DESIGN MANUAL FOR ROADS AND BRIDGES

SUMMARY OF CORRECTION - BD 37/01 Volume 1, Section 3, Part 14
LOADS FOR HIGHWAYS BRIDGES

In August 2001, page A/46, were issued incorrectly. (Clause 5.4.2 paragraph 2, had a typing error in the sentence) Please remove this page and insert new one attached, dated May 2002.

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VOLUME 1 HIGHWAY STRUCTURES: APPROVAL PROCEDURES AND GENERAL DESIGN

SECTION 3 GENERAL DESIGN

PART 14

BD 37/01

LOADS FOR HIGHWAY BRIDGES

SUMMARY

This Standard specifies the loading to be used for the design of highway bridges and associated structures through the attached revision of Composite Version of BS 5400: Part 2. This revision to BS 5400: Part 2 also includes the clauses that relate to railway bridge live load.

INSTRUCTIONS FOR USE

This is a revised document to be incorporated into the Manual.

1. Remove BD 37/88, which is superseded by BD 37/01 and archive as appropriate.

2. Insert BD 37/01 into Volume 1, Section 3.

3. 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.

August 2001

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THE HIGHWAYS AGENCY

SCOTTISH EXECUTIVE DEVELOPMENT DEPARTMENT

THE NATIONAL ASSEMBLY FOR WALES
CYNULLIAD CENEDLAETHOL CYMRU

THE DEPARTMENT FOR REGIONAL DEVELOPMENT NORTHERN IRELAND

Loads for Highway Bridges

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REGISTRATION OF AMENDMENTS

Amend No	Page No	Signature & Date of incorporation of amendments	Amend No	Page No	Signature & Date of incorporation of amendments

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PART 14

BD 37/01

LOADS FOR HIGHWAY BRIDGES

Contents

1. INTRODUCTION

1.1 BSI committee CSB 59/1 reviewed BS 5400: Part 2: 1978 (including BSI Amendment No 1 (AMD 4209) dated 31 March 1983) and agreed a series of major amendments including the revision of the HA loading curve. It was agreed that as an interim measure, pending a long term review of BS 5400 as a whole and bearing in mind the work on Eurocodes, the series of amendments to Part 2 would be issued by the Department of Transport rather than by BSI. Because of the large volume of technical and editorial amendments involved it has also been decided that a full composite version of BS 5400: Part 2 including all the agreed revision would be produced, forming an Appendix to the 1988 version of this Standard.

1.2 Since the incorporation of the above amendments, the new load code for wind (BS 6399: Part 2) has been published in the United Kingdom and further advances have been made in wind engineering. This has led to the need to amend the Appendix to this Standard in respect of:

- the United Kingdom wind map;
- the effect of terrain roughness on the properties of the wind;
- the effect of fetch of particular terrains on the properties of the wind;
- the effects of topography on the properties of the wind;
- the treatment of pressure coefficients (drag and lift); and
- the treatment of relieving areas.

The following amendments have been made to the clauses on thermal actions:

- additional profiles for steel plate girders and trusses;
- clarification of return periods for differential temperatures; and
- minor changes to effective bridge temperatures for box girders.

In addition, the following amendments have been made:

- updating certain aspects of highway bridges:
  - horizontal dynamic loading due to crowds on foot/cycle track bidges;
  - vehicle collision loads on support and superstructures; and
- updating of certain aspects of railway bridge loading related to:
  - deflection limits;
  - use of SW/0 loading;
  - live load distribution by sleepers;
  - effects of bridges with 3 or 4 tracks;
  - reference to UIC documents;
  - limitations on applicability of dynamic factors for RU loading;
  - reference to aerodynamic effects from passing trains; and
  - reference to combined response of track and structure to longitudinal loads.

1.3 BSI committee B525/10 has reviewed and agreed to the amendments described in 1.2.

2. SCOPE

2.1 This Standard supersedes the previous version of this standard BD 37/88.

2.2 This Standard does not cover all the loading requirements for the assessment of existing highway bridges and structures; additional requirements are given in BD 21 (DMRB 3.4).

3. USE OF THE COMPOSITE VERSION OF BS 5400: PART 2

3.1 Loads for the design of all highway bridges belonging to the Overseeing Organisation shall be as specified in the full composite version of BS 5400: Part 2 in Appendix A to this Standard.

3.2 Design loading requirements for rigid buried concrete box-type structures and for corrugated steel buried structures are given in BD 31 (DMRB 2.2) and BD 12 (DMRB 2.2), respectively.

4. ADDITIONAL REQUIREMENTS

4.1 All road bridges shall be designed to carry HA loading. In addition, a minimum of 30 units of type HB loading shall be taken for all road bridges except for accommodation bridges which shall be designed to HA loading only. The actual number of units shall be related to the class of road as specified below:

Class of road carried by structure	**Number of units of type HB loading**
Motorways and Trunk Roads (or principal road extensions of trunk routes)	45
Principal roads	37.5
Other public roads	30

4.2 For highway bridges where the superstructure carries more than seven traffic lanes (ie lanes marked on the running surface and normally used by traffic), application of type HA and type HB loading shall be agreed with the Overseeing Organisation.

4.3 Where reference is made in the composite version of BS 5400: Part 2 to the 'appropriate authority', this shall be taken to be the Overseeing Organisation, except where a reduced load factor is used for superimposed dead load in accordance with 5.2.2.1 of the document, it shall be ensured that the nominal superimposed dead load is not exceeded during the life of the bridge and that a note to this effect is given in the maintenance record for the structure.

4.4 Where a structure is designed for a purpose which is not specifically described in the composite version of BS 5400: Part 2 or in this Standard, the loading requirements must be agreed with the Overseeing Organisation and treated as an aspect not covered by current standards. This will include structures such as those carrying grass roads, access ways etc which may have to carry specific loading such as that due to emergency or maintenance vehicles. Bridleways shall normally be designed to the loading specified for foot/cycle track bridges unless they have to carry maintenance vehicles which impose a greater loading, in which case the loading requirements must be agreed with the Overseeing Organisation.

4.5 In determining the wind load (see 5.3 of the composite version of BS 5400: Part 2) and temperature effects (see 5.4 of the composite version of BS 5400: Part 2) for foot/cycle track bridges, the return period may be reduced from 120 years to 50 years subject to the agreement of the Overseeing Organisation.

4.6 The collision loads to be adopted and the safety fence provisions at bridge supports shall be agreed with the Overseeing Organisation. Generally the headroom clearance and collision loads shall be in accordance with TD 27 (DMRB 6.1) and BD 60 (DMRB 1.3) respectively.

4.7 In addition to 5.7 of the composite version of BS 5400: Part 2, the following conditions shall apply when assessing structures for the effects of loading caused by abnormal indivisible loads (AIL);

1. Wheel and axle loads shall be taken as nominal loads;

2. The longitudinal load caused by braking or traction shall be taken as whichever of the following produces the most severe effect;

(a) the HB traction/braking force applied in accordance with the composite version;

(b) a braking force of 15% of the gross weight of the AIL vehicle train distributed proportionally to the load carried by the individual braking axles;

(c) a traction force of 10% of the gross weight of the AIL vehicle train distributed proportionally to the load carried by the individual driving axles.

4.8 Departure from any of the requirements given in this Standard (including the composite version of BS 5400: Part 2) shall be agreed with the Overseeing Organisation.

5. REFERENCES

5.1 The following documents are referred to in the preceding sections of this Standard.

(1) BS 5400: Steel, concrete and composite bridges: Part 2: 1978: Specification for loads. Amendment No 1, 31 March 1983.

(2) BS 6399: Part 2: 1997: Code of practice for wind loads.

(4) BD 12 (DMRB 2.2): Design of Corrugated Steel buried structures with spans not exceeding 8m including circular arches.

(5) BD 21 (DMRB 3.4): The assessment of highway bridges and structures.

(6) BD 31 (DMRB 2.2): Buried concrete box type structures.

(7) BD 60 (DMRB 1.3): Design of highway bridges for vehicle collision loads.

(8) TD 27 (DMRB 6.1) Cross-Sections and Headrooms.

6. ENQUIRIES

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

Chief Highway Engineer The Highways Agency St Christopher House Southwark Street London SE1 0TE	G CLARKE Chief Highway Engineer

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: COMPOSITE VERSION OF BS 5400: PART 2

FOR THE SPECIFICATION OF LOADS USED FOR THE DESIGN OF HIGHWAY BRIDGES AND ASSOCIATED STRUCTURES.

BS 5400: Part 2: 1978

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FIGURES

FOREWORD

BS 5400 is a document combining codes of practice to cover the design and construction of steel, concrete and composite bridges and specifications for loads, materials and workmanship. It comprises the following Parts:

Part 1	General statement
Part 2	Specification of loads
Part 3	Code of practice for design of steel bridges
Part 4	Code of practice for design of concrete bridges
Part 5	Code of practice for design of composite bridges
Part 6	Specification for materials and workmanship, steel
Part 7	Specification for materials and workmanship, concrete, reinforcement and prestressing tendons
Part 8	Recommendations for materials and workmanship, concrete, reinforcement and prestressing tendons
Part 9	Bridge bearings Section 9.1 Code of practice for design of bridge bearings Section 9.2 Specification for materials, manufacture and installation of bridge bearings
Part 10	Code of practice for fatigue

British Standard

STEEL, CONCRETE AND COMPOSITE BRIDGES

Part 2. Specification for loads

1. SCOPE

1.1 **Documents comprising this British Standard.** This specification for loads should be read in conjunction with the other Parts of BS 5400 which deal with the design, materials and workmanship of steel, concrete and composite bridges.

1.2 **Loads and factors specified in this Part of BS 5400**. This Part of BS 5400 specifies nominal loads and their application, together with the partial factors, γ_{fL}, to be used in deriving design loads. The loads and load combinations specified are highway, railway and foot/cycle track bridges in the United Kingdom. Where different loading regulations apply, modifications may be necessary.

1.3 **Wind and temperature.** Wind and temperature effects relate to conditions prevailing in the United Kingdom and Eire. If the requirements of this Part of BS 5400 are applied outside this area, relevant local data should be adopted.

2. REFERENCES

The titles of the standards publications referred to in this Part of BS 5400 are listed at the end of this document (see page 97).

3. PRINCIPLES, DEFINITIONS AND SYMBOLS

3.1 **Principles.** *Part 1 of this standard sets out the principles relating to loads, limit states, load factors, etc.

3.2 **Definitions.** For the purposes of this Part of BS 5400 the following definitions apply.

3.2.1 **Loads.** External forces applied to the structure and imposed deformations such as those caused by restraint of movement due to changes in temperature.

3.2.1.1 **Load effects.** The stress resultants in the structure arising from its response to loads (as defined in 3.2.1).

3.2.2 **Dead load.** The weight of the materials and parts of the structure that are structural elements, but excluding superimposed materials such as road surfacing, rail track ballast, parapets, main, ducts, miscellaneous furniture, etc.

3.2.3 **Superimposed dead load.** The weight of all materials forming loads on the structure that are not strutural elements.

3.2.4 **Live loads.** Loads due to vehicle or pedestrian traffic.

3.2.4.1 **Primary live loads.** Vertical live loads, considered as static loads, due directly to the mass of traffic.

3.2.4.2 **Secondary live loads**. Live loads due to changes in speed or direction of the vehicle traffic eg lurching, nosing, centrifugal, longitudinal, skidding and collision loads.

*Attention is drawn to the difference in principle of this British Standard from its predecessor, BS 153.

3.2.5 **Adverse and relieving areas and effects.** Where an element or structure has an influence line consisting of both positive and negative parts, in the consideration of loading effects which are positive, the positive areas of the influence line are referred to as adverse areas and their effects as adverse effects and the negative areas of the influence line are referred to as relieving areas and their effects as relieving effects. Conversely, in the consideration of loading effects which are negative, the negative areas of the influence line are referred to as adverse areas and their effects as adverse effects and the positive areas of the influences line are referred to as relieving areas and their effects as relieving effects.

3.2.6 **Total effects.** The algebraic sum of the adverse and relieving effects.

3.2.7 **Dispersal.** The spread of load through surfacing, fill, etc.

3.2.8 **Distribution.** The sharing of load between directly loaded members and other members not directly loaded as a consequence of the stiffness of intervening connecting members, as eg diaphragms between beams, or the effects of distribution of a wheel load across the width of a plate or slab.

3.2.9 **Highway carriageway and lanes** (figure 1 gives a diagrammatic description of the carriageway and traffic lanes).

3.2.9.1 **Carriageway.** For the purposes of this Standard, that part of the running surface which includes all traffic lanes, hard shoulders, hard strips and marker strips. The carriageway width is the width between raised kerbs. In the absence of raised kerbs it is the width between safety fences, less the amount of set-back required for these fences, being not less than 0.6m or more than 1.0m from the traffic face of each fence. The carriageway width shall be measured in a direction at right angles to the line of the raised kerbs, lane marks or edge marking.

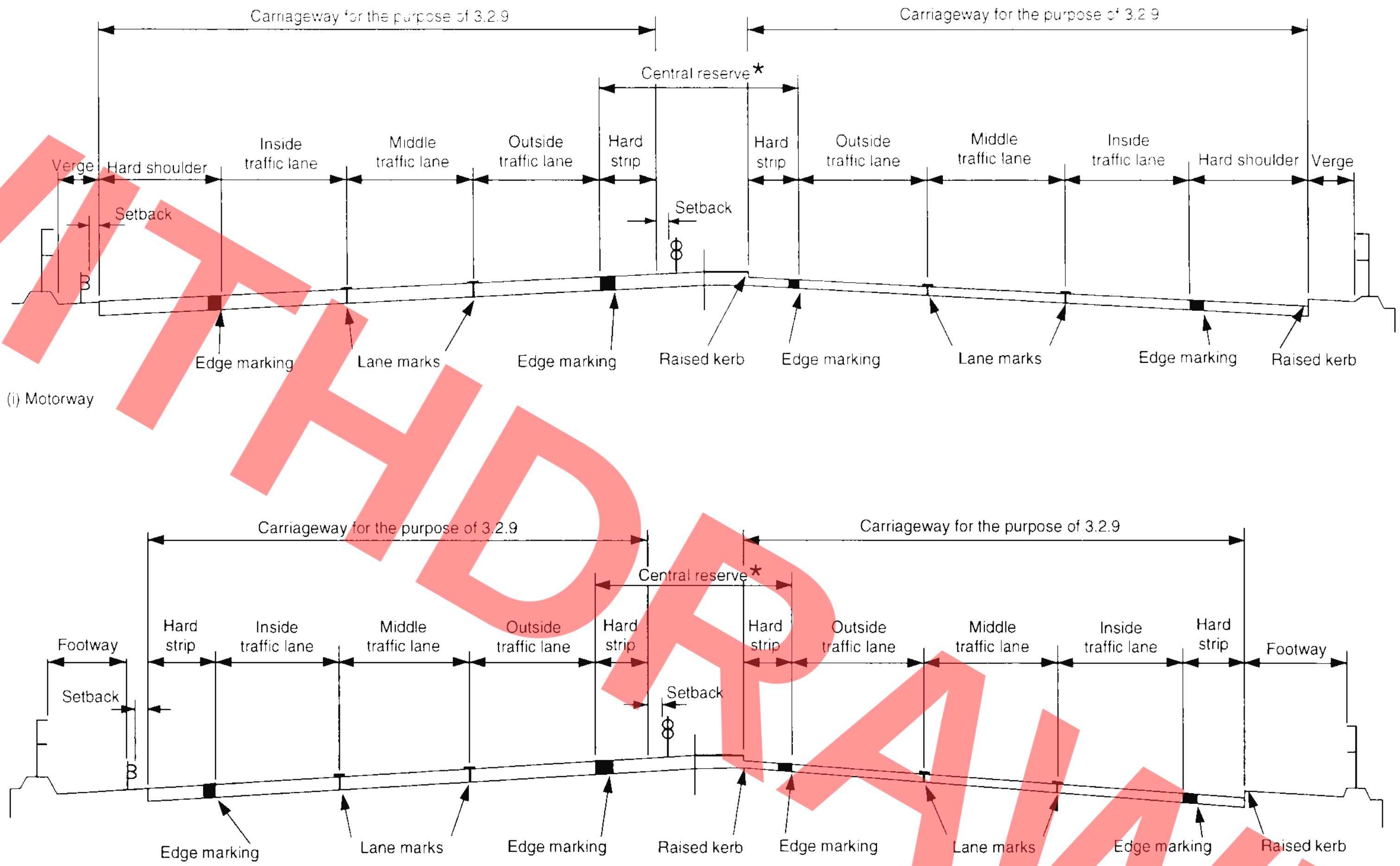
NOTE: For ease of use, the definition of "carriageway" given in this Standard differs from that given in BS 6100: Part 2.

