Analysed

Three stages shall be considered:

(i) The completed structure backfilled up to the top of the roof.

(ii) 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.

(iii) The structure, fully backfilled, in service.

4.1.4 Load Cases to be Considered

(a) Each element of the structure shall be designed for each of the three stages listed above, using the most onerous of the following Combinations 1 and 3 effects:

(i) Permanent loads with maximum or minimum dead load surcharge (excluding differential settlement in Stages i and ii)

(ii) Maximum or minimum horizontal earth pressures

(iii) The appropriate Combination 1 and 3 live loads positioned to give the most severe effect to the element under consideration.

(iv) Temperature effects (Combination 3 only)

(b) In addition, for the “In Service” stage (Stage iii of Clause 4.1.3), the structure shall be designed for the most onerous of the following Combination 4 effects:

(i) Permanent loads with maximum or minimum dead load surcharge.

(ii) The Combination 4 horizontal earth pressures described in Clause 3.1.3. (a) (ii) or (iii).

(iii) Either traction, skidding, centrifugal or parapet collision loading.

(iv) The associated Combination 4 vertical live loads positioned to give the most severe effect to the element under consideration.

(c) For Combinations 1 and 4 with traction, the load cases to be applied for the design of the structural elements are shown diagrammatically in Appendix A diagrams A/1 – A/5.

4.2 SPECIAL REQUIREMENTS AT THE SERVICEABILITY LIMIT STATE

4.2.1 Crack Control

In Combination 1 at SLS, crack widths shall be limited in accordance with BS5400 Part 4, except that where the cover (H) is greater than 0.6m, the crack width should be checked for 30 Units of HB rather than HA loading.

4.2.2 Early Thermal Cracking

Early thermal cracking need not be considered for precast segmental construction where the segments are monolithic and of 3m length (L_j) or less. For other buried structures the requirements for the control of early thermal cracking are as specified in BS5400 Part 4, BD 28 (DMRB 1.3) and BD 37 (DMRB 1.3), except that the horizontal steel required to resist early thermal cracking need not be placed outside the primary longitudinal reinforcement. Guidance is also given in BA 57 (DMRB 1.3.8).

4.2.3 Deflection

(a) In precast construction, and in in-situ structures with longitudinal joints that do not comply with 4.2.4 (c)(ii), the net vertical deflection at the midspan of the roof under the combined effects of the elastic deflection of the structure and the short-term settlement of the foundations under the application of vertical live loads at SLS shall be less than 0.015H. This limitation is required to prevent the occurrence of excessive movements at longitudinal joints in structures with low covers, which can seriously damage the overlying carriageway.

(b) Where an assessment of the live load deflection of the roof is required, it will be sufficiently accurate to estimate the midspan deflection of the roof using the empirical formula:

$$\Delta = 20M_{max}\ X^2/h^3 \text{ metres}$$

where X is the effective span in metres, h is the overall depth of the roof in millimetres and M_{max} is the maximum “free span” moment in kNm/m in the roof due to vertical live load only, at SLS. The free span moment is calculated assuming the roof slab to be simply supported over its effective span (X).

(c) The foundation settlement at SLS shall be taken to be the nominal live load settlement derived as in Clause 3.1.5.

4.2.4 Longitudinal Joints

(a) The structures shall be designed to accommodate all differential movements or to resist the forces set up by such movements.

(b) In precast construction the joints between the segments shall be designed to accommodate the anticipated settlements. The joints need not be designed to transfer load between the segments.

(c) In most cases, where cast in-situ construction is used, the structure acting as a deep beam is capable of accommodating curvatures induced by differential settlements and longitudinal joints should be avoided where possible, for reasons of durability. Where, however, the predicted movements are so large that articulation in the structure is required, the longitudinal joints shall be designed either:

(i) to accommodate all movements resulting from the differential settlement of the soil as well as the maximum differential live load deflection between sections which are not similarly loaded, or

(ii) to allow for the transfer of forces between units or sections joints. This shall be checked at both ULS and SLS.

4.3 ASPECTS OF REINFORCEMENT DESIGN

4.3.1 Primary Longitudinal Reinforcement (vertical, perpendicular to the walls or parallel to the edges of the structure).

The longitudinal reinforcement shall be provided to resist the moments and shears determined from the analysis in accordance with BS5400 Part 4.

4.3.2 Transverse Reinforcement in the Soffit of the Roof

Where the dispersed width of the applied load is less than the distance between joints (L_j), transverse reinforcement shall be provided in the soffit of the roof to resist the transverse bending caused by the local wheel effect. In this case, in the absence of a rigorous analysis, sufficient transverse soffit reinforcement shall be provided per metre to resist a moment equal to half the longitudinal sagging moment per metre caused by the vertical live load at ULS.

Where the dispersed width of the applied load is greater than or equal to L_j, the transverse reinforcement shall be in accordance with Clause 4.3.6.

4.3.3 Longitudinal Steel for Shear

The critical position for shear in a buried box or portal frame structure is frequently close to a point of contra-flexure, especially in the deck, so that, over a small distance, the tension face may change from one face of the member to the other while the shear force stays sensibly constant. In this case the value of the parameter $100A_s/bd$ (from BS5400 Part 4) used in the calculation of the shear resistance of the member shall be based on the area of longitudinal steel in the less heavily reinforced face.

4.3.4 Anchorage of Longitudinal Steel

Designers shall pay particular attention to the provision of adequate anchorage to the longitudinal reinforcement in accordance with BS5400 Part 4.

4.3.5 Reinforcement Details at Corners

(a) For Opening Moments (tension on the inside face)

(i) Where the bending moment applied to the corner of a buried structure causes that corner to open, the resolved component of the compressive and tensile bending forces in the members on either side of the corner produce a tensile force along the diagonal of the corner which tends to split the outer section of the corner from the main structure, as illustrated in Figure 4.1. As a result, the use of many conventional reinforcement details can lead to the flexural strength of the corner being significantly less than the strength of the adjacent members. These effects shall be taken into account in designing corner reinforcement.

The behaviour of opening corners is described by Somerville and Taylor, Nilsson and Losberg, Jackson and Noor (see Clause 6.4). A method for the design of corner reinforcement for opening moments is given in Appendix D.

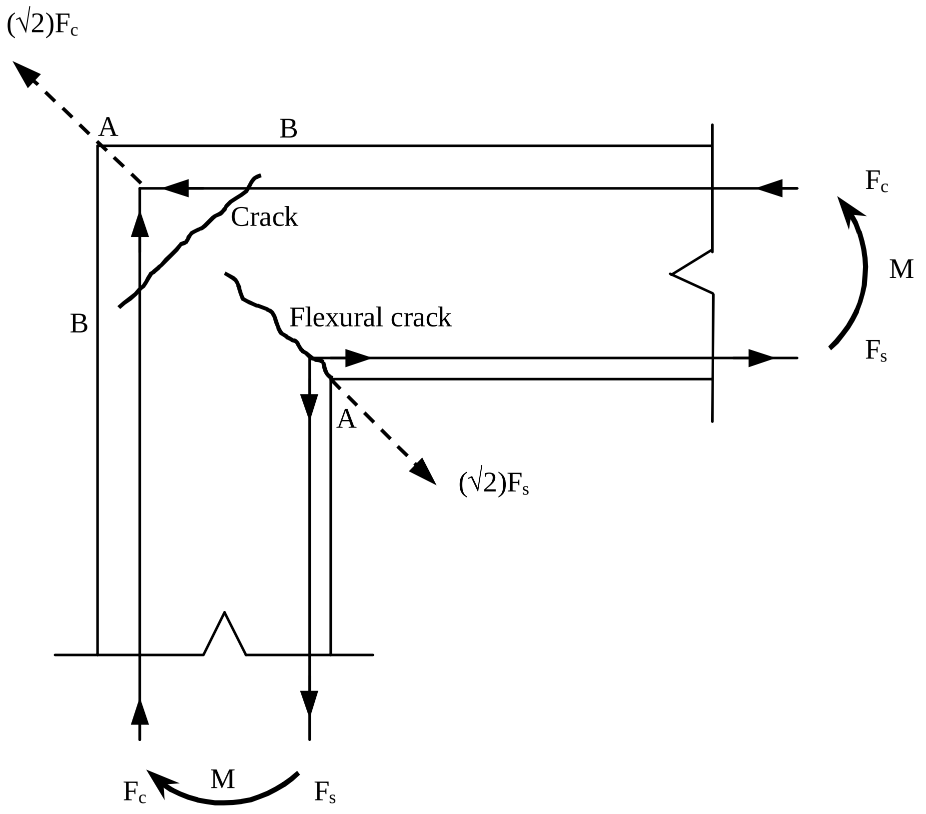


Figure 4.1: Cracking in an Opening Corner

(b) For Closing Moments (tension on the outside face)

(i) Where the applied moments tend to close a corner, the area of tension reinforcement provided around the outside of the corner to resist the peak corner moment may been determined on the assumption that the effective depth of the section, d_{cnr}, is the effective depth of the smaller adjacent member plus half the nominal fillet size.