3.2.9.2 **Traffic lanes.** The lanes that are marked on the running surface of the bridge and are normally used by traffic.

3.2.9.3 **Notional lanes.** The notional parts of the carriageway used solely for the purposes of applying the specified live loads. The notional lane width shall be measured in a direction at right angles to the line of the raised kerbs, lane markers or edge marking.

3.2.9.3.1 **Carriageway widths of 5.00m or more.** Notional lanes shall be taken to be not less than 2.50m wide. Where the number of notional lanes exceeds two, their individual widths should be not more than 3.65m. The carriageway shall be divided into an integral number of notional lanes have equal widths as follows:

Carriageway width m	Number of notional lanes
5.00 up to and including 7.50	2
above 7.50 up to and including 10.95	3
above 10.95 up to and including 14.60	4
above 14.60 up to and including 18.25	5
above 18.25 up to and including 21.90	6



(ii) All-purpose road

(a) Superstructures : dual carriageway

*Central reserve will be split on separate superstructures

Figure 1. Highway carriageway and traffic lanes

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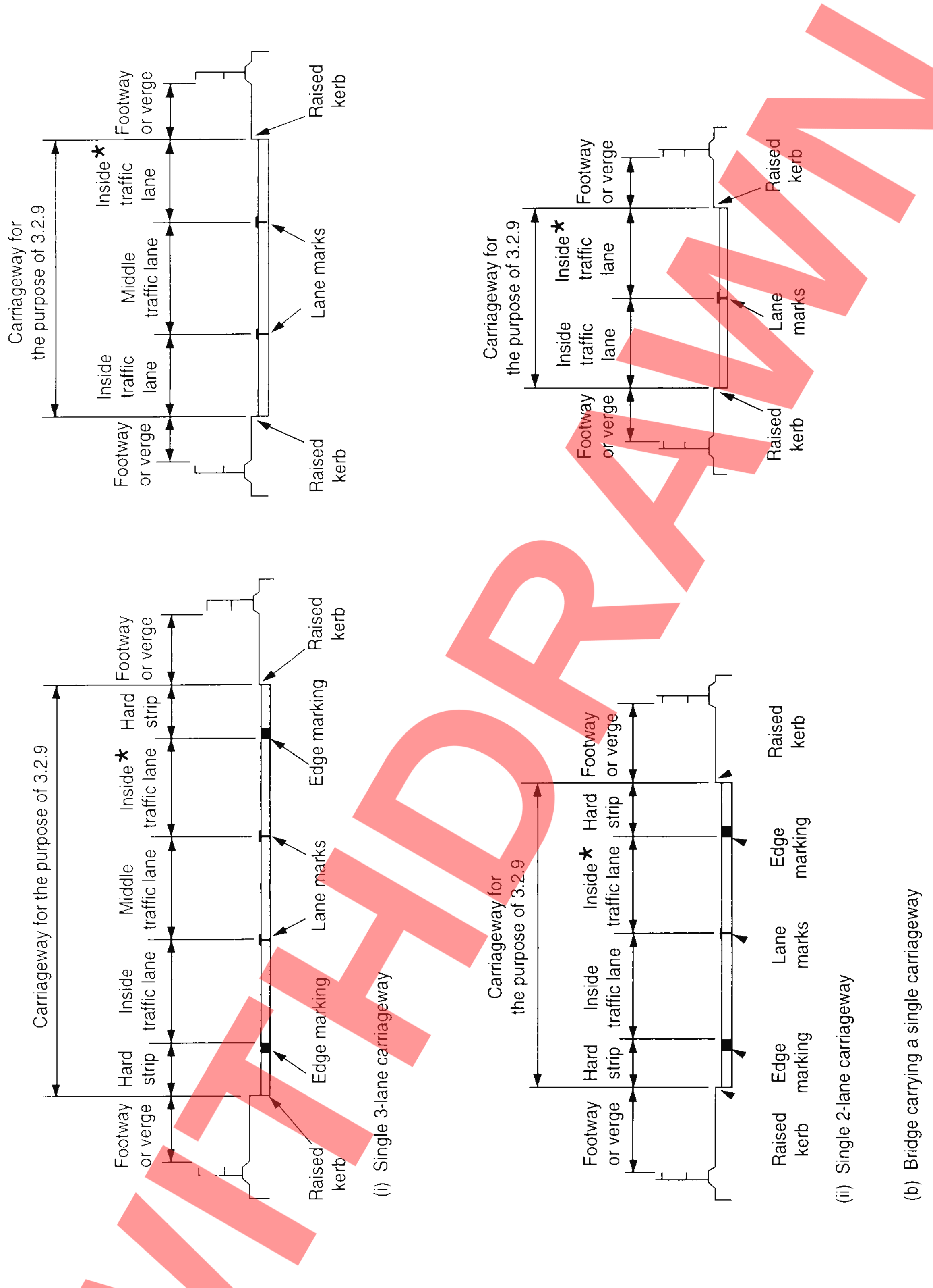

*Where the carriageway carries unidirectional traffic only, this lane becomes the outside traffic lane
NOTE 1. The same definitions of inside, middle and outside have been used for notional lanes.
NOTE 2. Where a safety fence replaces a raised kerb the limits of the footway or verge and the hard strip shall be as shown in figure 1(a).

Figure 1. (continued)

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3.2.9.3.2 **Carriageway widths of less than 5.00m.** The carriageway shall be taken to have one notional lane with a width of 2.50m. The loading on the remainder of the carriageway width should be as specified in 6.4.1.1.

3.2.9.3.3 **Dual carriageway structures.** Where dual carriageways are carried on one superstructure, the number of notional lanes on the bridge shall be taken as the sum of the number of notional lanes in each of the single carriageways as specified in 3.2.9.3.1.

3.2.10 **Bridge components**

3.2.10.1 **Superstructure.** In a bridge, that part of the structure which is supported by the piers and abutments.

3.2.10.2 **Substructure.** In a bridge, the wing walls and the piers, towers and abutments that support the superstructure.

3.2.10.3 **Foundation.** That part of the substructure in direct contact with, and transmitting load to, the ground.

3.3 **Symbols.** The following symbols are used in this Part of BS 5400.

a	maximum vertical acceleration
A_1	solid area in normal projected elevation
A_2	see 5.3.4.6
A_3	area in plan used to derive vertical wind load
b	width used in deriving wind load
b_L	notional lane width
c	spacing of plate girders used in deriving drag factor
C	configuration factor
C_D	drag coefficient
C_L	lift coefficient
d	depth used in deriving wind load
d_1	depth of deck
d_2	depth of deck plus solid parapet
d_3	depth of deck plus live load
d_L	depth of live load
E	modulus of elasticity
f	a factor used in deriving centrifugal load on railway tracks
f_o	fundamental natural frequency of vibration
F	pulsating point load
F_C	centrifugal load
h, h_1, h_2, h_3, h_4	depth (see figure 9)
H	height of bridge above local ground level
H_o	roof top level above ground level
I	second moment of area
j	maximum value of ordinate of influence line
k	a constant used to dervie primary live load on foot/cycle track bridges
K	configuration factor
K_F	fetch correction factor
l	main span
l_1	length of the outer span of a three-span superstructure
L	loaded length
L_D	actual length of downwind slope
L_U	actual length of upwind slope
L_b	effective base length of influence line (see figure 11)
L_e	effective length of upwind slope

M	weight per unit length (see B.2.3)
N	number of lanes
N	number of axles (see Appendix D)
n	number of beams or box girders
P	equivalent uniformly distributed load
P_L	nominal longitudinal wind load
P_t	nominal transverse wind load
P_v	nominal vertical wind load
q	dynamic pressure head
r	radius of curvature
s	topographic location factor
S_1	funnelling factor
S_a	altitude factor
S_b, S_b'	bridge and terrain factor
S_c, S_c'	hourly speed factor
S_d	direction factor
S_g	gust factor
S_h, S_h'	topography factor
S_p	probability factor
S_s	seasonal factor
t	thickness of pier
T	time in seconds (See B.3)
T, T_1, T_2, T_3, T_4	temperature differential (see figure 9 and appendix C)
T_c	hourly mean reduction factor for towns
T_g	gust reduction factor for towns
U	area under influence line
v_t	speed of highway or rail traffic
V_b	basic hourly mean wind speed
V_d	maximum wind gust speed
V_r	hourly mean wind speed for relieving areas
V_s	site hourly mean wind speed
W	load per metre of lane
X	horizontal distance of site from the crest
y_s	static deflection
Z	effective height of topographic feature
α_1, α_2	lane factors (see 6.4.1.1)
β_1	first lane factor
β_2	second lane factor
β_3	third lane factor
β_n	fourth and subsequent lane factor
γ_{fl}, γ_{f2}	see Part 1 of this standard
γ_{f3}	see 4.1.3 and Part 1 of this standard
γ_{fL}	partial load factor (γ_{fl} x γ_{f2})
δ	logarithmic decrement of decay of vibration
Δ, Δ_s	altitude above mean sea level
Δ_T	base of topography
η	shielding factor
θ	angle of wind (see 5.3.5)
ψ	dynamic response factor (see B.2.6)
ψ	average slope of ground (see Fig 3)
ψ_D	downwind slope
ψ_u	upwind slope
ϕ	wind direction (see 5.3.2.2.4)

4. LOADS: GENERAL

4.1 Loads and factors specified

4.1.1 **Nominal loads.** Where adequate statistical distribution are available, nominal loads are those appropriate to a return period of 120 years. In the absence of such statistical data, nominal load values that are considered to approximate to a 120-year return period are given.

4.1.2 **Design loads**. Nominal loads should be multiplied by the approporiate value of γ_{fL} to derive the design load to be used in the calculation of moments, shears, total loads and other effects for each of the limit states under consideration. Values of γ_{fL} are given in each relevant clause and also in table 1.

4.1.3 **Additional factor γ_{f3}.** Moments, shears, total loads and other effects of the design loads are also to be multiplied by γ_{f3} to obtain the design load effects. Values of γ_{f3} are given in Parts 3, 4 and 5 of this standard.

4.1.4 **Fatigue loads.** Fatigue loads to be considered for highway and railway bridges, together with the appropriate value of γ_{fL}, are given in Part 10 of this standard.

4.1.5 **Deflection, drainage and camber.** The requirements for calculating the deflection, camber and drainage characteristics of the structure are given in Parts 3, 4 and 5 of this standard.

4.2 **Loads to be considered.** The loads to be considered in different load combinations, together with the specified values γ_{fL}, are set out in the appropriate clauses and summarised in table 1.

4.3 **Classification of loads.** The loads applied to a structure are regarded as either permanent or transient.

4.3.1 **Permanent loads.** For the purposes of this standard, dead loads, superimposed dead loads and loads due to filling material shall be regarded as permanent loads.

4.3.1.1 **Loading effects not due to external action.** Loads deriving from the nature of the structural material, its manufacture or the circumstances of its fabrication are dealt with in the appropriate Parts of this standard. Where they occur they shall be regarded as permanent loads.

4.3.1.2 **Settlement.** The effect differential settlement of supports shall be regarded as a permanent load where there is reason to believe that this will take place, and no special provision has been made to remedy the effect.

4.3.2 **Transient loads.** For the purposes of this standard all loads other than permanent ones shall be considered transient.

The maximum effects of certain transient loads do not coexist with the maximum effects of certain others. The reduced effects that can coexist are specified in the relevant clauses.

4.4 **Combinations of loads.** Three principal and two secondary combinations of loads are specified; values of γ_{fL} for each load for each combination in which it is considered are given in the relevant clauses and also summarised in table 1.

4.4.1 **Combination 1.** For highway and foot/cycle track bridges, the loads to be considered are the permanent loads, together with the appropriate primary live loads, and, for railway bridges, the permanent loads, together with the appropriate primary and secondary live loads.

4.4.2 **Combination 2.** For all bridges, the loads to be considered are the loads in combination 1, together with those due to wind and, where erection is being considered, temporary erection loads.

4.4.3 **Combination 3.** For all bridges, the loads to be considered are the loads in combination 1, together with those arising from restraint due to the effects of temperature range and difference, and, where erection is being considered, temporary erection loads.

4.4.4 **Combination 4.** Combination 4 does not apply to railway bridges except for vehicle collision loading on bridge supports. For highway bridges, the loads to be considered are the permanent loads and the secondary live loads, together with the appropriate primary live loads associated with them. Secondary live loads shall be considered separately and are not required to be combined. Each shall be taken with its appropriate associated primary live load.

For foot/cycle track bridges, the only secondary live loads to be considered are the vehicle collision loads on bridge supports and superstructures (see 7.2).

Table 1. Loads to be taken in each combination with appropriate γ_{fL}

ULS: ultimate limit state SLS: serviceability limit state

Clause number	Load	Limit state	γ_{fL} to be considered in combination				
			1	2	3	4	5
5.1	Dead: steel	ULS* SLS	1.05 1.00	1.05 1.00	1.05 1.00	1.05 1.00	1.05 1.00
	concrete	ULS* SLS	1.15 1.00	1.15 1.00	1.15 1.00	1.15 1.00	1.15 1.00
5.2	Superimposed dead: deck surfacing	ULS+ SLS+	1.75 1.20	1.75 1.20	1.75 1.20	1.75 1.20	1.75 1.20
	other loads	ULS SLS	1.20 1.00	1.20 1.00	1.20 1.00	1.20 1.00	1.20 1.00
5.1.2.2 & 5.2.2.2	Reduced load factor for dead and superimposed dead load where this has a more severe total effect	ULS	1.00	1.00	1.00	1.00	1.00
5.3	Wind: during erection	ULS SLS		1.10 1.00			
	with dead plus superimposed dead load only, and for members primarily resisting wind loads	ULS SLS		1.40 1.00			
	with dead plus superimposed dead plus other appropriate combination 2 loads	ULS SLS		1.10 1.00			
	relieving effect of wind	ULS SLS		1.00 1.00			
5.4	Temperature: restraint to movement, except frictional	ULS SLS			1.30 1.00		
	frictional bearing restraint	ULS SLS					1.30 1.00
	effect of temperature difference	ULS SLS			1.00 0.80		
5.6	Differential settlement	ULS SLS	1.20 1.00	1.20 1.00	1.20 1.00	1.20 1.00	1.20 1.00
5.7	Exceptional loads		to be assessed and agreed between the engineer & the appropriate authority				
5.8	Earth pressure: retained fill and/ or live load — vertical loads	ULS SLS	1.20 1.00	1.20 1.00	1.20 1.00	1.20 1.00	1.20 1.00
	non-vertical loads	ULS SLS	1.50 1.00	1.50 1.00	1.50 1.00	1.50 1.00	1.50 1.00
	relieving effect	SLS	1.00	1.00	1.00	1.00	1.00
5.9	Erection: temporary loads	ULS SLS		1.15 1.00	1.15 1.00		
6.2	Highway bridges live loading: HA alone	ULS SLS	1.50 1.20	1.25 1.00	1.25 1.00		
6.3	HA with HB or HB alone	ULS SLS	1.30 1.10	1.10 1.00	1.10 1.00		
6.5	footway and cycle track loading	ULS SLS	1.50 1.00	1.25 1.00	1.25 1.00		
6.6	accidental wheel loading**	ULS SLS	1.50 1.20				

*γ_{fL} shall be increased to at least 1.10 and 1.20 for steel and concrete respectively to compensate for inaccuracies when dead loads are not accurately assessed.
+γ_{fL} may be reduced to 1.2 and 1.0 for the ULS and SLS respectively subject to approval of the appropriate authority (see 5.2.2.1).
**Accidental wheel loading shall not be considered as acting with any other primary live loads.

Table 1 (continued)

Clause number	Load		Limit state	γ_{fL} to be considered in combination 1	2	3	4	5
6.7.1	Loads due to vehicle collision with parapets & associated pimary live load:	Local effects: parapet load low & normal containment high containment	 ULS SLS ULS SLS	each secondary live load shall be considered separately together with the other combination 4 loads as appropriate			 1.50 1.20 1.40 1.15	
		associated primary live load: low, normal & high containment	ULS SLS				1.30 1.10	
6.7.2		Global effects: parapet load Massive structures: bridge superstructures and non-elastomeric bearings bridge substructures and wing & retaining walls elastomeric bearings Light structures: bridge superstructures & non-elastomeric bearings bridge substructures and wing & retaining walls elastomeric bearings associated primary live load: Massive & light structures: bridge superstructures, non-elastomeric bearings, bridge substructures & wing & retaining walls elastomeric bearings	 ULS ULS SLS ULS ULS SLS ULS SLS				 1.25 1.00 1.00 1.40 1.40 1.00 1.25 1.00	
6.8	Vehicle collision loads on bridge supports and superstructures:	Effects on all elements excepting elastomeric bearings Effects on elastomeric bearings	ULS SLS				1.50 1.00	
6.9	Centrifugal load & associated primary live load		ULS SLS				1.50 1.00	
6.10	Longitudinal load:	HA & associated primary live load	ULS SLS				1.25 1.00	
		HB associated primary live load	ULS SLS				1.10 1.00	
6.11	Accidental skidding load and associated primary live load		ULS SLS				1.25 1.00	
7	Foot/cycle track bridges:	live load & effects due to parapet load	ULS SLS	1.50 1.00	1.25 1.00	1.25 1.00		
		vehicle collision loads on supports and superstructures***	ULS				1.50	
8	Railway bridges:	type RU and RL, and SW/0 primary and secondary live loading	ULS SLS	1.40 1.10	1.20 1.00	1.20 1.00		

***This is the only secondary live load to be considered for foot/cycle track bridges.

NOTE. For loads arising from creep and shrinkage, or from welding and lack of fit, see Parts 3, 4 and 5 of this standard, as appropriate.

4.4.5 **Combination 5.** For all bridges, the loads to be considered are the permanent loads, together with the loads due to friction at bearings*

4.5 **Application of loads.** Each element and structure shall be examined under the effects of loads that can coexist in each combination.

4.5.1 **Selection to cause most adverse effect+.** Design loads shall be selected and applied in such a way that the most adverse total effect is caused in the element or structure under consideration.

4.5.2 **Removal of superimposed dead load.** Consideration shall be given to the possibility that the removal of superimposed dead load from part of the structure may diminish its relieving effect. In so doing the adverse effect of live load on the elements of the structure being examined may be modified to the extent that the removal of the superimposed dead load justifies this.

4.5.3 **Live load.** Live load shall not be considered to act on relieving areas except in the case of wind on live load when the presence of light traffic is necessary to generate the wind load (see 5.3.8).

4.5.4 **Wind on relieving areas.** Design loads due to wind on relieving areas shall be modified in accordance with 5.3.2.2 and 5.3.2.4.

4.6 **Overturning.** The stability of the superstructure and its parts against overturning shall be considered for the ultimate limit state.

4.6.1 **Restoring moment.** The least restoring moment due to the unfactored nominal loads shall be greater than the greatest overturning moment due to the design loads (ie γ_{fL} for the ultimate limit state x the effects of the nominal loads).

4.6.2 **Removal of loads.** The requirements specified in 4.5.2 relating to the possibe removal of superimposed dead load shall also be taken into account in considering overturning.

4.7 **Foundation pressures, sliding on foundations, loads on piles, etc.** In the design of foundations, the dead load (see 5.1) the superimposed dead load (see 5.2) and loads due to filling material (see 5.8.1) shall be regarded as permanent loads and all live loads, temperature effects and wind loads shall be regarded as transient loads, except in certain circumstances such as a main line railway bridge outside a busy terminal where it may be necessary to assess a proportion of live load as being permanent.

The design of foundations including consideration of overturning shall be based on the principles set out in BS 8004 using load combinations as given in this Part.

4.7.1 **Design loads to be considered with BS 8004.** BS 8004 has not been drafted on the basis of limit state design; it will therefore be appropriate to adopt the nominal loads specified in all relevant clauses of this standard as design loads (taking γ_{fL} = 1.0 and γ_{f3} = 1.0) for the purpose of foundation design in accordance with BS 8004.

*Where a member is required to resist the loads due to temperature restraint within the structure and to frictional restraint of temperature-induced movement at bearings, the sum of these effects shall be considered. An example is the abutment anchorage of a continuous structure where temperature movement is accommodated by flexure of piers in some spans and by roller bearings in others.

+It is expected that experience in the use of this standard will enable users to identify those load cases and combinations (as in the case of BS 153) which govern design provisions, and it is only those load cases and combinations which need to be established for use in practice.

5. LOADS APPLICABLE TO ALL BRIDGES

5.1 Dead load

5.1.1 **Nominal dead load.** Initial values for nominal dead load may be based on the densities of the materials given in BS 648. The nominal dead load initially assumed shall be accurately checked with the actual weights to be used in construction and, where necessary, adjustments shall be made to reconcile any discrepancies.

5.1.2 **Design load.** The factor, γ_{fL} to be applied to all parts of the dead load, irrespective of whether these parts have an adverse or relieving effect, shall be taken for all five load combinations as follows:

	For the ultimate limit state	For the serviceability limit state
Steel	1.05	1.0
Concrete	1.15	1.0

except as specified in 5.1.2.1 and 5.1.2.2.

These values for γ_{fL} assume that the nominal dead load has been accurately assessed, that the weld metal and bolts, etc, in steelwork and the reinforcement, etc, in concrete have been properly quantified and taken into acount and that the densities and materials have been confirmed.

5.1.2.1 **Approximations in assessment of load.** Any deviation from accurate assessment of nominal dead load for preliminary design or for other purposes should be accompanied by an appropriate and adequate increment in the value of γ_{fL}. Values of 1.1 for steel and 1.2 for concrete for the ultimate limit state will usually suffice to allow for the minor approximations normally made. It is not possible to specify the allowances required to be set against various assumptions and approximations, and it is the responsibility of the engineer to ensure that the absolute values specified in 5.1.2 are met in the completed structure.

5.1.2.2 **Alternative load factor.** Where the structure or element under consideration is such that the application of γ_{fL} as specfied in 5.1.2 for the ultimate limit state causes a less severe total effect (see 3.2.6) than would be the case if γ_{fL}, applied to all parts of the dead load, had been taken as 1.0, values of 1.0 shall be adopted. However, the γ_{fL} factors to be applied when considering overturning shall be in accordance with 4.6.

5.2 Superimposed dead load

5.2.1 **Nominal superimposed dead load.** Initial values for nominal superimposed dead load may be based on the densities of the materials given in BS 648. The nominal superimposed dead load initially assumed shall in all cases be accurately checked with the actual weights to be used in construction and, where necessary, adjustments shall be made to reconcile any discrepancies.

Where the superimposed dead load comprises filling, eg on spandrel filled arches, consideration shall be given to the fill becoming saturated.

5.2.2 **Design load.** The factor γ_{fL}, to be applied to all parts of the superimposed dead load, irrespective of whether these parts have an adverse or relieving effect, shall be taken for all five load combinations as follows:

	For the ultimate limit state	For the serviceability limit state
deck surfacing	1.75	1.20
other loads	1.20	1.00

exept as specified in 5.2.2.1 and 5.2.2.2 (Note also the requirements 4.5.2).

NOTE The term "other loads" here includes non-structural concrete infill, services and any surrounding fill, permanent formwork, parapets and street furniture.

5.2.2.1 **Reduction of load factor.** The value of γ_{fL} to be used in conjunction with the superimposed dead load may be reduced to an amount not less than 1.2 for the ultimate limit state and 1.0 for the serviceability limit state, subject to the approval of the appropriate authority which shall be responsible for ensuring that the nominal superimposed dead load is not exceeded during the life of the bridge.

5.2.2.2 **Alternative load factor.** Where the structure or element under consideration is such that the application of γ_{fL} as specfied in 5.2.2 for the ultimate limit state causes a less severe total effect (see 3.2.6) than would be the case if γ_{fL}, applied to all parts of the superimposed dead load, had been taken as 1.0, values of 1.0 shall be adopted. However, the γ_{fL} factors to be applied when considering overturning shall be in accordance with 4.6.

5.3 Wind loads

5.3.1 **General.** The wind pressure on a bridge depends on the geographical location, the terrain of the surrounding area, the fetch of terrains upwind of the site location, the local topography, the height of the bridge above ground, and the horizontal dimensions and cross-section of the bridge or element under consideration. The maximum pressures are due to gusts that cause local and transient fluctuations about the mean wind pressure.

The methods provided herein simulate the effects of wind actions using static analytical procedures. They shall be used for highway and railway bridges of up to 200m span and for footbridges up to 30m span. For bridges outside these limits consideration should be given to the effects of dynamic response due to turbulence taking due account of lateral, vertical and torsional effects; in such circumstances specialist advice should be sought.

Wind loading will generally not be significant in its effect on many highway bridges, such as concrete slab or slab and beam structures of about 20m or less in span, 10m or more in width and at normal heights above ground.

In general, a suitable check for such bridges in normal circumstances would be to consider a wind pressure of 6 kN/m^2 applied to the vertical projected area of the bridge or structural element under consideration, neglecting those areas where the load would be beneficial.

Design gust pressures are derived from a product of the basic hourly mean wind speed, taken from a wind map (Figure 2), and the values of several factors which are dependent upon the parameters given above.

5.3.2 **Wind Gust Speed.** Where wind on any part of the bridge or its elements increases the effect under consideration (adverse areas) the maximum design wind gust speed V_d shall be used.

5.3.2.1 **Maximum Wind Gust Speed V_d.** The maximum wind gust speed V_d on bridges without live load shall be taken as:

$$V_d = S_g V_s$$

where

V_s is the site hourly mean wind speed (see 5.3.2.2)
S_g is the gust factor (see 5.3.2.3)

For the remaining parts of the bridge or its elements which give relief to the member under consideration (relieving areas), the design hourly mean wind speed V_r shall be used as derived in 5.3.2.4.

5.3.2.2 **Site Hourly Mean Wind Speed V_s.** V_s is the site hourly mean wind speed 10m above ground at the altitude of the site for the direction of wind under consideration and for an annual probability of being exceeded appropriate to the bridge being designed, and shall be taken as:

$$V_s = V_b S_p S_a S_d$$

where

V_b is the basic hourly mean wind speed (see 5.3.2.2.1)

S_p is the probability factor (see 5.3.2.2.2)

S_a is the altitude factor (see 5.3.2.2.3)

S_d is the direction factor (see 5.3.2.2.4)

5.3.2.2.1 **Basic Hourly Mean Wind Speed V_b.** Values of V_b in m/s for the location of the bridge shall be obtained from the map of isotachs shown in Figure 2.

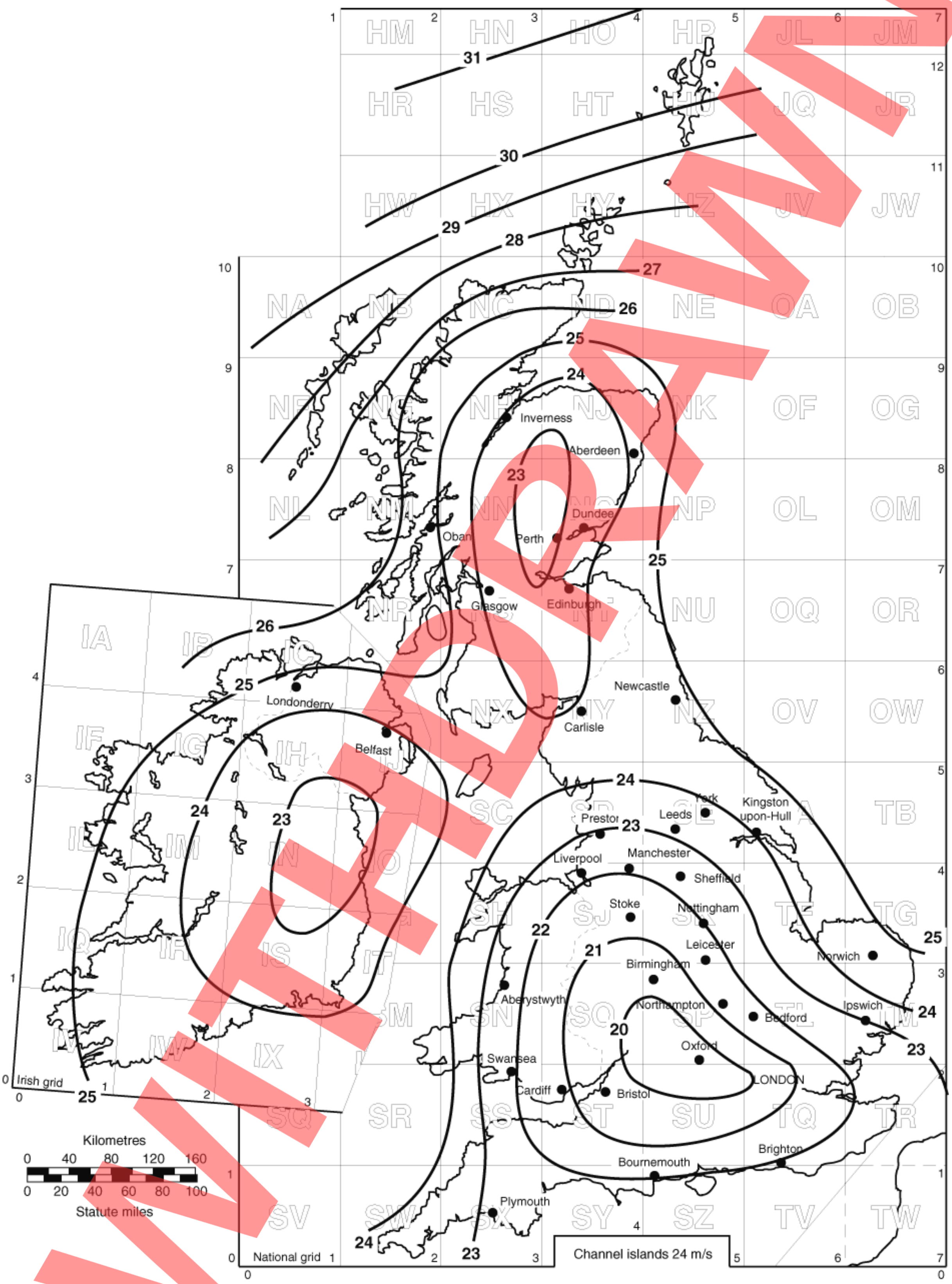
The values of V_b taken from Figure 2 are hourly mean wind speeds with an annual probability of being exceeded of 0.02 (equivalent to a return period of 50 years) in flat open country at an altitude of 10m above sea level.