(ii) Where a Type 2 corner detail is provided as shown in Figure D/1b in Appendix D, the area of the outside legs of either the vertical or the horizonal hairpin bars (but not both) may be considered as contributing to the moment of resistance providing they extend at least an anchorage length beyond the end of the fillet. Where the area of the horizontal and vertical hairpin bars differ, the smaller area should be used in the calculation.

(iii) Care should be taken to ensure that the bearing stress inside the bend of the corner bar does not exceed the limits allowed in BS5400 Part 4 (see Clause D.4 in Appendix D). For this purpose the mean radius of the bend may be increased to a value not exceeding d_{cnr}. If a radius in excess of this value is required to satisfy the bearing stress requirements, then the size of the fillet or the area of tensile reinforcement should be increased.

4.3.6 Minimum Areas of Reinforcement

The minimum area of reinforcement to be provided in the structural members shall be the greatest of any of the following:

Longitudinal steel on any tension face.	0.15% of the net section per face
Transverse steel on any tension face.	0.12% of the net section per face
Transverse steel on the soffit of the roof when the cover is less than 600mm.	0.15% of the net section but see Clause 4.3.2.
Vertical steel on an internal vertical face.	Enough vertical steel to resist a moment of not less than $0.8Y^3$ kNm/m at ULS
Any face.	Early thermal cracking steel in accordance with BD 28 (DMRB 1.3), except that transverse early thermal cracking steel is not required in members where L_j does not exceed 3m.

4.3.7 Cover to Reinforcement

(a) The nominal cover to reinforcement to be used for precast segments shall be in accordance with Table 13 in BS5400 Part 4.

(b) The nominal cover to be used for cast in-situ structures shall be based on Table 13 in BS5400 plus 10mm as specified in BD 57 (DMRB 1.3.7).

(c) Where the concrete is cast directly against the ground (as opposed to on blinding) the nominal cover shall be based on Table 13 in BS 5400 Part 4 plus a further 40mm.

(d) For cast in-situ concrete, where the surface is subject to flowing water the cover shall be increased by a further 10mm to allow for erosion.

4.3.8 Fatigue

(a) Fatigue due to repeated live loading need not be considered for structures where the cover depth (H) is more than one metre. For other structures the requirements are contained in BS5400 Part 4 as amended by Interim Advice Note IA.5.

(b) For cover depths greater than 0.6m the effective stress range in unwelded reinforcing bars under Load Combination 1 for the SLS shall be checked for 30 units HB loading instead of HA loading.

(c) The fatigue strength of tack welded reinforcing bars shall be checked in accordance with BA 40 (DRMB 1.3.4).

4.4 DESIGN OF FOUNDATIONS

4.4.1 Requirements

Consideration of sliding, global settlement and differential settlement is required to confirm whether or not the proposed geometry and structural form are suitable.

4.4.2 Sliding

(a) Requirements

Checks are required to ensure that the structure as a whole does not fail by sliding when subject to traction forces and/or skew effects. Furthermore, where the structure is supported on a number of individual foundations, as opposed to a single combined base slab, each individual foundation is required to remain stable (see paragraph (c) below).

(b) Sliding Resistance

This is a ULS check. The loads and partial load factors to be applied are as given in Clause 3.4.1 and Diagram A/6a in Appendix A for box structures and Diagram A/6b for portal frames.

The friction force (F_R) that can be developed on the base can be determined from BS8002 as follows:

$$F_R = V_{tot} \tan \delta_b$$

where

V_{tot} is the total applied vertical force on the footing due to permanent loads less any uplift due to Combination 4 loading and buoyancy.

δ_b is the design angle of base friction described in Clauses 2.2.8 and 3.2.6 of BS8002: 1994.

In the absence of tests δ_b may be determined from

$\tan \delta_b = 0.75 \tan \phi'$

where ϕ' is the design angle of shearing resistance of the material under the slab defined in Clause 3.2.5 of BS8002: 1994, which may be determined using Appendix B.

For a precast structure on granular bedding δ_b shall not be taken as greater than 20°.

No effective adhesion shall be taken between the base and foundation material.

(c) The following relationship shall be satisfied for the structure as a whole:

$$(\text{Traction} + \text{Disturbing Earth Pressure}).\ \gamma_{fL}.\ \gamma_{f3} < (\text{Relieving Earth Pressure} + F_R)$$

where F_R is the sliding resistance of the whole base for box structures, and the sum of the sliding resistances of the individual bases for portal frames.

The relieving earth pressure shall be based on a partial passive resistance coefficient, K_r, of 0.6, but in the event of the above criterion for sliding not being satisfied with this value of K_r, higher values of K_r, not exceeding $0.5K_p$, may be used if the structural elements are designed to carry the effects of these increased pressures.

In addition, portal frames should be checked to ensure that they are structurally capable of carrying the applied loads shown in Diagram A/6b in Appendix A in conjunction with the frictional resistance forces.

If it is necessary to increase the resistance against sliding this may be achieved by extending the footings of the abutments into the backfill to mobilise a greater weight of fill, or by extending the walls below the underside of the base.

(d) Rotational Sliding

Rotational sliding tends to occur with skewed in-situ structures as discussed in Clause 4.1.2(d) and also with eccentric traction. Checks are required to ensure that the structure as a whole does not fail by rotational sliding when subject to eccentric traction forces and/or skew effects.

4.4.3 Bearing Pressure

(a) The design shall ensure that the maximum net bearing pressure under the foundations due to nominal loadings does not exceed the allowable net bearing pressure determined in accordance with BS8004 as implemented by BD74 (DRMB 2.1.8). The purpose of this check is to ensure that there is an adequate factor of safety against failure of the founding soil and that settlements are kept within acceptable limits.

(b) The structure shall be able to accommodate any settlements either through movements at the structural joints or through adequate structural strength. In the latter case the stresses arising from differential settlement shall be considered at ULS and SLS as a Combination 1 load using the partial factors of safety given in Table 3.2. (see also Clause 4.2.4).

5. MATERIALS AND CONSTRUCTION

5.1 EXCAVATION

5.1.1 Trench Condition

(a) For a precast structure, excavation in materials other than hard material (see definitions) shall extend to at least 200mm for a granular bedding, or to at least 125mm for a concrete blinding with a granular overlay, below the base of the structure. The excavation shall extend at least 300mm beyond the outside wall faces.

(b) For a cast in-situ structure, excavation in materials other than hard material (see definitions) shall extend to at least 75mm below the base of the structure and shall extend at least 300mm beyond the outside wall faces.

(c) Excavation in hard material (see definitions) shall extend to at least 125mm for a precast structure, or 75mm for a cast in-situ structure, below the base of the structure, and at least 225mm beyond the outside wall faces.

(d) For both types of structure, excavation shall, where possible, be benched to a slope no steeper than 1.0 horizontally to 1.0 vertically to a height of not less than 500mm above the top of the structure or to the carriageway formation level, whichever is lower. Where the side slopes are steeper and in close proximity to the finished structure, consideration shall be given to the effects the native ground might have on horizontal earth pressures and thus to the earth pressure parameters that are appropriate for design. (See Clause 3.1.3(c)).

5.1.2 Embankment Condition

In the embankment situation, when the embankment is built before the structure, the embankment fill shall be benched to a slope no steeper than 0.6 horizontally to 1.0 vertically to a height of not less than 500mm above the top of the structure or to the carriageway formation level, whichever is the lower. When the structure is backfilled before or at the same time as the construction of the embankment, in addition to the above benching requirement the top of the backfilling shall have the same width as that required when the embankment is built first.

5.2 BLINDING AND BEDDING

(a) All box structures shall be founded on a suitably prepared blinding or bedding layer that shall extend at least 300mm, or 225mm when the excavation is in hard material, beyond the outside wall faces of the structure.

(b) Cast in-situ structures shall be constructed on a blinding layer of Class 20/20 concrete, as described in MCHW1 Series 1700, with a minimum thickness of 75mm.

(c) Precast units founded on other than hard material shall be laid on either a two layer granular bed which shall have a minimum thickness of 200mm or a concrete bed with a granular overlay. In the granular bed the lower 150mm shall be of selected well-graded Class 6N material and the upper 50mm shall be of Class 6L material as described in MCHW1 Series 600, except that for Class 6L, only the grading requirement applies and not the other material properties listed in Table 6/1 of MCHW1 (but the sulphate requirements of Clause 601 still apply). Alternatively the lower 150mm may be replaced by a 75mm minimum blinding concrete layer as described in (b) above.

(d) Precast units founded on hard material shall be laid on a two layer bed consisting of a lower 75mm minimum blinding concrete layer as described in (b) above covered by an upper 50mm minimum granular layer of Class 6L as described in (c) above.

5.3 FILLING AND COMPACTION

Backfilling for either trench or embankment condition shall be in accordance with Clause 610 (Fill to Structures) of MCHW1 except that Classes 7A and 7B shall not be used. The backfilling material shall be used to a height of 500mm above the structure or to the carriageway formation level, whichever is lower.