5.3.2.2.2 **Probability Factor.** The probability factor, S_p, shall be taken as 1.05 for highway, railway and foot/cycle track bridge appropriate to a return period of 120 years.

For foot/cycle track bridges, subject to the agreement of the appropriate authority, a return period of 50 years may be adopted and S_p shall be taken as 1.00.

During erection, the value of S_p may be taken as 0.90 corresponding to a return period of 10 years. For other probability levels S_p may be obtained from Appendix E. Where a particular erection will be completed in a short period, S_p shall be combined with a seasonal factor also obtained from Appendix E.

Figure 2: Basic wind speed V_b in m/sec



5.3.2.2.3 **Altitude factor S_a.** The altitude factor S_a shall be used to adjust the basic wind speed V_b for the altitude of the site above sea level and shall be taken as:

$$S_a = 1 + 0.001\,\Delta$$

where

Δ is the altitude in metres above mean sea level of:

a) the ground level of the site when topography is not significant, or

b) the base of the topographic feature when topography is significant in accordance with 5.3.2.3.3 and Figure 3.

5.3.2.2.4 **Direction factor S_d.** The direction factor, S_d may be used to adjust the basic wind speed to produce wind speeds with the same risk of exceedance in any wind direction. Values are given in Table 2 for the wind direction $\phi = 0^o$ to $\phi = 330^o$ in 30^o intervals (where the wind direction is defined in the conventional manner: an east wind is a wind direction of $\phi = 90^o$ and blows from the east to the site). If the orientation of the bridge is unknown or ignored, the value of the direction factor shall be taken as $S_d = 1.00$ for all directions. When the direction factor is used with other factors that have a directional variation, values from Table 2 shall be interpolated for the specific direction being considered.

Table 2. Values of direction factor S_d

Direction	ϕ	S_d
N	0º	0.78
	30º	0.73
	60º	0.73
E	90º	0.74
	120º	0.73
	150º	0.80
S	180º	0.85
	210º	0.93
	240º	1.00
W	270º	0.99
	300º	0.91
	330º	0.82

5.3.2.3 **Gust factor S_g.** The gust factor, S_g, depends on the terrain of the site which is defined in terms of three categories:

a) Sea - the sea (and inland areas of water extending more than 1km in the wind direction providing the nearest edge of water is closer than 1km upwind of the site);

b) Country - all terrain which is not defined as sea or town;

c) Town - built up areas with a general level of roof tops at least H_o = 5m above ground level.

NOTE: permanent forest and woodland may be treated as town category.

The gust factor, S_g, shall be taken as:

$$S_g = S_b \; T_g \; S_h'$$

where

$$S_b = S_b' \; K_F$$

S_b' is the bridge and terrain factor (see 5.3.2.3.1)

K_F is the fetch correction factor (see 5.3.2.3.1)

T_g is the town reduction factor for sites in towns (see 5.3.2.3.2)

S_h' is the topography factor (see 5.3.2.3.3)

5.3.2.3.1 **The bridge and terrain factor S_b'.** Values of S_b' are given in Table 3 for the appropriate height above ground and adversely loaded length, and apply to sea shore locations. To allow for the distance of the site from the sea in the upwind direction for the load case considered, these values may be multiplied by the fetch correction factor, K_F, also given in Table 3, for the relevant height above ground.

5.3.2.3.2 **The town reduction factor T_g.** Where the site is not situated in town terrain or is within 3km of the edge of a town in the upwind direction for the load case considered, T_g shall be taken as 1.0.

For sites in town terrain, advantage may be taken of the reduction factor T_g. The values of T_g should be obtained from Table 4 for the height above ground and distance of the site from the edge of town terrain in the upwind direction for the load case considered.

5.3.2.3.3 **The topography factor S_h'.** The values of S_h' shall generally be taken as 1.0. In valleys where local funnelling of the wind occurs, or where a bridge is sited to the lee of a range of hills causing local acceleration of wind, a value not less than 1.1 shall be taken. For these cases specialist advice should be sought.

Where local topography is significant S_h' shall be calculated in accordance with Appendix F.

Topography can only be significant when the upwind slope is greater than 0.05; see Figure 3.

5.3.2.4 **Hourly mean wind speed for relieving areas V_r for bridges without live load.** V_r shall be taken as:

$$V_r = S_m V_s$$

where

V_s is the site hourly mean wind speed (see 5.3.2.2)

S_m is the hourly mean speed factor which shall be taken as:

$$S_m = S_c T_c S_h'$$

where

$S_c = S_c' K_F$

S_c' is the hourly speed factor (see 5.3.2.4.1)

K_F is the fetch correction factor (see 5.3.2.3.1)

T_c is the hourly mean town reduction factor (see 5.3.2.4.2)

S_h' is the topography factor (see 5.3.2.3.3 and Figure 3)

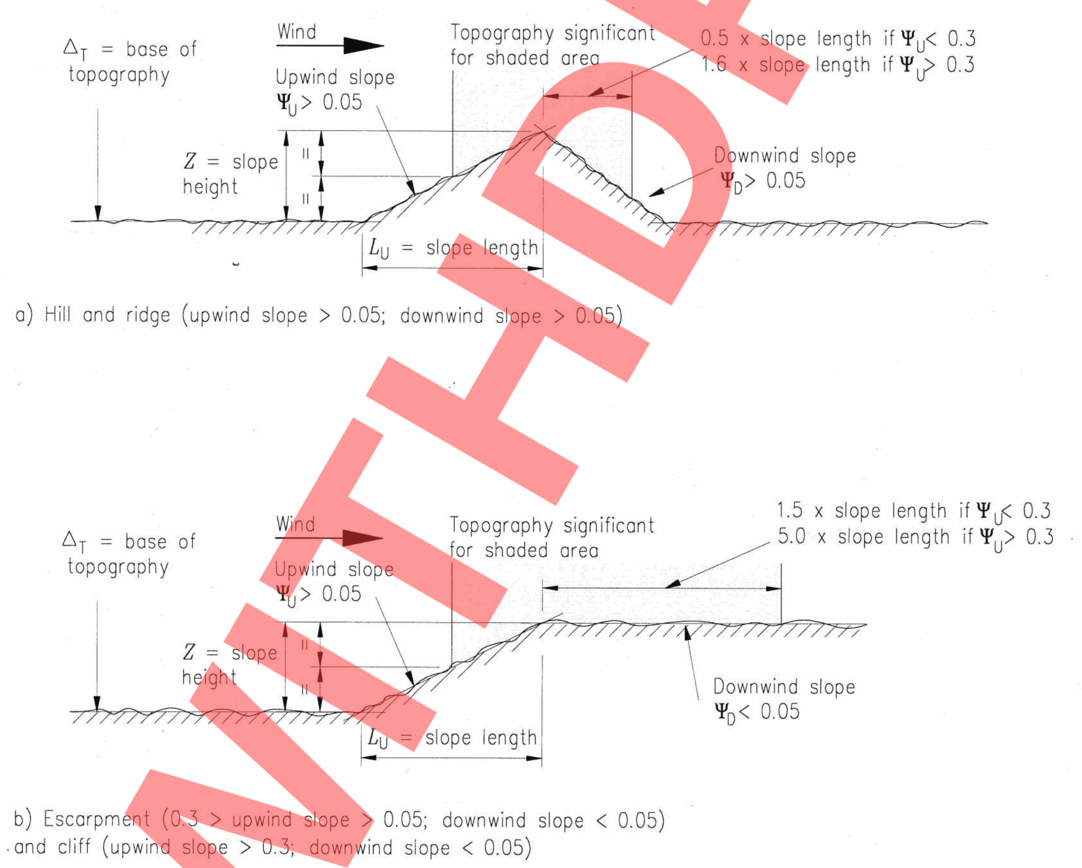


Figure 3 Definition of significant topography

Table 3. Values of terrain and bridge factor S_b', hourly speed factor S_c' and fetch correction factor, K_F

Height above ground (m)	Terrain and Bridge Factor S_b'						Hourly Speed Factor S_c'
	LOADED LENGTH (m)						
	20	40	60	100	200	400	
5	1.56	1.51	1.48	1.44	1.39	1.34	1.02
10	1.68	1.64	1.61	1.57	1.52	1.47	1.17
15	1.76	1.71	1.68	1.64	1.60	1.55	1.25
20	1.81	1.76	1.73	1.69	1.65	1.60	1.31
30	1.88	1.83	1.80	1.76	1.71	1.66	1.39
40	1.92	1.87	1.85	1.81	1.76	1.71	1.43
50	1.96	1.91	1.88	1.84	1.80	1.75	1.47
60	1.98	1.94	1.91	1.87	1.83	1.78	1.50
80	2.02	1.98	1.95	1.92	1.87	1.82	1.55
100	2.05	2.01	1.98	1.95	1.90	1.86	1.59
150	2.11	2.06	2.04	2.01	1.97	1.92	1.67
200	2.15	2.11	2.08	2.05	2.01	1.97	1.73

Height above ground (m)	Fetch Correction Factor, K_F					
	UPWIND DISTANCE OF SITE FROM SEA (km)					
	≤0.3	1	3	10	30	≥100
5	1.00	0.96	0.94	0.91	0.90	0.85
10	1.00	0.99	0.96	0.94	0.92	0.88
15	1.00	0.99	0.98	0.96	0.94	0.89
20	1.00	1.00	0.99	0.97	0.95	0.90
30	1.00	1.00	0.99	0.98	0.96	0.92
40	1.00	1.00	1.00	0.99	0.98	0.93
50	1.00	1.00	1.00	0.99	0.98	0.93
60	1.00	1.00	1.00	0.99	0.99	0.94
80	1.00	1.00	1.00	1.00	0.99	0.95
100	1.00	1.00	1.00	1.00	0.99	0.95
150	1.00	1.00	1.00	1.00	1.00	0.96
200	1.00	1.00	1.00	1.00	1.00	0.97

Table 4. Gust speed reduction factor T_g for bridges in towns

Height above ground (m)	Distance from edge of town in upwind direction (km)		
	3	10	30
5	0.84	0.81	0.79
10	0.91	0.87	0.85
15	0.94	0.90	0.88
20	0.96	0.92	0.90
30	0.98	0.95	0.92
40	0.99	0.97	0.94
50	0.99	0.98	0.95
60	0.99	0.99	0.96
80	0.99	0.99	0.98
100 150 200	1.0 throughout		

Table 5. Hourly mean reduction factor T_c for bridges in Towns

Height above ground (m)	Distance from edge of town (km)		
	3	10	30
5	0.74	0.71	0.69
10	0.81	0.78	0.76
15	0.84	0.82	0.80
20	0.87	0.84	0.82
30	0.89	0.86	0.84
40	0.91	0.88	0.86
50	0.93	0.90	0.87
60	0.94	0.91	0.88
80	0.95	0.92	0.90
100	0.96	0.93	0.91
150	0.98	0.95	0.93
200	1.00	0.96	0.94

NOTES FOR TABLES 3 to 5:

NOTE 1. The horizontal wind loaded length shall be that giving the most severe effect. Where there is only one adverse area (see 3.2.5) for the element or structure under consideration, the wind loaded length is the base length of the adverse area. Where there is more than one adverse area, as for continuous construction, the maximum effect shall be determined by consideration of any one adverse area or a combination of adverse areas, using the maximum wind gust speed V_d appropriate to the base length or the total combined base lengths. The remaining adverse areas, if any, and the relieving areas, are subjected to wind having the relieving wind speed V_r. The wind speeds V_d and V_r are given separately in 5.3.2 for bridges with and without live load.

NOTE 2. Where the bridge is located near the top of a hill, ridge, cliff or escarpment, the height above the local ground level shall allow for the significance of the topographic feature in accordance with 5.3.2.3.3. For bridges over tidal waters, the height above ground shall be measured from the mean water level.

NOTE 3. Vertical elements such as piers and towers shall be divided into units in accordance with the heights given in column 1 of tables 3 to 5, and the appropriate factor and wind speed shall be derived for each unit.

5.3.2.4.1 **The hourly speed factor S_c'.** Values of S_c' shall be taken from Table 3 for the appropriate height above ground and apply to sea shore locations. To allow for the distance of the site from the sea in the upwind direction for the load case considered, these values may be multiplied by the fetch correction factor, K_F, also given in Table 3, for the relevant height above ground.

5.3.2.4.2 **The hourly mean town reduction factor T_c.** Where the site is not situated in town terrain or is within 3km of the edge of a town in the upwind direction for the load case considered, T_c shall be taken as 1.0.

For sites in town terrain, advantage may be taken of the reduction factor T_c. The values of T_c shall be obtained from Table 5 for the height above ground and distance of the site from the edge of town terrain in the upwind direction for the load case considered.

5.3.2.5 **Maximum wind gust speed V_d on bridges with live load.** The maximum wind gust speed, V_d, on those parts of the bridge or its elements on which the application of wind loading increases the effect being considered shall be taken as:

a) for highways and foot/cycle track bridges, as specified in 5.3.2.1, but not exceeding 35 m/s;

b) for railway bridges, as specified in 5.3.2.1.

5.3.2.6 **Hourly mean wind speed for relieving areas V_r for bridges with live load.** Where wind on any part of a bridge or element gives relief to the member under consideration the effective coexistent value of wind gust speed V_r on the parts affording relief shall be taken as:

a) for highway and foot/cycle track bridges the lesser of:

$$35 \times \frac{S_c}{S_b} \text{ m/s and } V_r \text{ m/s as specified in 5.3.2.4}$$

b) for railway bridges, V_r m/s as specified in 5.3.2.4.

5.3.3 **Nominal transverse wind load.** The nominal transverse wind load P_t (in N) shall be taken as acting at the centroids of the appropriate areas and horizontally unless local conditions change the direction of the wind, and shall be derived from:

$$P_t = q\ A_1\ C_D$$

where

q is the dynamic pressure head taken as:

$0.613\ V_d^{\ 2}\ N/m^2$ for those parts of the bridge on which the application of wind loading increases the effect being considered; or

$0.613\ V_r^{\ 2}\ N/m^2$ for those parts where wind loading gives relief to the effect being considered

A_1 is the solid area (in m^2) (see 5.3.3.1)

C_D is the drag coefficient (see 5.3.3.2 to 5.3.3.6)

5.3.3.1 **Area A_1.** The area of the structure or element under consideration shall be the solid area in normal projected elevation, derived as follows.

5.3.3.1.1 **Erection stages for all bridges.** The area A_1, at all stages of construction, shall be the appropriate unshielded solid area of the structure or element.

5.3.3.1.2 **Highway and railway bridge superstructures with solid elevation.** For superstructures with or without live load, the area A_1 shall be derived using the appropriate value of d as given in figure 4.

(a) Superstructures without live load. P_t shall be derived separately for the areas of the following elements.

(1) For superstructures with open parapets:

(i) the superstructure, using depth d_1 from figure 4;

(ii) the windward parapet or safety fence;

(iii) the leeward parapet or safety fence.

Where there are more than two parapets or safety fences, irrespective of the width of the superstructure, only those two elements having the greatest unshielded effect shall be considered.

(2) For superstructures with solid parapets: the superstructure, using depth d_2 from figure 4 which includes the effects of the windward and leeward parapets. Where there are safety fences or additional parapets, P_t shall be derived separately for the solid areas of the elements above the top of the solid windward parapet.

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(b) Superstructures with live load. P_t shall be derived for the area A_1 as given in figure 4 which includes the effects of the superstructure, the live load and the windward and leeward parapets. Where there are safety fences or leeward parapets higher than the live load depth d_L, P_t shall be derived separately for the solid areas of the elements above the live load.

(c) Superstructures separated by an air gap. Where two generally similar superstructures are separated transversely by a gap not exceeding 1m, the nominal load on the windward structure shall be calculated as if it were a single structure, and that on the leeward superstructure shall be taken as the difference between the loads calculated for the combined and the windward structures (see note 7 to figure 5).

Where the superstructures are dissimilar or the air gap exceeds 1m, each superstructure shall be considered separately without any allowance for shielding.

5.3.3.1.3 **Foot/cycle track bridge superstructures with solid elevation.**

(a) Superstructures without live load. Where the ratio b/d as derived from figure 4 is greater than, or equal to, 1.1, the area A_1 shall comprise the solid area in normal projected elevation of the windward exposed face of the superstructure and parapet only. P_t shall be derived for this area, the leeward parapet being disregarded.

Where b/d is less than 1.1, the area A_1 shall be derived as specified in 5.3.3.1.2.

(b) Superstructures with live load. Where the ratio b/d as derived from figure 4 is greater than, or equal to, 1.1, the area A_1 shall comprise the solid area in normal projected elevation of the deck, the live load depth (taken as 1.25m above the footway) and the parts of the windward parapet more than 1.25m above the footway. P_t shall be derived for this area, the leeward parapet being disregarded.

Where b/d is less than 1.1, P_t shall be derived for the area A_1 as specified in 5.3.3.1.2.

5.3.3.1.4 **All truss girder bridge superstructures.**

(a) Superstructures without live load. The area A_1 for each truss, parapet, etc, shall be the solid area in normal projected elevation. The area A_1 for the deck shall be based on the full depth of the deck.

P_t shall be derived separately for the areas of the following elements:

(1) the windward and leeward truss girders;

(2) the deck;

(3) the windward and leeward parapets;

except that P_t need not be considered on projected areas of:

(4) the windward parapet screened by the windward truss, or vice versa;

(5) the deck screened by the windward truss, or vice versa;

(6) the leeward truss screened by the deck;

(7) the leeward parapet screened by the leeward truss, or vice versa.

(b) Superstructures with live load. The area A_1 for the deck, parapets, trusses, etc, shall be as for the superstructure without live load. The area A_1 for the live load shall be derived using the appropriate live load depth d_L as given in figure 4.

P_t shall be derived separately for the areas of the following elements:

(1) the windward and leeward truss girders;

(2) the deck;

(3) the windward and leeward parapets;

(4) the live load depth;

except that P_t need not be considered on projected areas of:

(5) the windward parapet screened by the windward truss, or vice versa;

(6) the deck screened by the windward truss, or vice versa;

(7) the live load screened by the windward truss or the parapet;

(8) the leeward truss screened by the live load and the deck;

(9) the leeward parapet screened by the leeward truss and the live load;

(10) the leeward truss screened by the leeward parapet and the live load.

5.3.3.1.5 **Parapets and safety fences.** For open and solid parapets and fences, P_t shall be derived for the solid area in normal projected elevation of the element under consideration.

5.3.3.1.6 **Piers.** P_t shall be derived for the solid area in normal projected elevation for each pier. No allowance shall be made for shielding.

5.3.3.2 **Drag coefficient C_D for erection stages for beams and girders.** In 5.3.3.2.1 to 5.3.3.2.5 requirements are specified for discrete beams or girders before deck construction or other infilling (eg shuttering).

5.3.3.2.1 **Single beam or box girder.** C_D shall be derived from figure 5 in accordance with the ratio b/d.

5.3.3.2.2 **Two or more beams or box girder.** C_D for each beam or box shall be derived from figure 5 without any allowance for shielding. Where the combined beams or boxes are required to be considered, C_D shall be derived as follows.

Where the ratio of the clear distance between the beams or boxes to the depth does not exceed 7, C_D for the combined structure shall be taken as 1.5 times C_D derived as specified in 5.3.3.2.1 for the single beam or box.

Where this ratio is greater than 7, C_D for the combined structure shall be taken as n times the value derived as specified in 5.3.3.2.1 for the single beam or box, where n is the number of beams or box girders.

5.3.3.2.3 **Single plate girder.** C_D shall be taken as 2.2.

5.3.3.2.4 **Two or more plate girders.** C_D for each girder shall be taken as 2.2 without any allowance for shielding. Where the combined girders are required to be considered, C_D for the combined structure shall be taken as 2(1 + c/20d), but not more than 4, where c is the distance centre to centre of adjacent girders, and d is the depth of the windward girder.

5.3.3.2.5 **Truss girders.** The discrete stages of erection shall be considered in accordance with 5.3.3.4.

5.3.3.3 **Drag coefficient C_D for all superstructures with solid elevation.** For superstructures with or without live load, C_D shall be derived from figure 5 in accordance with the ratio b/d as derived from figure 4. Drag coefficients shall be ascertained from wind tunnel tests, for any superstructures not encompassed within (a) and also for any special structures, such as shown in (b), of figure 4. See also notes 5 and 6 of figure 5.

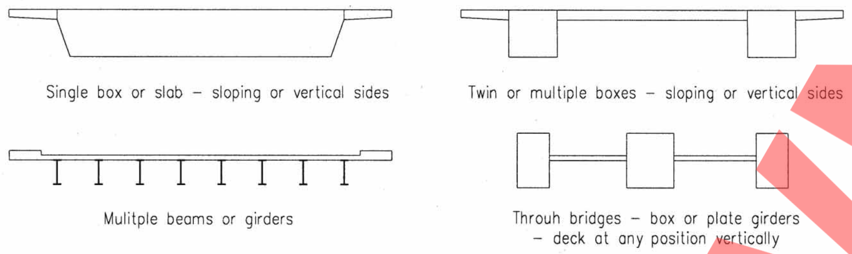


Figure 4(a) Typical superstructures to which figure 5 applies

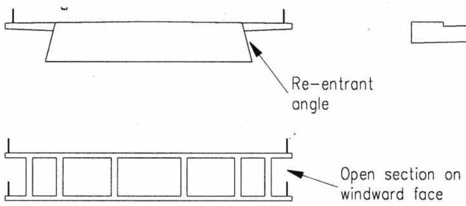
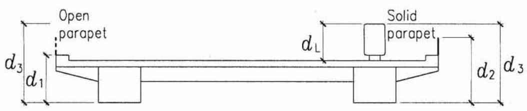


Figure 4(b) Typical superstructures that require wind tunnel tests

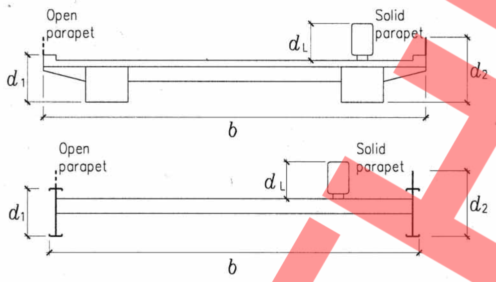


Parapet	Unloaded bridge	Live loaded bridge
Open	$d = d_1$	$d = d_3$
Solid	$d = d_2$	$d = d_2$ or d_3 whichever is greater

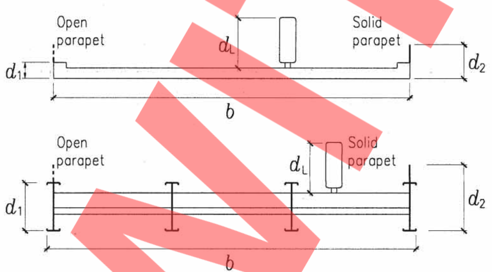
d_L = 2.5m above the highway carriageway, or
3.7m above the rail level, or
1.25m above footway or cycle track

Figure 4(c) Depth to be used in deriving A_1

(1) Superstructures where the depth of the superstructure (d_1 or d_2) exceeds d_L



(2) Superstructures where the depth of the superstructure (d_1 or d_2) is less than d_L



	Parapet	Superstructures without live load	Superstructures with live load
(1)	Open	$d = d_1$	$d = d_1$
(1)	Solid	$d = d_2$	$d = d_2$
(2)	Open	$d = d_1$	$d = d_L$
(2)	Solid	$d = d_2$	$d = d_L$

Figure 4(d) Depth d to be used in deriving C_D

Figure 4 Typical superstructures to which Figure 5 applies; those that require wind tunnel tests and depth d to be used for deriving A_1 and C_D

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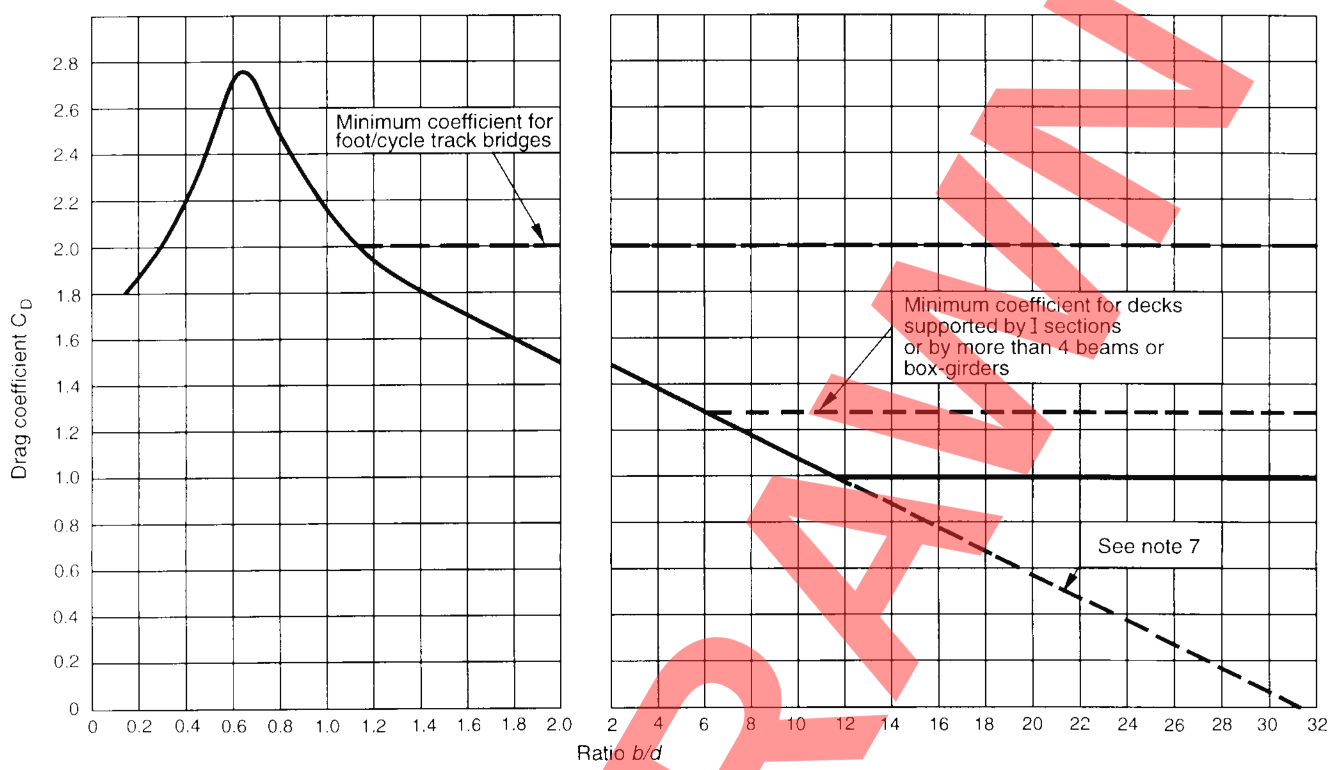


Figure 5. Drag coefficient C_D for superstructures with solid elevation

NOTES to figure 5

NOTE 1. These values are given for vertical elevations and for horizontal wind.

NOTE 2. Where the windward face is inclined to the vertical, the drag coefficient C_D may be reduced by 0.5% per degree of inclination from the vertical, subject to a maximum reduction of 30%.

NOTE 3. Where the windward face consists of a vertical and a sloping part or two sloping parts inclined at different angles, C_D shall be derived as follows.

For each part of the face, the depth shall be taken as the total vertical depth of the face (ie over all parts), and values of C_D derived in accordance with notes 1 and 2.

These separate values of C_D shall be applied to the appropriate area of the face.

NOTE 4. Where a superstructure is superelevated, C_D shall be increased by 3% per degree of inclination to the horizontal, but not by more than 25%.

NOTE 5. Where a superstructure is subject to inclined wind not exceeding 5° inclination, C_D shall be increased by 15%. Where the angle of inclination exceeds 5°, the drag coefficient shall be derived from tests.

NOTE 6. Where the superstructure is superelevated and also subject to inclined wind, the drag coefficient C_D shall be specially investigated.

NOTE 7. Where two generally similar superstructures are separated transversely by a gap not exceeding 1m, the drag coefficient for the combined superstructure shall be obtained by taking b as the combined width of the superstructure. In assessing the distribution of the transverse wind load between the two separate superstructures (see 5.3.3.1.2(c)) the drag coefficient C_D for the windward superstructure shall be taken as that of the windward superstructure alone, and the drag coefficient C_D of the leeward superstructure shall be the difference between that of the combined superstructure and that of the windward superstructure. for the purposes of determining this distribution, if b/d is greater than 12 the broken line in figure 5 shall be used to derive C_D. The load on the leeward structure is generally opposite in sign to that on the windward superstructure.

Where the gap exceeds 1m, C_D for each superstructure shall be derived separately, without any allowance being made for shielding.

5.3.3.4 **Drag coefficient C_D for all truss girder superstructures**

(a) **Superstructures without live load.** The drag coefficient C_D for each truss and for the deck shall be derived as follows.

(1) For a windward truss, C_D shall be taken from table 6.

Table 6. Drag coefficient C_D for a single truss

Solidity ratio	For flatsided members	For round members where d is diameter of member	
		$dV_d < 6m^2/s$ or dV'_c	$dV_d \geq 6m^2/s$ or dV'_c
0.1	1.9	1.2	0.7
0.2	1.8	1.2	0.8
0.3	1.7	1.2	0.8
0.4	1.7	1.1	0.8
0.5	1.6	1.1	0.8

NOTE: On relieving areas use V_r instead of V_d

The solidity ratio of the truss is the ratio of the net area to the overall area of the truss.

(2) For the leeward truss of a superstructure with two trusses the drag coefficient shall be taken as ηC_D. Values of η are given in table 7.

Table 7. Shielding factor η

Spacing ratio	Value of η for solidity ratio of:				
	0.1	0.2	0.3	0.4	0.5
Less than 1	1.0	0.90	0.80	0.60	0.45
2	1.0	0.90	0.80	0.65	0.50
3	1.0	0.95	0.80	0.70	0.55
4	1.0	0.95	0.85	0.70	0.60
5	1.0	0.95	0.85	0.75	0.65
6	1.0	0.95	0.90	0.80	0.70

The spacing ratio is the distance between centres of trustess divided by the depth of the windward truss.

(3) Where a superstructure has more than two trusses, the drag coefficient for the truss adjacent to the windward truss shall be derived as specified in (2). The coefficient for all other trusses shall be taken as equal to this value.

(4) For the deck construction the drag coefficient C_D shall be taken as 1.1.

(b) **Superstructures with live load.** The drag coefficient C_D for each truss and for the deck shall be as for the superstructure without live load. C_D for unshielded parts of the live load shall be taken as 1.45.

5.3.3.5 **Drag coefficient C_D for parapets and safety fences.** For the windward parapet or fence, C_D shall be taken from table 8.

Where there are two parapets or fences on a bridge, the value of C_D for the leeward element shall be taken as equal to that of the windward element. Where there are more than two parapets or fences the values of C_D shall be taken from table 8 for the two elements having the greatest unshielded effect.