5.4 REINFORCED AND PRESTRESSED CONCRETE

The Overseeing Organisation's requirements for reinforced and prestressed concrete are contained in Series 1700 of MCHW1. The concrete mix shall be Grade 40 or higher. In Scotland, all structural concrete above foundation level shall be air entrained. Further guidance is given in Series NG1700 of Notes for Guidance on the Specification for Highway Works MCHW2. Detailed guidance on the assessment of ground aggressivity to concrete is given in Part 1 of BRE Special Digest SD1 "Concrete in aggressive ground" (2001). The ground to be assessed should include the surrounding ground, the groundwater, the general embankment fill and the backfill material, and any contained water or effluent to be carried by the structure. Detailed guidance on the specification of concrete for foundations in aggressive ground is given in parts 2 and 3 of BRE Special Digest SD1. Provision is made in the Digest for the use of various cement and cement combinations in conjunction with aggregates of differing carbonate content. The Digest also recommends additional protective measures for concrete where ground water is mobile and sulfate concentrations are high.

5.5 WATERPROOFING

(a) For both precast and in-situ construction, the top surface, and the top of the adjoining vertical external surfaces to a level of 200mm below the soffit of the top slab, shall be protected with a suitable bridge deck waterproofing system in accordance with MCHW1 Series 2000. The Overseeing Organisation's requirements for the waterproofing system are contained in MCHW1 Clause 2003, the preparation of the concrete surfaces to receive the waterproofing system, and the laying of the waterproofing system are contained in MCHW1 Clause 2005.

(b) For precast construction only, all other concrete surfaces of the box structure in contact with soil, backfill, or bedding shall be waterproofed in accordance with the requirements for Below Ground Concrete Surfaces. The Overseeing Organisation's requirements for waterproofing these surfaces are contained in MCHW1 Clauses 2004 and 2006.

(c) For in-situ construction all other concrete surfaces in contact with soil or backfill shall be waterproofed in accordance with the requirements for Below Ground Concrete Surfaces as given in MCHW1 Clauses 2004 and 2006.

5.6 PERMEABLE DRAINAGE LAYER

A permeable drainage layer in accordance with Clause 513 (Permeable Backing to Earth Retaining Structures) of MCHW1 shall be provided adjacent to all vertical buried concrete faces of box structures which do not carry water or effluent. A perforated drainage pipe, not less than 150mm diameter, with adequate facilities for rodding shall be incorporated at the bottom of the drainage layer. This drainage pipe shall be connected to a positive outfall. All drainage to comply with the requirements of MCHW1, Series 500.

5.7 JOINTS

(a) Joints in structures which are to be maintained in a dry condition internally, such as subways, shall be made watertight through the use of a continuous waterstop or by placing a suitable hydrophillic swellable waterstop or compression sealant strip in the joints and/or pointing internally with elastomeric or bitumen-based sealant where the units are of sufficient size to allow entry. In the case of precast segmental units sealant strips shall be placed in the joints prior to the butting of adjacent box segments. These provisions are also applicable when groundwater is to be protected from any leakage of the effluent carried by the structure or where the leakage of water or effluent carried by the structure could lead to erosion of the backfill. For all structures any gaps in the joints at both internal and external surfaces, where not covered by a waterproofing membrane, or concrete blinding, shall be filled with a flexible sealant, to prevent the entry of backfilling materials and other debris into the joints.

(b) A supporting compressible joint filler shall be inserted prior to laying the waterproofing membrane or flexible sealant.

(c) The Overseeing Organisations' requirements for the sealing of gaps and joints are given in MCHW1 series 2300.

Note: in-situ structures should be jointless unless there are unavoidable construction reasons.

5.8 SCOUR PROTECTION

In the case of culverts the design shall contain adequate provision for preventing scour at the inlet and outlet of the structure. This may include the use of cut off walls below the base slab level and lateral training walls.

6. REFERENCES

6.1 DESIGN MANUAL FOR ROADS AND BRIDGES

Volume 1: Section 1 Approval Procedures

BD 2: Part 1 - Technical Approval of Highway Structures (DMRB 1.1)

Volume 1: Section 3 General Design

BD 24: Design of Concrete Highway Bridges and Structures, Use of BS5400 Part 4 (as amended by Interim Advice Note IA.5) (DMRB 1.3.1)

BD 28: Early Thermal Cracking of Concrete [and Amendment No 1] (DMRB 1.3)

BD 37: Loads for Highway Bridges (DMRB 1.3)

BD 57: Design for Durability (DMRB 1.3.7)

BA 40: Fatigue Strength of Tack Welded Reinforcing Bars (DMRB 1.3.4)

BA 57: Design for Durability (DMRB 1.3.8)

Volume 2: Section 1 Substructures

BD 74: Foundations: Use of BS8004: 1986 (DMRB 2.1.8)

Volume 10: Environmental Design

6.2 MANUAL OF CONTRACT DOCUMENTS FOR HIGHWAY WORKS

Volume 0: Section 2 Implementing Standards

SD4: Procedures for Adoption of Proprietary Manufactured Structures (MCHW 0.2.4)

Volume 1: Specification for Highway Works (MCHW1) HMSO 1998 with revisions to 2001

Volume 2: Notes for Guidance on the Specification for Highway Works (MCHW2) HMSO 1998 with revisions to 2001

Volume 4: Bills of Quantities for Highway Works (MCHW4) HMSO 1998 with revisions to 2001

6.3 BRITISH STANDARDS

BS5400: Steel, Concrete and Composite Bridges.
Part 2: 1978: Specification for Loads.
Part 4: 1990: Code of Practice for Design of Concrete Bridges

BS8002: 1994 Code of Practice for Earth Retaining Structures

BS8004: 1986 Foundations

BS8110: 1997 Structural Use of Concrete

6.4 BIBLIOGRAPHY

BRE Special Digest SD1. Concrete in aggressive ground. (2001).

Jackson, N. “Design of Reinforced Concrete Opening Corners”, The Structural Engineer, Vol. 73, No. 13, pp209-213, 1995.

Jackson, N. “Design of Reinforced Concrete Corners with Diagonals”, The Structural Engineer, Vol. 75, Nos. 23 & 24, pp417-420, 1997.

Nilsson, I.H.E and Losberg, A., “Design of Reinforced Concrete Corners and Joints Subjected to Bending Moments”, ASCE National Structural Engineering Convention, New Orleans, US, April 14-18, 1975.

Noor, F.A. [1977], “Ultimate Strength and Cracking of Wall Corners”, Concrete, Vol. 11, No. 7. pp 31-35, 1977.

Pucher, A., “Influence Surfaces of Elastic Plates”, Springer-Verlag Wien, New York, 1977.

Somerville, G. and Taylor, H.P.J., “The Influence of Reinforcement Detailing on the Strength of Concrete Structures”, The Structural Engineer, Vol. 50, No. 1, January 1972, pp 7-19, and discussion in The Structural Engineer, Vol. 50, No. 8, August 1972, pp 309-321.

7. ENQUIRIES

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

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Chief Road Engineer Scottish Executive Development Department Victoria Quay Edinburgh EH6 6QQ	J HOWISON Chief Road Engineer

Chief Highway Engineer The National Assembly for Wales Cynulliad Cenedlaethol Cymru Crown Buildings Cathays Park Cardiff CF10 3NQ	J R REES Chief Highway Engineer

Director of Engineering Department for Regional Development Roads Service Clarence Court 10-18 Adelaide Street Belfast BT2 8GB	G W ALLISTER Director of Engineering

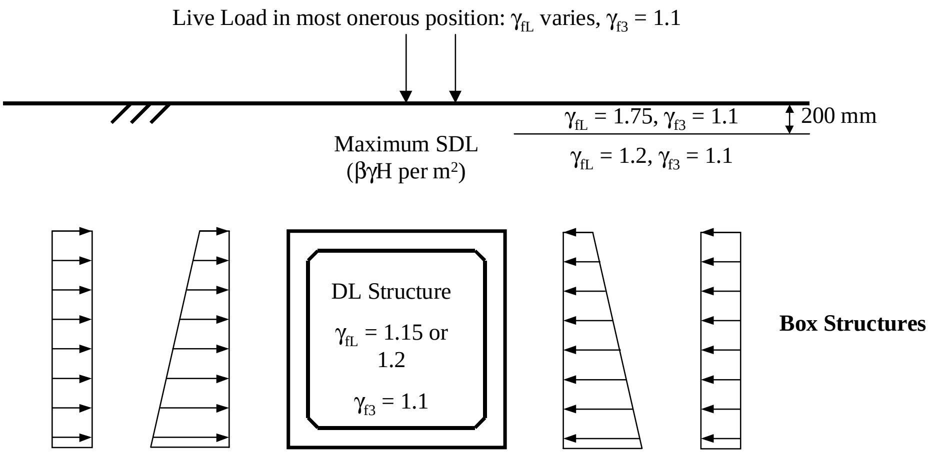
APPENDIX A - DIAGRAMS SHOWING LOAD CASES TO BE CONSIDERED

The diagrams give a graphical representation of the load cases to be applied and the associated partial load factors to be used with these load cases. The diagrams are not fully comprehensive and should be read in conjunction with Table 3.2 and Chapters 3 and 4.