Where parapets have mesh panels, consideration shall be given to the possibility of the mesh becoming filled with ice. In these circumstances, the parapet shall be considered as solid.

5.3.3.6 **Drag coefficient C_D for piers.** The drag coefficient shall be taken from table 9. For piers with cross sections dissimilar to those given in table 9, wind tunnel tests shall be carried out.

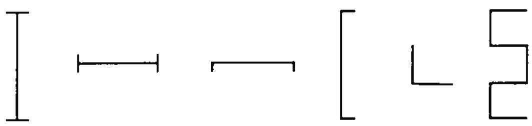
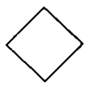
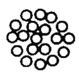
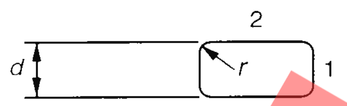
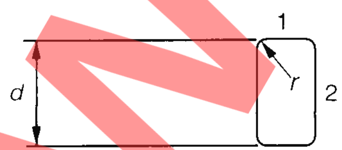
C_D shall be derived for each pier, without reduction for shielding.

5.3.4 **Nominal longitudinal wind load.** The nominal longitudinal wind load P_L (in N), taken as acting at the centroids of the appropriate areas, shall be the more severe of either:

(a) the nominal longitudinal wind load on the superstructure, P_{Ls}, alone; or

(b) the sum of the nominal longitudinal wind load on the superstructure, P_{Ls}, and the nominal longitudinal wind load on the live load, P_{LL}, derived separately, as specified as appropriate in 5.3.4.1 to 5.3.4.3.

Table 8. Drag coefficient C_D for parapets and safety fences

	Circular sections $dV_d < 6$	1.2
(where V_d in m/s and d in m)	$dV_d > 6$	0.7
NOTE: On relieving areas use V_r instead of V_d		
	Flat members with rectangular corners, crash barrier rails and solid parapets	2.2
	Square members diagonal to wind	1.5
	Circular stranded cables	1.2
	Rectangular members with circular corners $r > d/12$	1.1*
	Rectangular members with circular corners $r > d/12$	1.5*
	Rectangular members with circular corners $r > d/24$	2.1

*For sections with intermediate proportions, C_D may be obtained by interpolation

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Table 9. Drag coefficient C_D for piers

Plan shape	$\frac{t}{b}$	C_D for pier $\frac{\text{height}}{\text{breadth}}$ ratios of 1	2	4	6	10	20	40
WIND → (rectangle, t along wind, b across)	$\leqslant \frac{1}{4}$	1.3	1.4	1.5	1.6	1.7	1.9	2.1
→ (rectangle)	$\frac{1}{3}$ $\frac{1}{2}$	1.3	1.4	1.5	1.6	1.8	2.0	2.2
→ (rectangle)	$\frac{2}{3}$	1.3	1.4	1.5	1.6	1.8	2.0	2.2
→ (square)	1	1.2	1.3	1.4	1.5	1.6	1.8	2.0
→ (rectangle)	$1\frac{1}{2}$	1.0	1.1	1.2	1.3	1.4	1.5	1.7
→ (rectangle)	2	0.8	0.9	1.0	1.1	1.2	1.3	1.4
→ (rectangle)	3	0.8	0.8	0.8	0.9	0.9	1.0	1.2
→ (rectangle, t, b)	$\geqslant 4$	0.8	0.8	0.8	0.8	0.8	0.9	1.1
SQUARE OR OCTAGONAL		1.0	1.1	1.1	1.2	1.2	1.3	1.4
12 SIDED POLYGON		0.7	0.8	0.9	0.9	1.0	1.1	1.3
CIRCLE WITH SMOOTH SURFACE WHERE $tV_d \geq$ 6 m²/s		0.5	0.5	0.5	0.5	0.5	0.6	0.6
CIRCLE WITH SMOOTH SURFACE WHERE $tV_d <$ 6 m²/s. ALSO CIRCLE WITH ROUGH SURFACE OR WITH PROJECTIONS		0.7	0.7	0.8	0.8	0.9	1.0	1.2

NOTE 1. After erection of the superstructure, C_D shall be derived for a height/breadth ratio of 40.

NOTE 2. For a rectangular pier with radiused corners, the value of C_D derived from table 9 shall be multiplied by (1-1.5r/b) or 0.5, whichever is greater.

NOTE 3. For a pier with triangular nosings, C_D shall be derived as for the rectangle encompassing the outer edges of the pier.

NOTE 4. For a pier tapering with height, C_D shall be derived for each of the unit heights into which the support has been subdivided (see Note 3 to tables 3 to 5). Mean values of t and b for each unit height shall be used to evaluate t/b. The overall pier height and the mean breadth of each unit height shall be used to evaluate height/breadth.

NOTE 5: On relieving areas use V_r instead of V_d.

5.3.4.1 **All superstructures with solid elevation**

$P_{LS} = 0.25qA_1C_D$

where

q is as defined in 5.3.3, the appropriate value of V_d or V_r for superstructures with or without live load being adopted

A_1 is as defined in 5.3.3.1.2 and 5.3.3.1.3 for the superstructure alone

C_D is the drag coefficient for the superstructure (excluding reduction for inclined webs) as defined in 5.3.3.3, but not less than 1.3.

5.3.4.2 **All truss girder superstructures**

$P_{LS} = 0.5qA_1C_D$

where

q is as defined in 5.3.3, the appropriate value of V_d or V_r for structures with or without live load being adopted

A_1 is as defined in 5.3.3.1.4 (a)

C_D is as defined in 5.3.3.4 (a), C_D being adopted where appropriate.

5.3.4.3 **Live load on all superstructures**

$P_{LL} = 0.5qA_1C_D$

where

q is as defined in 5.3.3

A_1 is the area of live load derived from the depth d_L as given in figure 4 and the appropriate horizontal wind loaded length as defined in note 1 to table 3.

$C_D = 1.45$

5.3.4.4 **Parapets and safety fences**

(a) With vertical infill members, $P_L = 0.8P_t$

(b) With two or three horizontal rails only, $P_L = 0.4P_t$

(c) With mesh panels, $P_L = 0.6P_t$

where P_t is the appropriate nominal transverse wind load on the element.

5.3.4.5 **Cantilever brackets extending outside main girders or trusses.** P_L is the load derived from a horizontal wind acting at 45° to the longitudinal axis on the areas of each bracket not shielded by a fascia girder or adjacent bracket. The drag coefficient C_D shall be taken from table 8.

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5.3.4.6 **Piers.** The load derived from a horizontal wind acting along the longitudinal axis of the bridge shall be taken as

$$P_L = qA_2C_D$$

where

q is as defined in 5.3.3

A_2 is the solid area in projected elevation normal to the longitudinal wind direction (in m^2)

C_D is the drag coefficient, taken from table 9, with values of b and t interchanged.

5.3.5 **Nominal vertical wind load.** An upward or downward nominal vertical wind load P_v (in N), acting at the centroids of the appropriate areas, for all superstructures shall be derived from

$$P_v = q\ A_3C_L$$

where

q is as defined in 5.3.3

A_3 is the area in plan (in m^2)

C_L is the lift coefficient defined as:

$$C_L = 0.75\left[1 - \frac{b}{20d}(1 - 0.2\alpha)\right]$$

but $0.15 < C_L < 0.90$

where

α is the sum of the angle of superelevation and the wind inclination to be considered (taken as a positive number in the above equation, irrespective of the inclination and superelevation)

α exceeds 10°, the value of C_L shall be determined by testing

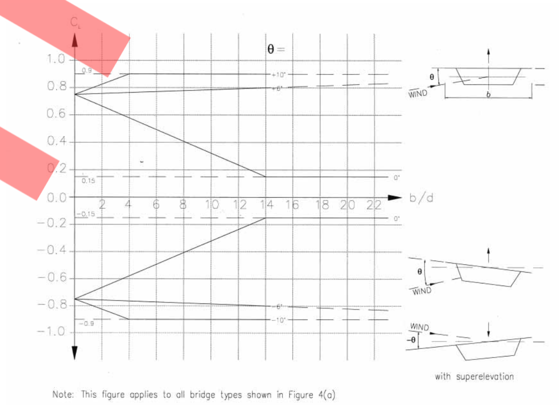


Figure 6. Lift coefficient C_L

5.3.6 **Load combination.** The wind loads P_t, P_L and P_V shall be considered in combination with the other loads in combination 2, as appropriate, taking four separate cases:

(a) P_t alone;

(b) P_t in combination with $\pm P_v$;

(c) P_L alone;

(d) $0.5P_t$ in combination with $P_L \pm 0.5P_v$.

5.3.7 **Design loads.** For design loads the factor γ_{fL} shall be taken as follows:

Wind considered with	For the ultimate limit state	For the serviceability limit state
(a) erection	1.1	1.0
(b) dead load plus superimposed dead load only, and for members primarily resisting wind loads	1.4	1.0
(c) appropriate combination 2 loads	1.1	1.0
(d) relieving effects of wind	1.0	1.0

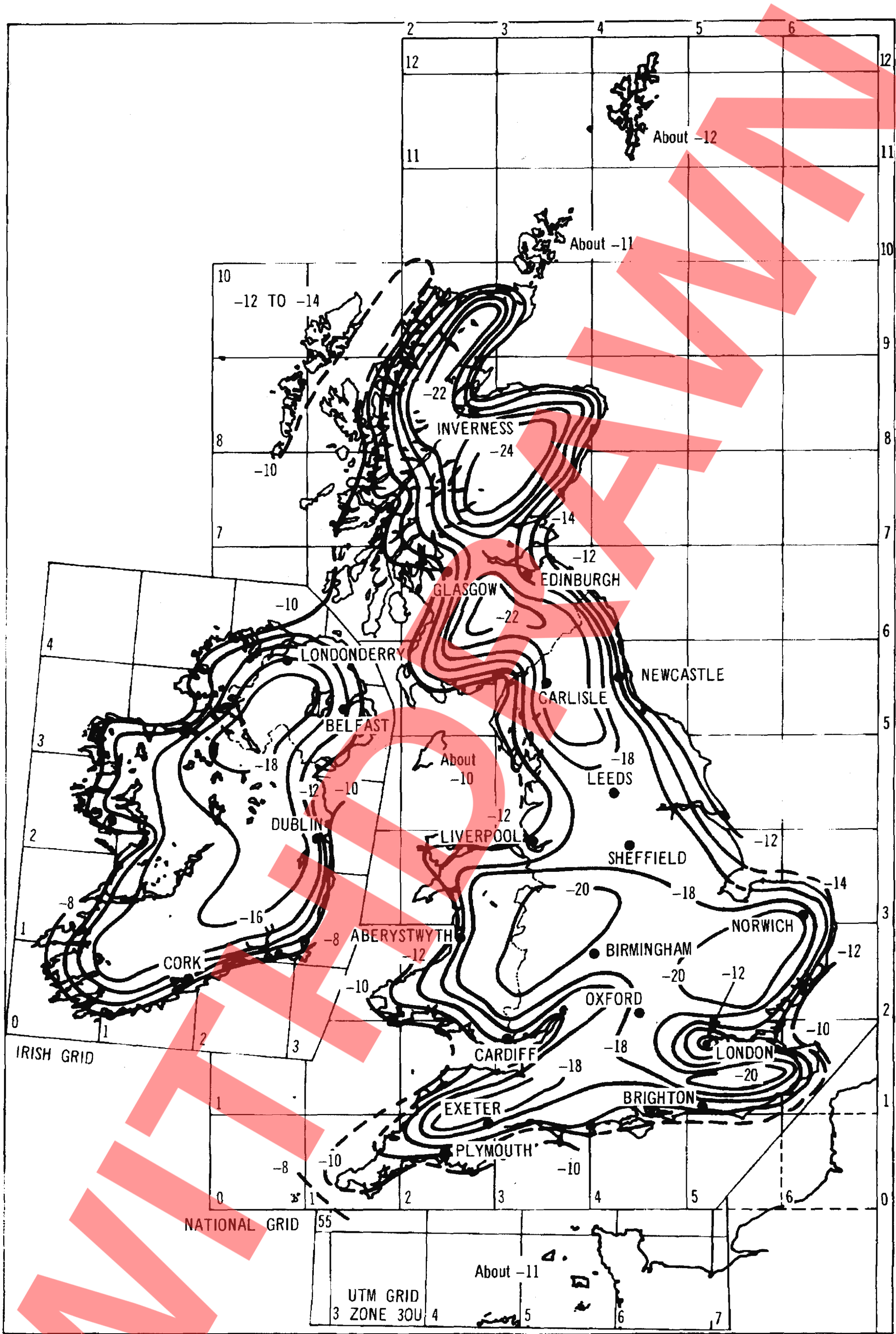
5.3.8 **Overturning effects.** Where overturning effects are being investigated the wind load shall also be considered in combination with vertical traffic live load. Where the vertical traffic live load has a relieving effect, this load shall be limited to one notional lane or one track only, and shall have the following value:

on highway bridges, not more than 6kN/m of bridge;

on railway bridges, not more than 12kN/m of bridge.

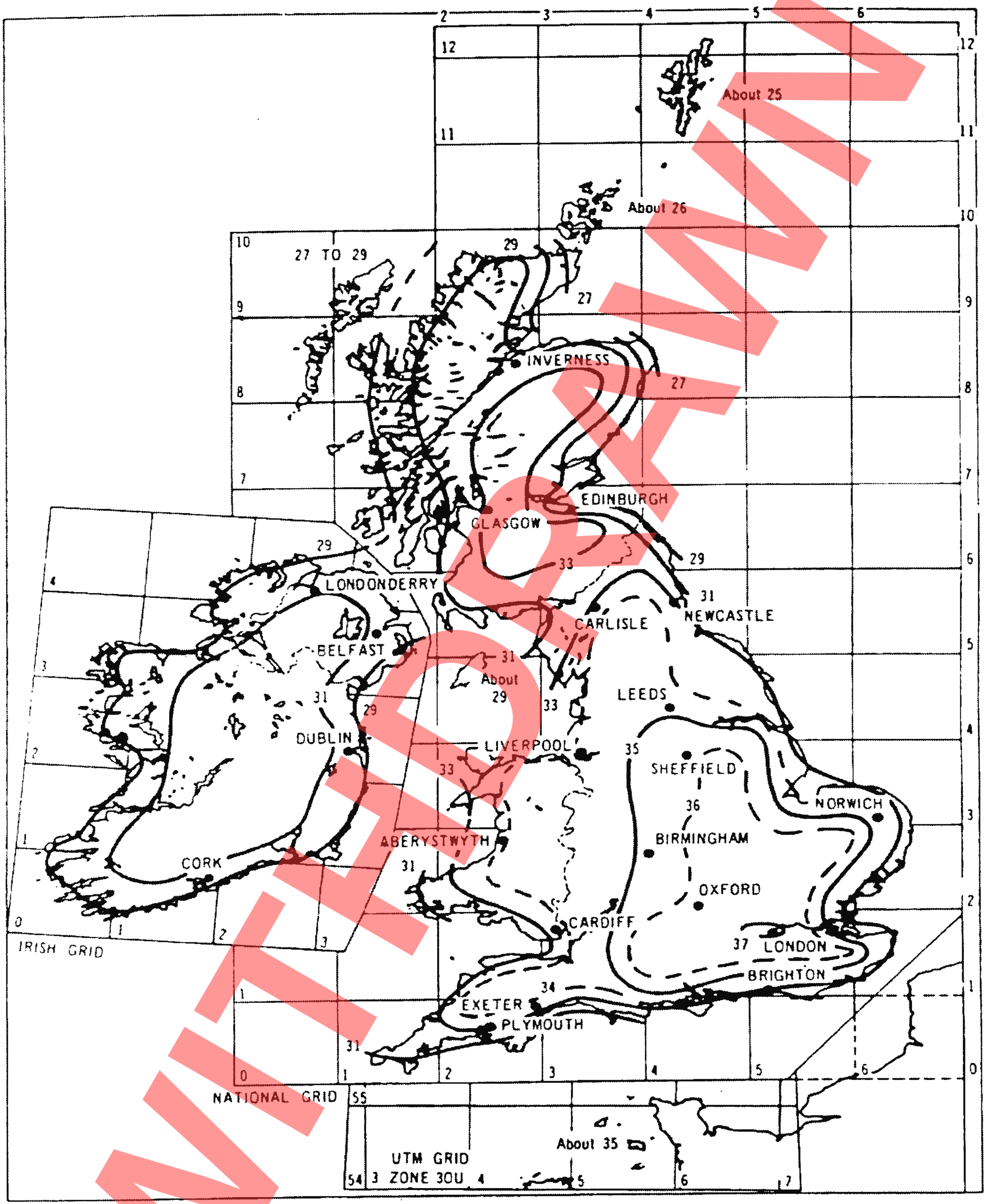
5.3.8.1 **Load factor for relieving vertical live load.** For live load producing a relieving effect, γ_{fL} for both ultimate limit states and serviceability limit states shall be taken as 1.0.