Each diagram is presented in two sections, the upper section relating to closed box structures and the lower to portal structures. The diagrams are also relevant to multi-celled boxes and multi-span structures on individual foundations, respectively.

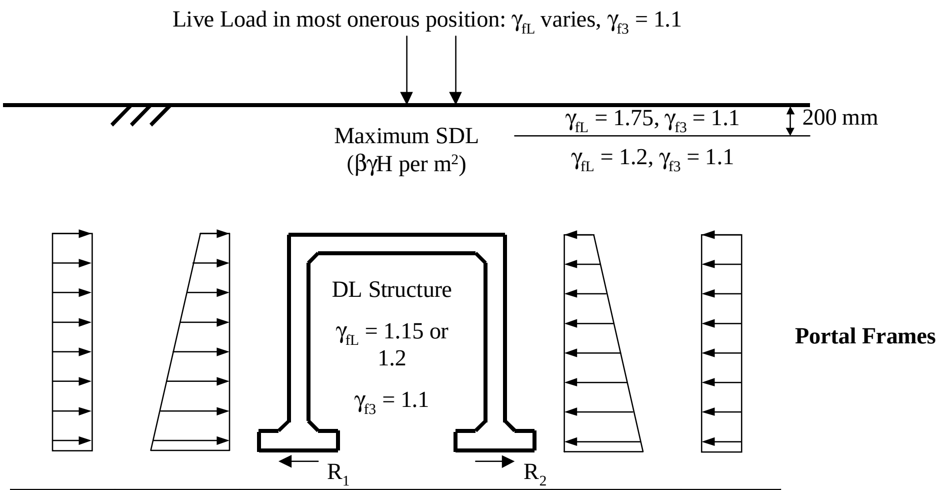
It should be noted that where γ_{f3} is shown as equal to 1.1 on the diagrams, it should be increased to 1.15 when the design is being carried out by plastic analysis (see Clause 3.3.3(b)(ii)).

In diagrams A/1 to A/5, for portal frames R_1 and R_2 should be calculated in accordance with Clause 4.1.1(g).



LLSC	Earth		Earth	LLSC
K = 0.6 γ_{fL} = 1.5 γ_{f3} = 1.1	K = 0.6 γ_{fL} = 1.5 γ_{f3} = 1.1	Default values	K = 0.6 γ_{fL} = 1.5 γ_{f3} = 1.1	K = 0.6 γ_{fL} = 1.5 γ_{f3} = 1.1
K = K_o γ_{fL} = 1.5 γ_{f3} = 1.1	K = K_o γ_{fL} = 1.2 γ_{f3} = 1.1	BS 8002 coefficients	K = K_o γ_{fL} = 1.2 γ_{f3} = 1.1	K = K_o γ_{fL} = 1.5 γ_{f3} = 1.1

Diagram A/1a

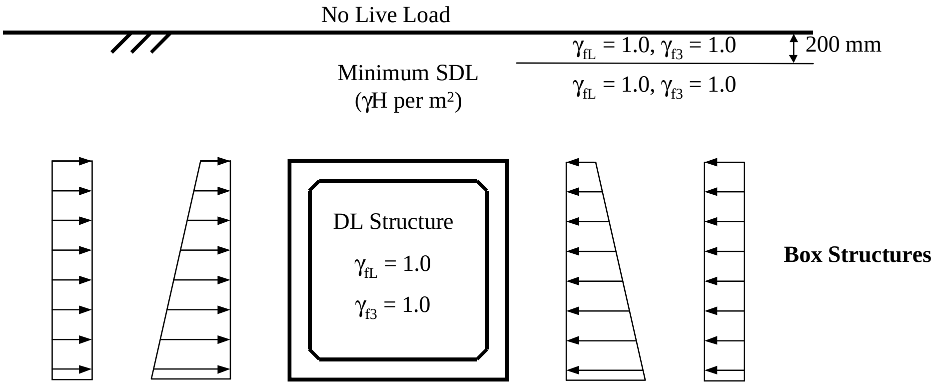


LLSC	Earth		Earth	LLSC
K = 0.6 γ_{fL} = 1.5 γ_{f3} = 1.1	K = 0.6 γ_{fL} = 1.5 γ_{f3} = 1.1	Default values	K = 0.6 γ_{fL} = 1.5 γ_{f3} = 1.1	K = 0.6 γ_{fL} = 1.5 γ_{f3} = 1.1
K = K_o γ_{fL} = 1.5 γ_{f3} = 1.1	K = K_o γ_{fL} = 1.2 γ_{f3} = 1.1	BS 8002 coefficients	K = K_o γ_{fL} = 1.2 γ_{f3} = 1.1	K = K_o γ_{fL} = 1.5 γ_{f3} = 1.1

Diagram A/1b

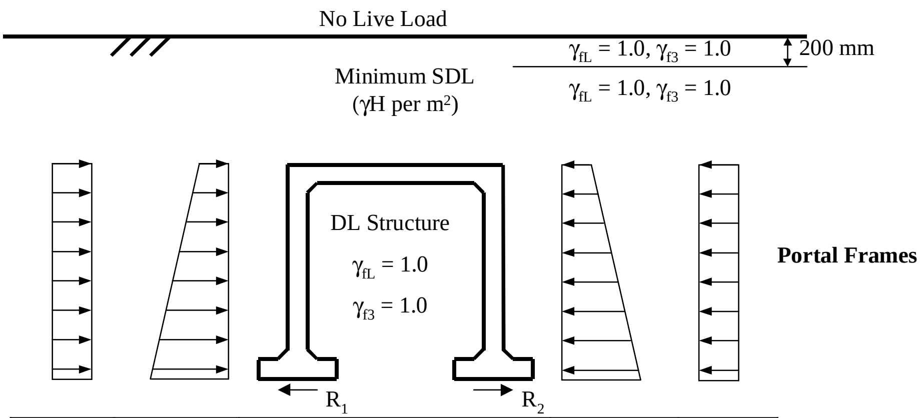
MAXIMUM VERTICAL LOAD WITH MAXIMUM HORIZONTAL LOAD

Diagram A/1



LLSC	Earth		Earth	LLSC
K = 0.6 γ_{fL} = 1.5 γ_{f3} = 1.1	K = 0.6 γ_{fL} = 1.5 γ_{f3} = 1.1	Default values	K = 0.6 γ_{fL} = 1.5 γ_{f3} = 1.1	K = 0.6 γ_{fL} = 1.5 γ_{f3} = 1.1
K = K_o γ_{fL} = 1.5 γ_{f3} = 1.1	K = K_o γ_{fL} = 1.2 γ_{f3} = 1.1	BS 8002 coefficients	K = K_o γ_{fL} = 1.2 γ_{f3} = 1.1	K = K_o γ_{fL} = 1.5 γ_{f3} = 1.1

Diagram A/2a

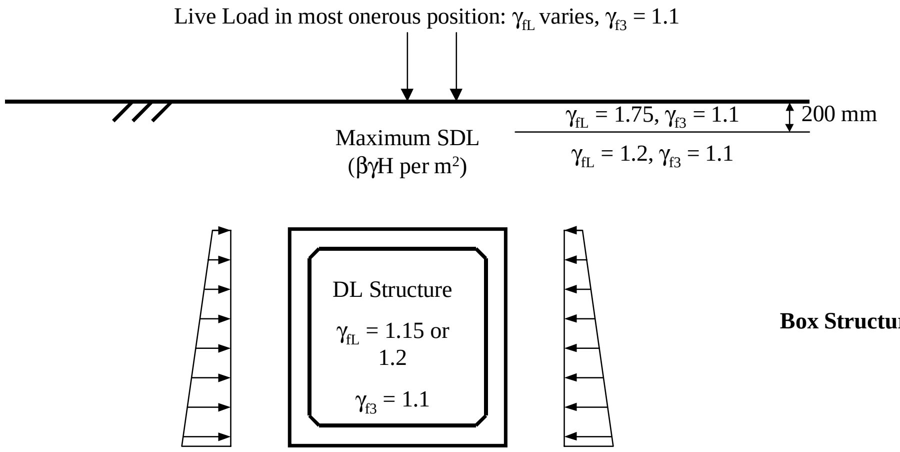


LLSC	Earth		Earth	LLSC
K = 0.6 γ_{fL} = 1.5 γ_{f3} = 1.1	K = 0.6 γ_{fL} = 1.5 γ_{f3} = 1.1	Default values	K = 0.6 γ_{fL} = 1.5 γ_{f3} = 1.1	K = 0.6 γ_{fL} = 1.5 γ_{f3} = 1.1
K = K_o γ_{fL} = 1.5 γ_{f3} = 1.1	K = K_o γ_{fL} = 1.2 γ_{f3} = 1.1	BS 8002 coefficients	K = K_o γ_{fL} = 1.2 γ_{f3} = 1.1	K = K_o γ_{fL} = 1.5 γ_{f3} = 1.1

Diagram A/2b

MINIMUM VERTICAL LOAD WITH MAXIMUM HORIZONTAL LOAD

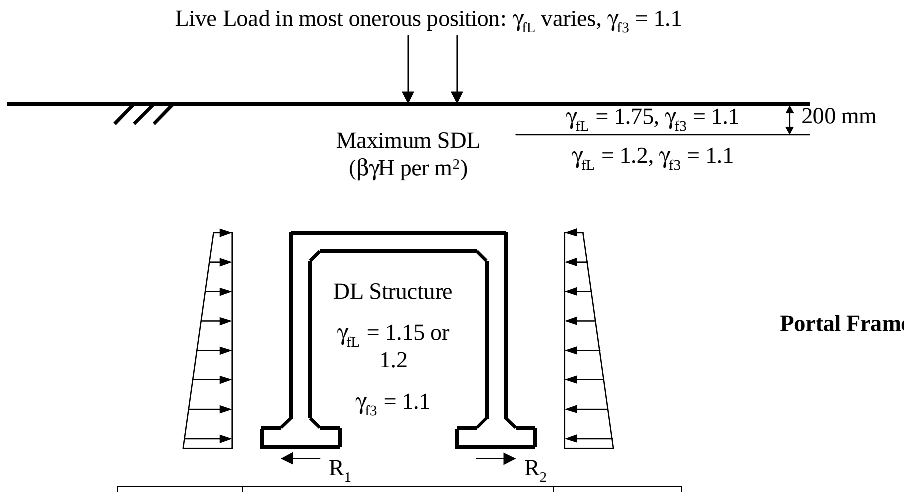
Diagram A/2



Box Structures

Earth		Earth
K = 0.2 γ_{fL} = 1.0 γ_{f3} = 1.0	Default values	K = 0.2 γ_{fL} = 1.0 γ_{f3} = 1.0
K = 0.2 γ_{fL} = 1.0 γ_{f3} = 1.0	BS 8002 coefficients	K = 0.2 γ_{fL} = 1.0 γ_{f3} = 1.0

Diagram A/3a



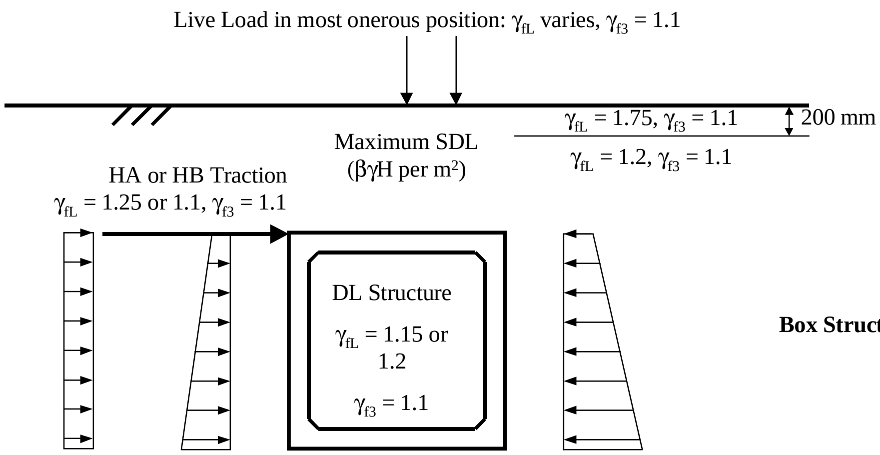
Portal Frames

Earth		Earth
K = 0.2 γ_{fL} = 1.0 γ_{f3} = 1.0	Default values	K = 0.2 γ_{fL} = 1.0 γ_{f3} = 1.0
K = 0.2 γ_{fL} = 1.0 γ_{f3} = 1.0	BS 8002 coefficients	K = 0.2 γ_{fL} = 1.0 γ_{f3} = 1.0

Diagram A/3b

MAXIMUM VERTICAL LOAD WITH MINIMUM HORIZONTAL LOAD

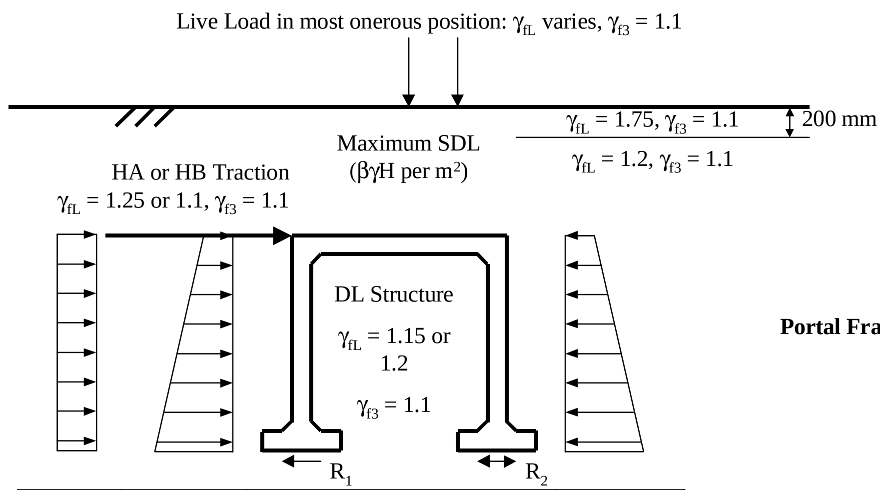
Diagram A/3



Box Structures

LLSC	Earth		Earth
K = 0.33 γ_{fL} = 1.5 γ_{f3} = 1.1	K = 0.33 γ_{fL} = 1.5 γ_{f3} = 1.1	Default values	K = 0.6 γ_{fL} = 1.0 γ_{f3} = 1.0
K = K_a γ_{fL} = 1.5 γ_{f3} = 1.1	K = K_a γ_{fL} = 1.2 γ_{f3} = 1.1	BS 8002 coefficients	K = 0.6 γ_{fL} = 1.0 γ_{f3} = 1.0

Diagram A/4a



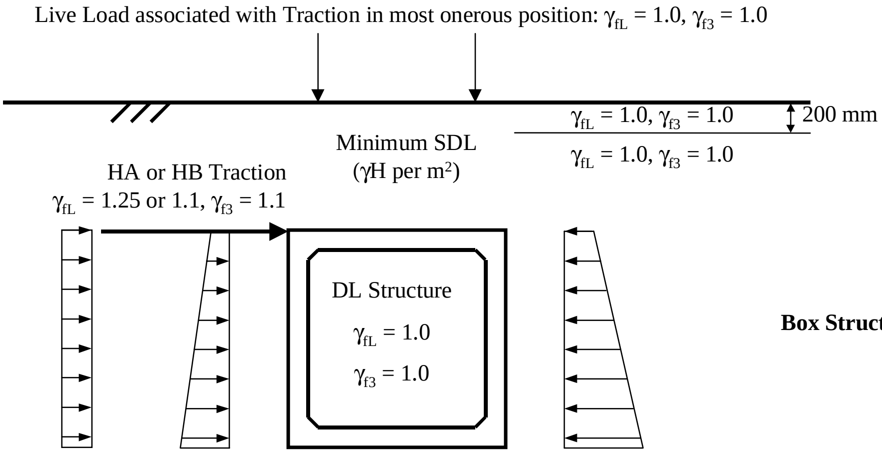
Portal Frames

LLSC	Earth		Earth
K = 0.33 γ_{fL} = 1.5 γ_{f3} = 1.1	K = 0.33 γ_{fL} = 1.5 γ_{f3} = 1.1	Default values	K = 0.6 γ_{fL} = 1.0 γ_{f3} = 1.0
K = K_a γ_{fL} = 1.5 γ_{f3} = 1.1	K = K_a γ_{fL} = 1.2 γ_{f3} = 1.1	BS 8002 coefficients	K = 0.6 γ_{fL} = 1.0 γ_{f3} = 1.0