5.3.9 **Aerodynamic effects.** Aerodynamic effects shall be taken into account as and when required by the appropriate DMRB standard or as agreed with the appropriate authority.



NOTE. The isotherms are derived from Meteorological Office data

Figure 7. Isotherms of minimum shade air temperature (in °C)



NOTE. The isotherms are derived from Meteorological Office data

Figure 8. Isotherms of maximum shade air temperature (in °C)

5.4 Temperature

5.4.1 **General.** Daily and seasonal fluctuations in shade air temperature, solar radiation, re-radiation etc, cause the following:

(a) Changes in the effective temperature of a bridge superstructure which, in turn govern its movement.

The effective temperature is a theoretical temperature derived by weighting and adding temperatures at various levels within the superstructure. The weighting is in the ratio of the area of cross-section at the various levels to the total area of cross-section of the superstructure. (See also Appendix C). Over a period of time there will be a minimum, a maximum, and a range of effective bridge temperature, resulting in loads and/or load effects within the superstructure due to:

(1) restraint of associated expansion or contraction by the form of construction (eg portal frame, arch, flexible pier, elastomeric bearings) referred to as temperature restraint; and

(2) friction at roller or sliding bearings where the form of the structure permits associated expansion and contraction, referred to as frictional bearing restraint.

(b) Differences in temperature between the top surface and other levels in the superstructure. These are referred to as temperature differences and they result in loads and/or load effects within the superstructure.

Effective bridge temperatures for design purposes are derived from the isotherms of shade air temperature shown in figures 7 and 8. These shade air temperatures are appropriate to mean sea level in open country and a 120-year return period.

NOTE 1. It is only possible to relate the effective bridge temperature to the shade air temperature during a period of extreme environmental conditions.

NOTE 2. Daily and seasonal fluctuations in shade air temperature, solar radiation, etc., also cause changes in the temperature of other structural elements such as piers, towers and cables. In the absence of codified values for effective temperatures of, and temperature differences within, these elements, appropriate values should be derived from first principles.

5.4.2 **Minimum and maximum shade air temperatures.** For all bridges, 1 in 120 year minimum and maximum shade air temperatures for the location of the bridge shall be obtained from the maps of isotherms shown in figures 7 and 8.

For foot/cycle track bridges, subject to the agreement of the appropriate authority, a return period of 50 years may be adopted, and the shade air temperatures may be reduced as specified in 5.4.2.1.

Carriageway joints and similar equipment that will be replaced during the life of the structure may be designed for temperatures related to a 50-year return period and the shade air temperature may be reduced as specified in 5.4.2.1.

During erection, a 50 year return period may be adopted for all bridges and the shade air temperatures may be reduced as specified in 5.4.2.1. Alternatively, where a particular erection will be completed within a period of one or two days for which reliable shade air temperature and temperature range predictions can be made, these may be adopted.

5.4.2.1 **Adjustment for a 50-year return period.** The minimum shade air temperature, as derived from figure 7, shall be adjusted by the addition of 2°C.

The maximum shade air temperature, as derived from figure 8, shall be adjusted by the subtraction of 2°C.

5.4.2.2 **Adjustment for height above mean sea level.** The values of shade air temperature shall be adjusted for height above sea level by subtracting 0.5°C per 100m height for minimum shade air temperatures and 1.0°C per 100m height for maximum shade air temperatures.

5.4.2.3 **Divergence from minimum shade air temperature.** There are locations where the minimum values diverge from the values given in figure 7 as, for example, frost pockets and sheltered low lying areas where the minimum may be substantially lower, or in urban areas (except London) and coastal sites, where the minimum may be higher, than that indicated by figure 7. These divergences shall be taken into consideration. (In coastal areas, values are likely to be 1°C higher than the values given in figure 7.)

5.4.3 **Minimum and maximum effective bridge temperatures.** The minimum and maximum effective bridge temperatures for different types of construction shall be derived from the minimum and maximum shade air temperatures by reference to tables 10 and 11 respectively. The different types of construction are as shown in figure 9. The minimum and maximum effective bridge temperatures will be either 1 in 120 year or 1 in 50 year values depending on the return period adopted for the shade air temperature.

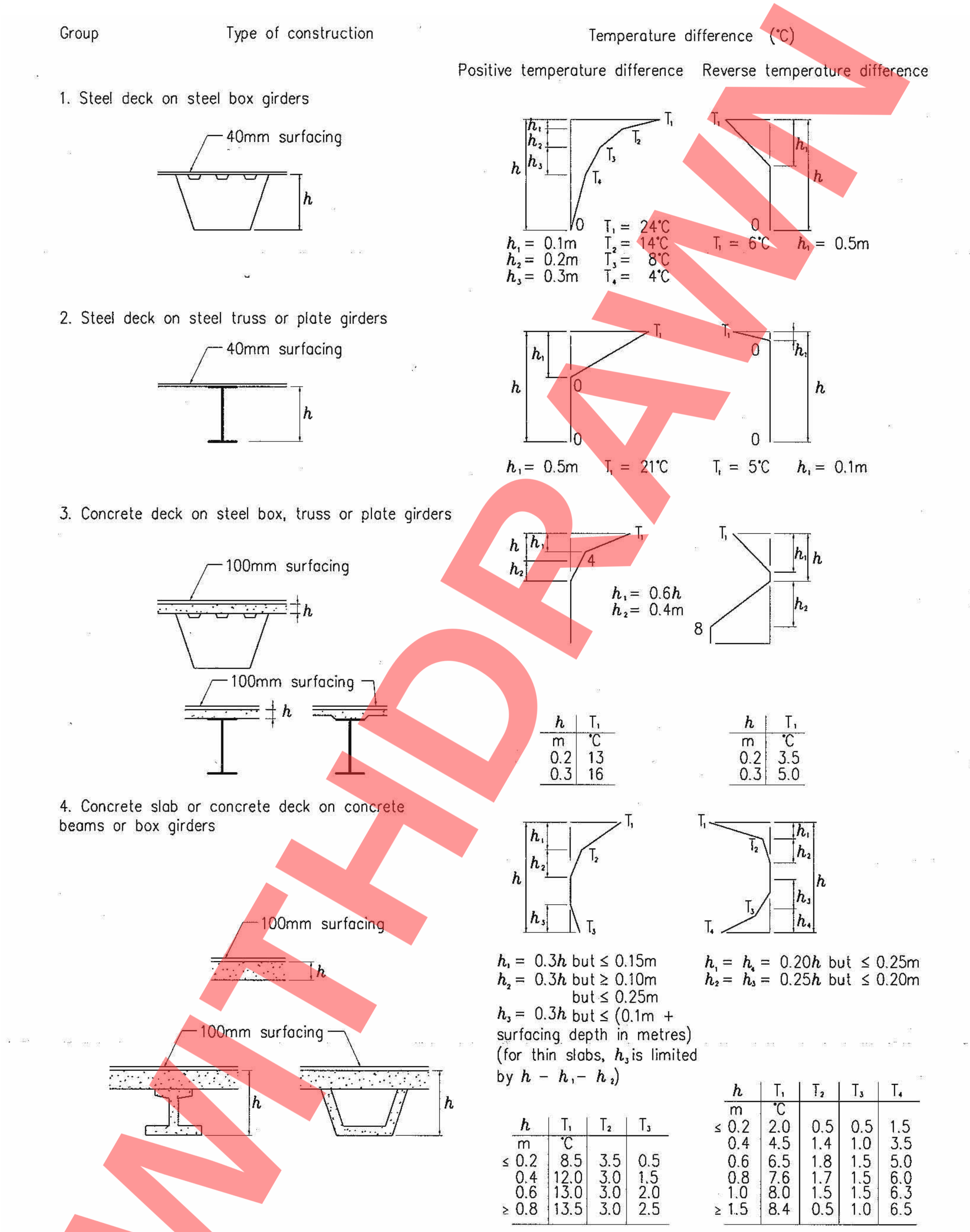
5.4.3.1 **Adjustment for thickness of surfacing.** The effective bridge temperatures are dependent on the depth of surfacing on the bridge deck and the values given in tables 10 and 11 assume depths of 40mm for groups 1 and 2 and 100mm for groups 3 and 4. Where the depth of surfacing differs from these values, the minimum and maximum effective bridge temperatures may be adjusted by the amounts given in table 12.

Table 10. Minimum effective bridge temperature

Minimum shade air temperature	Minimum effective bridge temperature			
	Type of superstructure			
	Group 1	Group 2	Group 3	Group 4
°C	°C	°C	°C	°C
-24	-26	-25	-19	-14
-23	-25	-24	-18	-13
-22	-24	-23	-18	-13
-21	-23	-22	-17	-12
-20	-22	-21	-17	-12
-19	-21	-20	-16	-11
-18	-20	-19	-15	-11
-17	-19	-18	-15	-10
-16	-18	-17	-14	-10
-15	-17	-16	-13	- 9
-14	-16	-15	-12	- 9
-13	-15	-14	-11	- 8
-12	-14	-13	-10	- 7
-11	-13	-12	-10	- 6
-10	-12	-11	- 9	- 6
- 9	-11	-10	- 8	- 5
- 8	-10	- 9	- 7	- 4
- 7	- 9	- 8	- 6	- 3
- 6	- 8	- 7	- 5	- 3
- 5	- 7	- 6	- 4	- 2

Table 11. Maximum effective bridge temperature

Maximum shade air temperature	Maximum effective bridge temperature			
	Type of superstructure			
	Group 1	Group 2	Group 3	Group 4
°C	°C	°C	°C	°C
24	38	34	31	27
25	39	35	32	28
26	40	36	33	29
27	41	37	34	29
28	42	38	34	30
29	43	39	35	31
30	44	40	36	32
31	45	41	36	32
32	46	42	37	33
33	47	43	37	33
34	48	44	38	34
35	49	45	39	35
36	50	46	39	36
37	51	47	40	36
38	52	48	40	37



h	T_1
m	°C
0.2	13
0.3	16

h	T_1
m	°C
0.2	3.5
0.3	5.0

h	T_1	T_2	T_3
m	°C		
≤ 0.2	8.5	3.5	0.5
0.4	12.0	3.0	1.5
0.6	13.0	3.0	2.0
≥ 0.8	13.5	3.0	2.5

h	T_1	T_2	T_3	T_4
m	°C			
≤ 0.2	2.0	0.5	0.5	1.5
0.4	4.5	1.4	1.0	3.5
0.6	6.5	1.8	1.5	5.0
0.8	7.6	1.7	1.5	6.0
1.0	8.0	1.5	1.5	6.3
≥ 1.5	8.4	0.5	1.0	6.5

Figure 9. Temperature difference for different types of construction

Table 12. Adjustment to effective bridge temperature for deck surfacing

Deck Surface	Addition to minimum effective bridge temperature				Addition to maximum effective bridge temperature			
	Group 1	Group 2	Group 3	Group 4	Group 1	Group 2	Group 3	Group 4
	°C	°C	°C	°C	°C	°C	°C	°C
Unsurfaced	0	0	-3	-1	+4	+2	0	0
Waterproofed#	0	0	-3	-1	+4	+2	+4	+2
40mm surfacing*	0	0	-2	-1	0	0	+2	+1
100mm surfacing*	N/A	N/A	0	0	N/A	N/A	0	0
200mm surfacing*	N/A	N/A	+3	+1	N/A	N/A	-4	-2

* Surfacing depths include waterproofing.
# Waterproofed deck values are conservative, assuming dark material; there may be some alleviations when light coloured waterproofing is used; specialist advice should be sought if required.

5.4.4 **Range of effective bridge temperature.** In determining load effects due to temperature restraint, the effective bridge temperature at the time the structure is effectively restrained shall be taken as datum in calculating expansion up to the maximum effective bridge temperature and contraction down to the minimum effective bridge temperature.

5.4.5 **Temperature difference.** Effects of temperature differences within the superstructure shall be derived from the data given in figure 9.

Positive temperature differences occur when conditions are such that solar radiation and other effects cause a gain in heat through the top surface of the superstructure. Conversely, reverse temperature differences occur when conditions are such that heat is lost from the top surface of the bridge deck as a result of re-radiation and other effects.

5.4.5.1 **Adjustment for thickness of surfacing.** Temperature differences are sensitive to the thickness of surfacing, and the data given in figure 9 assume depths of 40mm for groups 1 and 2 and 100m for groups 3 and 4. For other depths of surfacing different values will apply. Values for other thicknesses of surfacing are given in Appendix C.

5.4.5.2 **Application with effective bridge temperatures.** Maximum positive temperature differences shall be considered to coexist with effective bridge temperatures at above 25°C (groups 1 and 2) and 15°C (groups 3 and 4). Maximum reversed temperature differences shall be considered to coexist with effective bridge temperatures up to 8°C below the maximum for groups 1 and 2, up to 4°C below the maximum for group 3, and up to 2°C below the maximum for group 4.

The method of deriving temperatures to be used in the calculation of loads and/or load effects within the superstructure is given in Appendix C.

5.4.6 **Coefficient of thermal expansion.** For the purpose of calculating temperature effects, the coefficients of thermal expansion for structural steel and for concrete may be taken as 12×10^{-6}/°C, except when limestone aggregates are used in concrete, when a value of 9×10^{-6}/°C shall be adopted for the concrete.

5.4.7 **Nominal values**

5.4.7.1 **Nominal range of movement.** The effective bridge temperature at the time the structure is attached to those parts permitting movement shall be taken as datum and the nominal range of movement shall be calculated for expansion up to the maximum effective bridge temperature and for contraction down to the minimum effective bridge temperature.

5.4.7.2 **Nominal load for temperature restraint.** The load due to temperature restraint of expansion or contraction for the appropriate effective bridge temperature range (see 5.4.4.) shall be taken as the nominal load.

Where temperature restraint is accompanied by elastic deformations in flexible piers and elastomeric bearings, the nominal load shall be derived as specified in 5.4.7.2.1 to 5.4.7.2.2.

5.4.7.2.1 **Flexure of piers.** For flexible piers pinned at one end and fixed at the other, or fixed at both ends, the load required to displace the pier by the amount of expansion or contraction for the appropriate effective bridge temperature range (see 5.4.4) shall be taken as the nominal load.

5.4.7.2.2 **Elastomeric bearings.** For temperature restraint accommodated by shear in an elastomer, the load required to displace the elastomer by the amount of expansion or contraction for the appropriate effective bridge temperature range (see 5.4.4) shall be taken as the nominal load.