Diagram A/4b

TRACTION WITH MAXIMUM VERTICAL LOAD

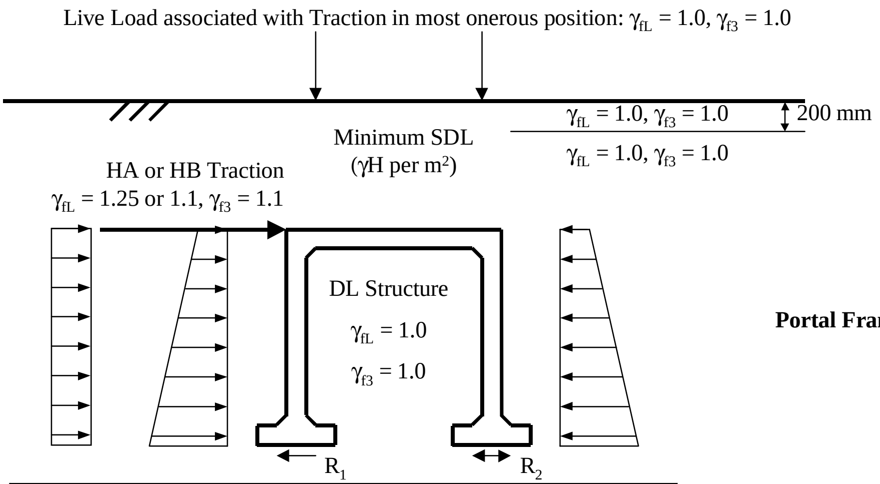
Diagram A/4



Box Structures

LLSC	Earth		Earth
K = 0.33 γ_{fL} = 1.5 γ_{f3} = 1.1	K = 0.33 γ_{fL} = 1.5 γ_{f3} = 1.1	Default values	K = 0.6 γ_{fL} = 1.0 γ_{f3} = 1.0
K = K_a γ_{fL} = 1.5 γ_{f3} = 1.1	K = K_a γ_{fL} = 1.2 γ_{f3} = 1.1	BS 8002 coefficients	K = 0.6 γ_{fL} = 1.0 γ_{f3} = 1.0

Diagram A/5a



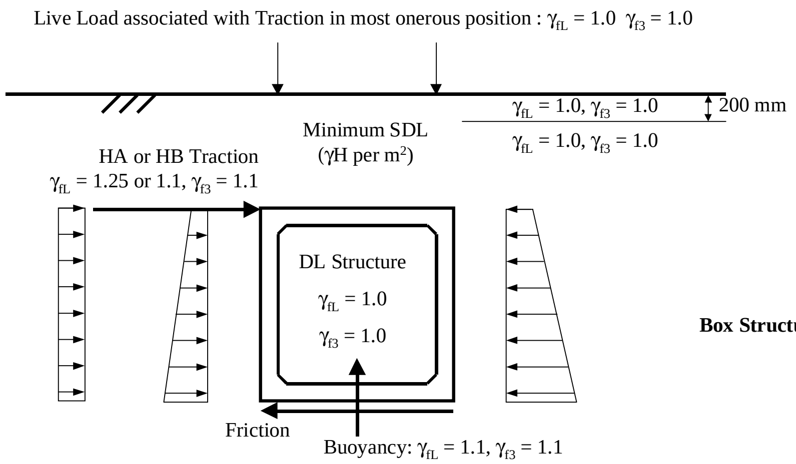
Portal Frames

LLSC	Earth		Earth
K = 0.33 γ_{fL} = 1.5 γ_{f3} = 1.1	K = 0.33 γ_{fL} = 1.5 γ_{f3} = 1.1	Default values	K = 0.6 γ_{fL} = 1.0 γ_{f3} = 1.0
K = K_a γ_{fL} = 1.5 γ_{f3} = 1.1	K = K_a γ_{fL} = 1.2 γ_{f3} = 1.1	BS 8002 coefficients	K = 0.6 γ_{fL} = 1.0 γ_{f3} = 1.0

Diagram A/5b

TRACTION WITH MINIMUM VERTICAL LOAD

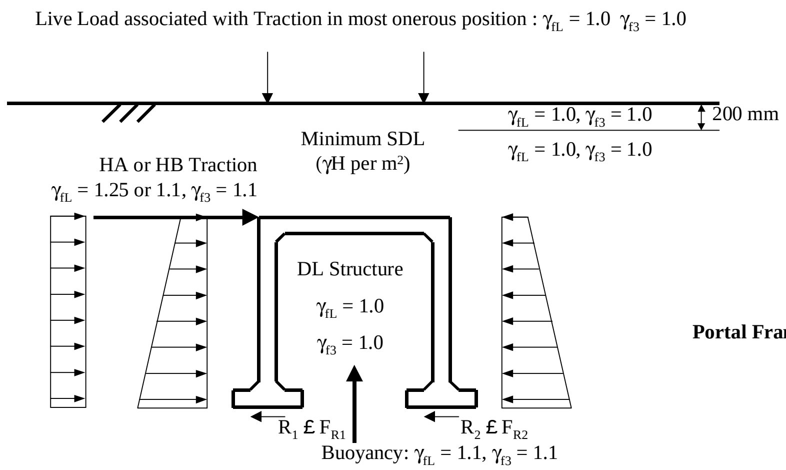
Diagram A/5



Box Structures

LLSC	Earth		Earth	
K = 0.33 γ_{fL} = 1.5 γ_{f3} = 1.1	K = 0.33 γ_{fL} = 1.5 γ_{f3} = 1.1	Default values	K_r < 1.5 γ_{fL} = 1.0 γ_{f3} = 1.0	See clause 4.4.2 (c)
K = K_a γ_{fL} = 1.5 γ_{f3} = 1.1	K = K_a γ_{fL} = 1.2 γ_{f3} = 1.1	BS 8002 coefficients	K_r < 0.5K_p γ_{fL} = 1.0 γ_{f3} = 1.0	See clause 4.4.2 (c)

Diagram A/6a



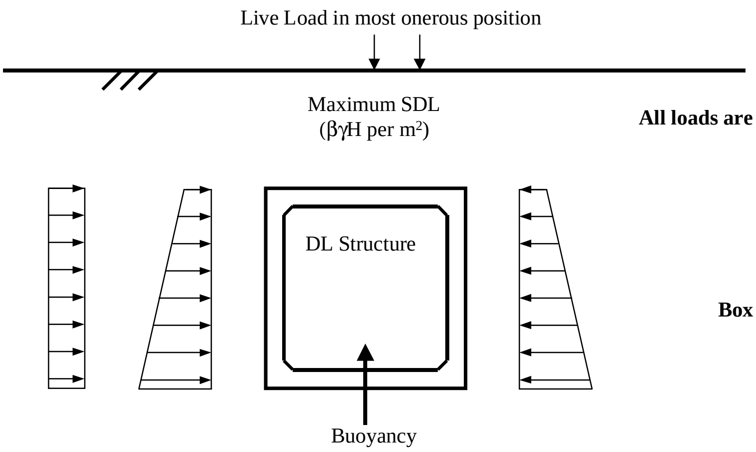
Portal Frames

LLSC	Earth		Earth	
K = 0.33 γ_{fL} = 1.5 γ_{f3} = 1.1	K = 0.33 γ_{fL} = 1.5 γ_{f3} = 1.1	Default values	K_r < 1.5 γ_{fL} = 1.0 γ_{f3} = 1.0	See clause 4.4.2 (c)
K = K_a γ_{fL} = 1.5 γ_{f3} = 1.1	K = K_a γ_{fL} = 1.2 γ_{f3} = 1.1	BS 8002 coefficients	K_r < 0.5K_p γ_{fL} = 1.0 γ_{f3} = 1.0	See clause 4.4.2 (c)

Diagram A/6b

SLIDING

Diagram A/6

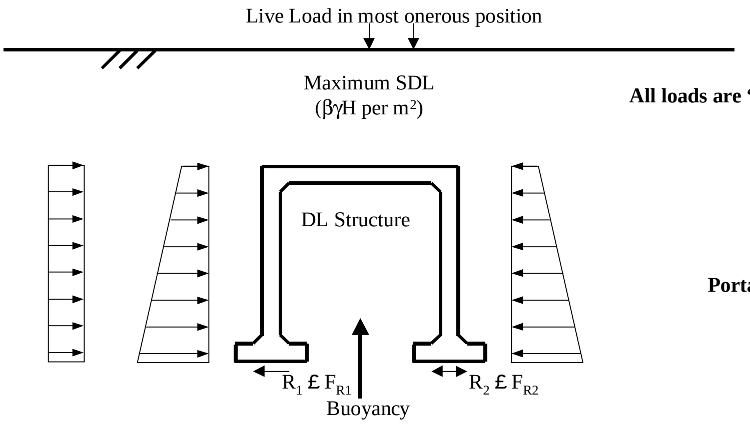


All loads are "nominal"

Box Structures

LLSC	Earth		Earth
$K = 0.6$	$K = 0.6$	Default values	$K = 0.6$
$K = K_o$	$K = K_o$	BS 8002 coefficients	$K = K_o$

Diagram A/7a



All loads are "nominal"

Portal Frames

LLSC	Earth		Earth
$K = 0.6$	$K = 0.6$	Default values	$K = 0.6$
$K = K_o$	$K = K_o$	BS 8002 coefficients	$K = K_o$

Diagram A/7b

BEARING PRESSURE (NOMINAL LOADS)

Diagram A/7

APPENDIX B - CALCULATION OF BACKFILL PRESSURES IN ACCORDANCE WITH BS8002

B.1 The backfill materials permitted by this standard are granular and cohesionless as described in Chapter 5.

B.2 In BS8002 the earth pressure coefficients K_a, K_o and K_p are related to ϕ'_{crit} and ϕ'_{max}. However, ϕ'_{max} is dependent on the SPT which, in most cases is not known. In this case the pressure coefficients may be based on ϕ'_{crit} alone as follows:

Calculate the “representative value” of ϕ'_{crit} from BS8002 Clause 2.2.4.

Take the design value of ϕ' equal to ϕ'_{crit}

K_o is given by:

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

Take K_a and K_p from Figures A/1 and A/2 in Annex A to BS8002 using an angle of wall friction δ, not exceeding 20^o and interpolating for δ/ϕ'.

Alternatively if wall friction is to be ignored ($\delta = 0$), K_a and K_p can be calculated as follows:

$$K_a = (1 - \sin \phi')/(1 + \sin \phi')$$
$$K_p = 1/K_a$$

Earth pressures based on the above values of K_a, K_o and K_p are nominal

B.3 If however the backfill is already in place (in an assessment for example), or if for some other reason the SPT value of the backfill is known, the procedure for calculating the earth pressure coefficients is as follows:

Calculate the “representative value” of ϕ'_{crit} and ϕ'_{max} from BS8002 clause 2.2.4 and Table 3.

Take the design value of tan ϕ' equal to the lesser of tan ϕ'_{max} /1.2 or tan ϕ'_{crit}

Thus find ϕ'

Calculate K_a, K_o and K_p using ϕ' as described in B.2 above.

APPENDIX C - CALCULATION OF MOMENTS AND SHEARS DUE TO DIFFERENTIAL TEMPERATURE EFFECTS

C.1 In buried box and portal frame structures Differential Temperature is considered to occur in the roof slab only.

C.2 Fixed End Moments In Roof

If the roof slab was encastre at both ends and subject to the differential temperatures specified in BD 37, Figure 9, Group 4, the nominal "fixed end moments", M_f, occurring in the roof (based on $\alpha = 12 \times 10^{-6}$ and Youngs modulus = 32 kN/mm^2) would be as shown in Table C1.

Roof Thickness h (mm)	200	400	600	800
M_f with Positive Temperature Difference (kNm/m)	11	50	104	154
M_f with Reverse Temperature Difference (kNm/m)	-1	-4	-12	-21

Encastre Differential Temperature Moment, M_f
based on BD 37, Figure 9, Group 4

TABLE C1

C.3 Single Span Integral Structures

If the fixed end moment restraints are then relaxed, the resulting nominal moments in single span integral structures may conservatively be taken as follows:

Roof Slab: $M_{roof} = M_f /(1+\lambda)$

where $\lambda = 0.5\ (h/h_w)^3\ (Y/X)$

Base Slab: $M_{base} = -M_{roof}/2$

Walls: Vary linearly from M_{roof} at the top to M_{base} at the bottom.

Positive moments are those which cause tension on the inner face of the member.

C.4 Other Structures

For other structures the differential temperature moments should be calculated from M_f using moment distribution, or by other rigorous methods.

C.5 Reduction Factors

Where $X_{clear} < 0.2L_t$ the moments and shears found from the above analyses should be multiplied by the reduction factor η given in Table 3.1 and Clause 3.2.8.

APPENDIX D - DESIGN OF CORNERS FOR OPENING MOMENTS

D.1 As described in Clause 4.3.5(a), the use of some conventional reinforcement details can lead to the flexural strength of opening corners (i.e. those where the applied moment produces tension on the inside face) being significantly less than the strength of the adjacent members. The following method may be used for the design of reinforcement for opening corners.

D.2 The tension reinforcement on the inside face of the members on either side of an opening corner should be extended across the full width of the corner and anchored using U-bars or "hairpin" bars in the Type 1 or Type 2 configurations shown in Figure D/1a and b. For the Type 2 corner detail the legs of the U-bars or "hairpin" bars in the outer face should be extended at least 20 bar diameters beyond the face of the adjacent member as shown in Figure D/1b. This dimension may need to be increased if the bar is also considered to form part of the tension steel for a closing moment (see Clause 4.3.5(b)(ii)).

D.3 Fillet bars should be provided to resist the peak opening corner moment, based on an effective depth d_f as shown in Figure D/2, which assumes that a crack has developed along BB and the concrete beyond it is ineffective. The area of the fillet bars should not be less then half the area the main inner face tension bars in either adjacent member.

D.4 In the design of tension reinforcement to resist corner moments, the maximum tension that may be considered to exist in any bar under ultimate load at a distance L_{st} from the start of a bend should not exceed $f_b \pi \phi L_{st} + F_{bt}$ or f_y/γ_{ms} where:

> f_b is the allowable bond stress in accordance with BS5400 Part 4.
>
> ϕ is the bar diameter.
>
> F_{bt} is the allowable tensile force in a bend in a bar in respect of the bearing stress inside that bend, as given in Clause 5.8.6.9 of BS5400 Part 4.
>
> f_y is the characteristic strength of the reinforcement

D.5 When a Type 1 or Type 2 corner detail is used, the resolved component of the tension in the vertical and horizontal U-bars or "hairpin" bars may be taken to supplement the fillet bars in resisting the corner moment as shown in Figure D/2 provided the tensions in the bars are limited in accordance with Clause D.4.

D.6 When for practical reasons it is essential to have a buried structure without a fillet, inclined "fillet bars" should be provided inside the corner (if there is an opening moment) and the corner should be designed as above assuming the fillet has zero size.

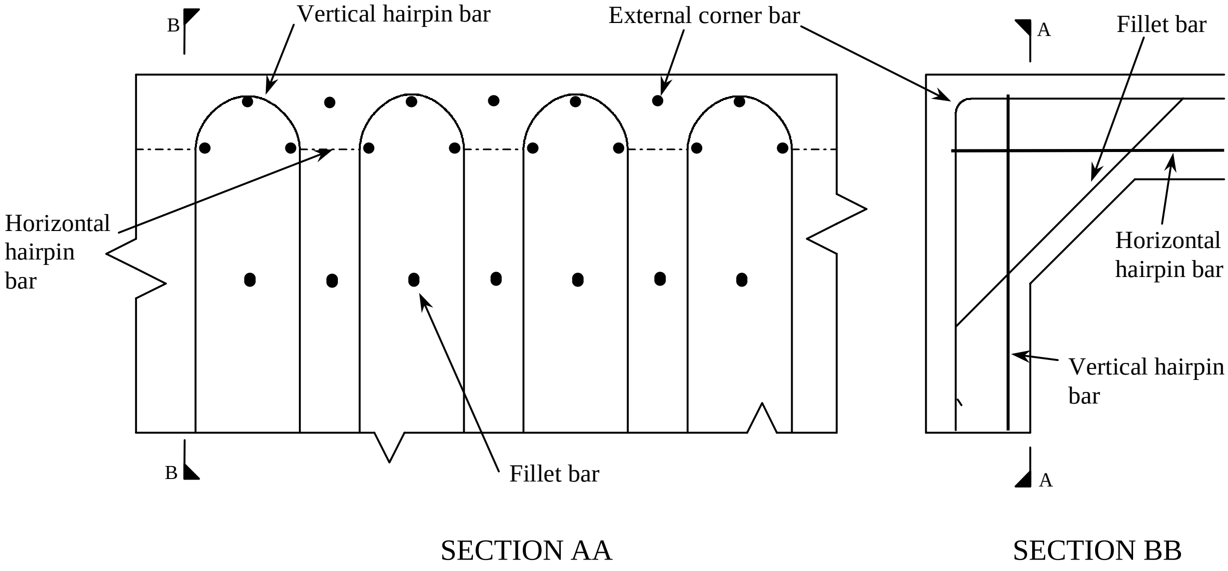
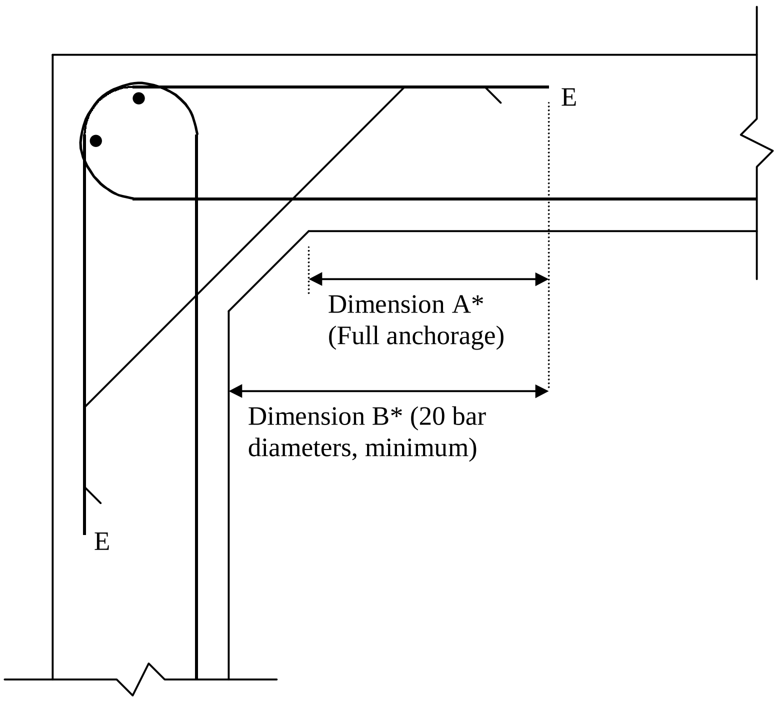
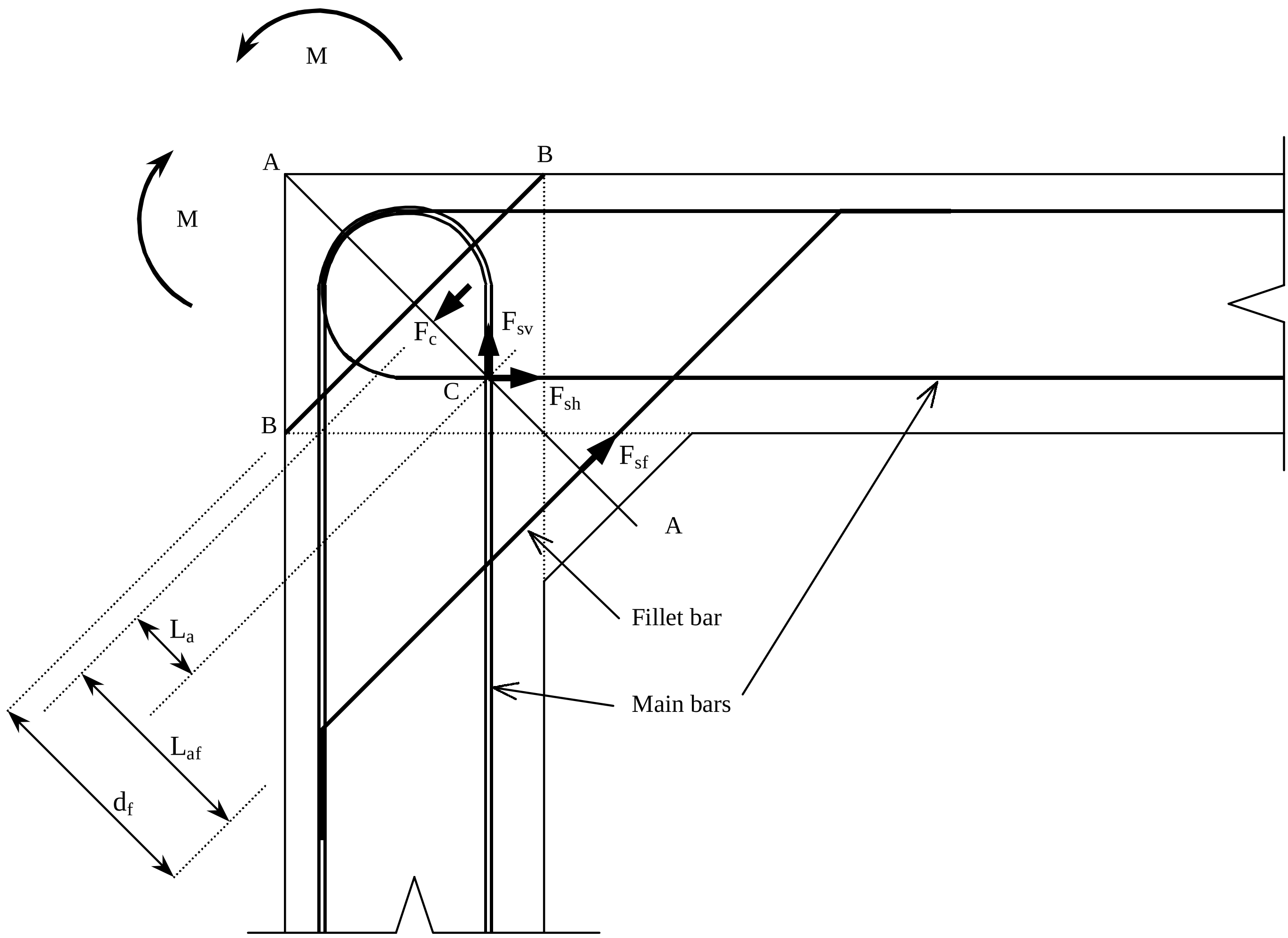


Figure D/1a: Type 1 Corner Detail



* If the outer leg of the U-bar or "hairpin" is used to contribute to the tension steel of the closing corner moment the location of the end of the leg, E, is determined by Dimension A (as Clause 4.3.5 (b)(iv)). Otherwise, E is determined by Dimension B (as Clause D.2).

Figure D/1b : Type 2 Corner Detail



The figure shown is a Type 2 corner detail but the design procedure is the same for a Type 1 detail. The compression face of the effective corner is assumed to lie along BB.

d_f is the distance from the fillet bar to BB

L_{af} is the lever arm to the fillet reinforcement

L_a is the lever arm to the vertical and horizontal reinforcement at C

F_{sh} is the tensile force in the horizontal bars at point C determined in accordance with Clause D.4

F_{sv} is the tensile force in the vertical bars at point C determined in accordance with Clause D.4

F_{sf} is the design strength of the fillet bars.

Moment of resistance of the corner is given by

$$Mr = F_{sf}.L_{af} + (F_{sh} + F_{sv}).L_a/\sqrt{2}$$

The moment of resistance of the members should also be checked at the ends of the fillet

Figure D/2 : Design Method for Opening Corners

APPENDIX E - TECHNICAL REQUIREMENTS AND DESIGN CHECKLIST

E.1 Cast in-situ buried concrete box type structures are non-proprietary and the Overseeing Organisation does not have any special requirements for the procedure to be followed, including technical approval, when they are specified. The complete design is to be carried out by the Design Organisation.

E.2 Precast segmental buried concrete box type structures are normally proprietary products and the Overseeing Organisation's procedures to be followed, when specifying a proprietary manufactured structure are given in Standard SD4 (MCHW 0.2.4).

E.3 The Design Organisation, prior to the invitation of tenders, normally submits an Outline Approval in Principle (O/AIP), as part of the Technical Approval/ Technical Appraisal procedure, described in BD 2 (DMRB 1.1), for each alternative type of proprietary structure which satisfies the requirements of the scheme, (including corrugated steel buried structures where appropriate) and contains a Schedule of Employer's Requirements. (Note: Currently BD 2 is not applicable in Northern Ireland, however, the requirements as outlined in this Appendix may be followed as appropriate. An update to BD 2 is planned for issue during 2002 and is intended to be applicable in Northern Ireland.) For precast buried concrete box type structures the Schedule of Employer's Requirements should include the following information:

i. Location plan and name of structure.

ii. Environmental considerations (see clause 1.4).

iii. Long Section along centre-line of structure.

iv. Finished levels of carriageways and side slopes within designated outline.

v. Skew of structure.

vi. Minimum internal span of structure.

vii. Minimum headroom of structure.

viii. Hydraulic requirements or clearance envelope, if any and requirements for invert protection.

ix. Gradient of invert.

x. End detail requirements, including requirements for headwalls.

xi. Highway loading requirements.

xii. Details of existing soils and other materials within the designated outline.

xiii. Requirements to satisfy any anticipated differential settlement.

xiv. Details of ground water, anticipated hydrostatic pressure and buoyancy effects (if applicable).

xv. Aggressivity of existing soil, ground water and contained water/effluent and of any requirement for sulphate-resisting or other type of cement.

xvi. Drainage requirements including provision for pumping.

xvii. Public safety requirements including lighting and protection for pedestrians around headwalls.

xviii. Aesthetic requirements.

xix. Protection of structures against vehicle impact.

xx. Identification of any design elements outside the scope of the Proprietary Manufacturer's Responsibilities, and/or identification of any elements in addition to the proprietary box structure requiring design.

xxi. Any other essential requirements.

Special requirements should be avoided. However where the circumstances are such that they are justified then care should be taken to avoid requirements implicitly favouring the system of a particular manufacturer.

E.4 Subsequent to the award of contract and prior to the commencement of construction, the Contractor normally completes the AIP form (containing the Schedule of Employer's Requirements), the design and the design certificate (including check certificate where appropriate) and submits these for approval/appraisal. The full design normally contains the following additional information relating to the particular proprietary product which forms the basis of the design.

a. Structure Geometry
- Structure type
- Internal span
- Internal height
- Length between joints

b. Materials
- Whether reinforced or prestressed concrete
- Concrete mix and grade
- Concrete sections adopted
- Details of reinforcement/ prestressing

c. Confirmation of foundation depth and material

d. Bedding and/or blinding details

e. Joints and joint treatment including sealing for water-tightness (if required)

f. Details of waterproofing system for roof and walls and of other waterproofing materials to be used

g. End treatment
- Geometry
- Concrete type and reinforcement

h. Excavation and construction sequence with drawings

E.5 The further stages in the post award contract procedures are given in Chapter 4 of Standard SD4 (MCHW 0.2.4).

E.6 Construction copies of the following will be required for inclusion in the maintenance manual and health and safety file:

The completed AIP used for design.
A set of as constructed drawings.
A design summary for information listed in Clause E.4.
Design and Check certificates.
Other relevant information.