The nominal load shall be determined in accordance with 5.14.2.6 of BS 5400: Part 9: Section 9.1: 1983.

5.4.7.3 **Nominal load for frictional bearing restraint.** The nominal load due to frictional bearing restraint shall be derived from the nominal dead load (see 5.1.1), the nominal superimposed dead load (see 5.2.1) and the snow load (see 5.7.1), using the appropriate coefficient of friction given in tables 2 and 3 of BS 5400: Part 9: Section 9.1: 1983.

5.4.7.4 **Nominal effects of temperature difference.** The effects of temperature difference shall be regarded as nominal values.

5.4.8 **Design values**

5.4.8.1 **Design range of movement.** The design range of movement shall be taken as 1.3 times the appropriate nominal value for the ultimate limit state and 1.0 times the nominal value for the serviceability limit state.

For the purpose of this clause the ultimate limit state shall be regarded as a condition where expansion or contraction beyond the serviceability range up to the ultimate range would cause collapse or substantial damage to main structural members. Where expansion or contraction beyond the serviceability range will not have such consequences, only the serviceability range need to be provided for.

5.4.8.2 **Design load for temperature restraint.** For combination 3, γ_{fL} shall be taken as follows:

For the ultimate limit state	For the serviceability limit state
1.30	1.00

5.4.8.3 **Design load for frictional bearing restraint.** For combination 5, γ_{fL} shall be taken as follows:

For the ultimate limit state	For the serviceability limit state
1.30	1.00

5.4.8.3.1 **Associated vertical design load.** The design dead load (see 5.1.2) and design superimposed dead load (see 5.2.2) shall be considered in conjunction with the design load due to frictional bearing restraint.

5.4.8.4 **Design effects of temperature difference.** For combination 3, γ_{fL} shall be taken as follows:

For the ultimate limit state	For the serviceability limit state
1.30	0.80

5.5 **Effects of shrinkage and creep, residual stresses, etc.** Where it is necessary to take into account the effects of shrinkage or creep in concrete, stresses in steel due to rolling, welding or lack of fit, variations in the accuracy of bearing levels and similar sources of strain arising from the nature of the material or its manufacture or from circumstances associated with fabrication and erection, requirements are specified in the appropriate Parts of this standard.

5.6 **Differential settlement.** Where differential settlement is likely to affect the structure in whole or in part, the effects of this shall be taken into account.

5.6.1 **Assessment of differential settlement.** In assessing the amount of differential movement to be provided for, the engineer shall take into account the extent to which its effect will be observed and remedied before damage ensues. The nominal value selected shall be agreed with the appropriate authority.

5.6.2 **Load factors.** The values of γ_{fL} shall be chosen in accordance with the degree of reliability of assessment, taking account of the general basis of probability of occurrence set out in Part 1 of this standard and the provisions for ensuring remedial action.

5.6.3 **Design load**. The values of γ_{fL} given below are based on the assumption that the nominal values of settlement assumed have a 95% probability of not being exceeded during the design life of the structure. The factor γ_{fL} to be applied to the effects of differential settlement, shall be taken for all five load combinations as follows:

For the ultimate limit state	For the serviceability limit state
1.20	1.00

5.7 **Exceptional loads.** Where other loads not specified in this standard are likely to be encountered, eg the effects of abnormal indivisible live loads, earthquakes, stream flows or ice packs, these shall be taken into account. The nominal loading to be adopted shall have a value in accordance with the general basis of probability of occurrence set out in Part 1 of this standard and shall be agreed with the appropriate authority.

5.7.1 **Snow load.** Snow loading should be considered in accordance with local condition; for those prevailing in Great Britain, this loading may generally be ignored in combinations 1 to 4 (see 4.4.1 to 4.4.4), but there are circumstances, eg for opening bridges or where dead load stability is critical, when consideration should be given to it.

5.7.2 **Design loads.** For abnormal indivisible live loads, γ_{fL} shall be taken as specified for HB loading (see 6.3.4). For other exceptional design loads, γ_{fL} shall be assessed in accordance with the general basis of probability of occurrence set out in Part 1 of this standard and shall be agreed with the appropriate authority.

5.8 Earth pressure on retaining structures

5.8.1 Filling material

5.8.1.1 **Nominal load.** Where filling material is retained by abutments or other parts of the structure, the loads calculated by soil mechanics principles from the properties of the filling material shall be regarded as nominal loads.

The nominal loads initially assumed shall be accurately checked with the properties of the material to be used in construction and, where necessary, adjustments shall be made to reconcile any discrepancies.

Consideration shall be given to the possibility that the filling material may become saturated or may be removed in whole or in part from either side of the fill-retaining part of the structure.

5.8.1.2 **Design load.** For all five design load combinations, γ_{fL} shall be taken as follows:

	For the ultimate limit state	For the serviceability limit state
Vertical loads	1.2	1.0
Non-vertical loads	1.5	1.0

5.8.1.3 **Alternative load factor**. Where the structure or element under consideration is such that the application γ_{fL} as given in 5.8.1.2 for the ultimate limit state causes a less severe total effect (see 3.2.6) than would be the case if γ_{fL}, applied to all parts of the filling material, had been taken as 1.0, values of 1.0 shall be adopted.

5.8.2 **Live load surcharge.** The effects of live load surcharge shall be taken into consideration.

5.8.2.1 **Nominal load.** In the absence of more exact calculations the nominal load due to live load surcharge for suitable material properly consolidated may be assumed to be

(a) for HA loading: 10kN/m^2;

(b) for HB loading
45 units: 20 kN/m^2 (intermediate values
30 units: 12 kN/m^2 by interpolation);

(c) for RU loading: 50 kN/m^2 on areas occupied by tracks;

(d) for RL loading: 30 kN/m^2 on areas occupied by tracks.

5.8.2.2 **Design load.** For combinations 1 to 5 γ_{fL} shall be as specified in 5.8.1.2.

5.9 **Erection loads.** For the ultimate limit state, erection loads shall be considered in accordance with 5.9.1 to 5.9.5.

For the serviceability limit state, nothing shall be done during erection that will cause damage to the permanent structure or will alter its response in service from that considered in design.

5.9.1 **Temporary loads**

5.9.1.1 **Nominal loads.** The total weight of all temporary materials, plant and equipment to be used during erection shall be taken into account. This shall be accurately assessed to ensure that the loading is not underestimated.

5.9.1.2 **Design loads.** For the ultimate limit state for combinations 2 and 3, γ_{fL} shall be taken as 1.15 except as specified in 5.9.1.3. For the serviceability limit state for combinations 2 and 3, γ_{fL} shall be taken as 1.00.

5.9.1.3 **Relieving effect.** Where any temporary materials have a relieving effect, and have not been introduced specifically for this purpose, they shall be considered not to be acting. Where, however, they have been so introduced, precautions shall be taken to ensure that they are not inadvertently removed during the period for which they are required. The weight of these materials shall also be accurately assessed to ensure that the loading is not over-estimated. This value shall be taken as the design load.

5.9.2 **Permanent loads**

5.9.2.1 **Nominal loads.** All dead and superimposed dead loads affecting the structure at each stage of erection shall be taken into account.

The effects of the method of erection of permanent materials shall be considered and due allowance shall be made for impact loading or shock loading.

5.9.2.2 **Design loads.** The design loads due to permanent loads for the serviceability limit state and the ultimate state for combinations 2 and 3 shall be as specified in 5.1.2 and 5.2.2 respectively.

5.9.3 **Disposition of permanent and temporary loads.** The disposition of all permanent and temporary loads at all stages of erection shall be taken into consideration and due allowance shall be made for possible inaccuracies in their location. Precautions shall be taken to ensure that the assumed disposition is maintained during erection.

5.9.4 **Wind and temperature effects.** Wind and temperature effects shall be considered in accordance with 5.3 and 5.4, respectively.

5.9.5 **Snow and ice loads.** When climatic conditions are such that there is a possibility of snowfall or of icing, an appropriate allowance shall be made. Generally, a distributed load of 500 N/m² may be taken as adequate but may require to be increased for regions where there is a possibility of snowfalls and extremes of low temperature over a long period. The effects of wind in combination with snow loading may be ignored.

6 HIGHWAY BRIDGE LIVE LOADS

6.1 **General.** Standard highway loading consists of HA and HB loading.

HA loading is a formula loading representing normal traffic in Great Britain. HB loading is an abnormal vehicle unit loading. Both loadings include impact. (See Appendix A for the basis of HA and HB loading).

6.1.1 **Loads to be considered.** The structure and its elements shall be designed to resist the more severe effects of either:

design HA loading (see 6.4.1) or
design HA loading combined with design HB loading (see 6.4.2)

6.1.2 **Notional lanes, hard shoulders, etc.** The width and a number of notional lanes, and the presence of hard shoulders, hard strips, verges and central reserves are integral to the disposition of HA and HB loading. Requirements for deriving the width and number of notional lanes for design purposes are specified in 3.2.9.3. Requirements for reducing HA loading for certain lane widths and loaded length are specified in 6.4.1.

6.1.3 **Distribution analysis of structure.** The effects of the design standard loadings shall, where appropriate, be distributed in accordance with a rigorous distribution analysis or from data derived from suitable tests. In the latter case the use of such data shall be subject to the approval of the appropriate authority.

6.2 **Type HA loading.** Type HA loading consists of a uniformly distributed load (see 6.2.1) and a knife edge load (see 6.2.2) combined, or of a single wheel load (see 6.2.5).

6.2.1 **Nominal uniformly distributed load (UDL).** For loaded lengths up to and including 50m the UDL, expressed in kN per linear metre of notional lane, shall be derived from the equation,

$$W = 336\left(\frac{1}{L}\right)^{0.67}$$

and for loaded lengths in excess of 50m but less than 1600m the UDL shall be derived from the equation,

$$W = 36\left(\frac{1}{L}\right)^{0.1}$$

where L is the loaded length (in m) and W is the load per metre of notional lane (in kN).

For loaded lengths above 1600m, the UDL shall be agreed with the appropriate authority.

Values of the load per linear metre of notional lane are given in table 13 and the loading curve is illustrated in figure 10.

Table 13. Type HA uniformly distributed load.

Loaded length	Load	Loaded length	Load	Loaded length	Load
m	kN/m	m	kN/m	m	kN/m
2	211.2	55	24.1	370	19.9
4	132.7	60	23.9	410	19.7
6	101.2	65	23.7	450	19.5
8	83.4	70	23.5	490	19.4
10	71.8	75	23.4	530	19.2
12	63.6	80	23.2	570	19.1
14	57.3	85	23.1	620	18.9
16	52.4	90	23.0	670	18.8
18	48.5	100	22.7	730	18.6
20	45.1	110	22.5	790	18.5
23	41.1	120	22.3	850	18.3
26	37.9	130	22.1	910	18.2
29	35.2	150	21.8	980	18.1
32	33.0	170	21.5	1050	18.0
35	31.0	190	21.3	1130	17.8
38	29.4	220	21.0	1210	17.7
41	27.9	250	20.7	1300	17.6
44	26.6	280	20.5	1400	17.4
47	25.5	310	20.3	1500	17.3
50	24.4	340	20.1	1600	17.2

NOTE. Generally, the loaded length for the member under consideration shall be the full base length of the adverse area (see 3.2.5). Where there is more than one adverse area, as for example in continuous construction, the maximum effect should be determined by consideration of the adverse area or combination of adverse areas using the loading appropriate to the full base length or the sum of the full base lengths of any combination of the adverse areas selected. Where the influence line has a cusped profile and lies wholly within a triangle joining the extremities of its base to its maximum ordinate, the base length shall be taken as twice the area under the influence line divided by the maximum ordinate (see figure 11).

6.2.2 **Nominal knife edge load (KEL).** The KEL per notional lane shall be taken as 120 kN.

6.2.3 **Distribution.** The UDL and KEL shall be taken to occupy one notional lane, uniformly distributed over the full width of the lane and applied as specified in 6.4.1.

6.2.4 **Dispersal.** No allowance for the dispersal of the UDL and KEL shall be made.

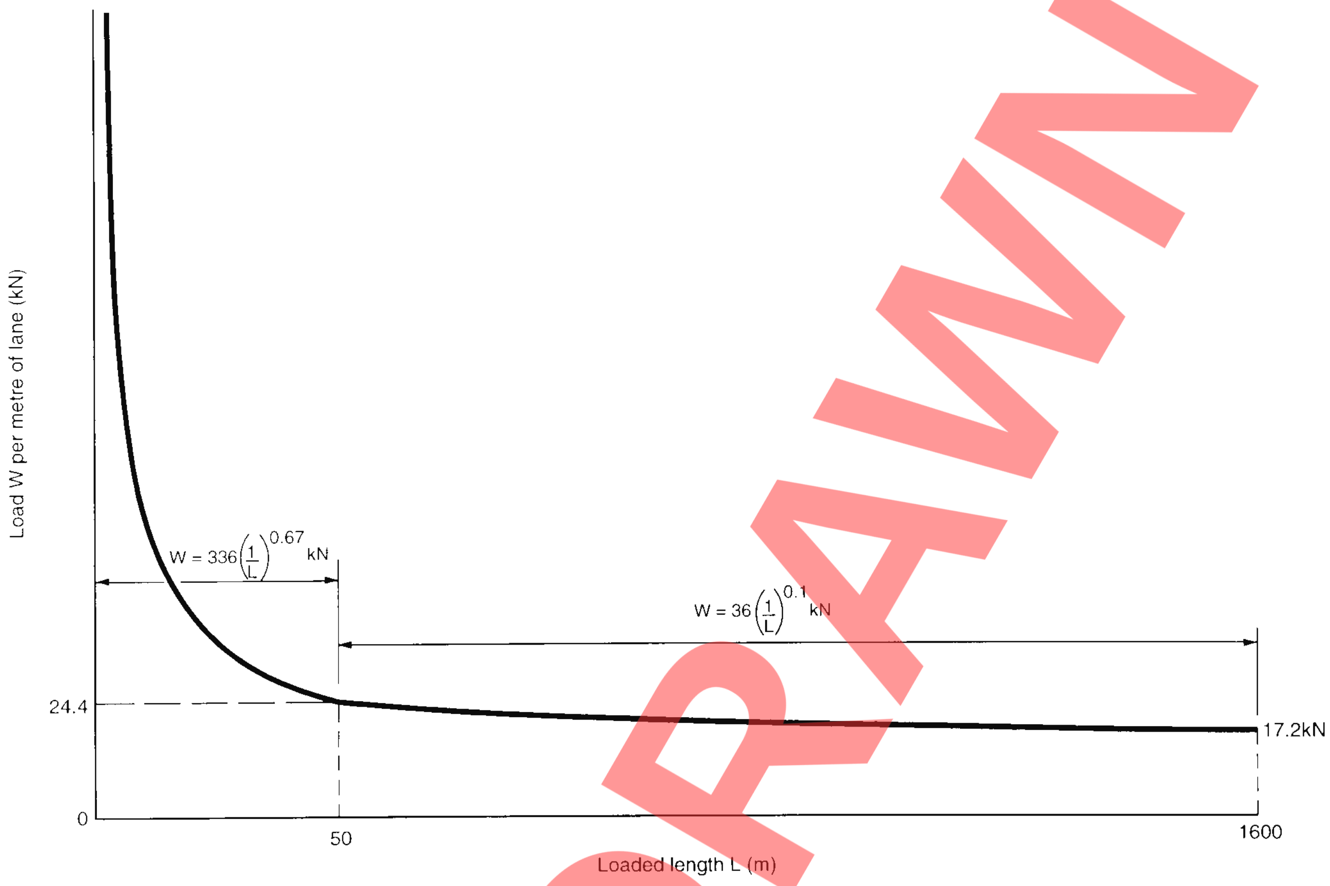
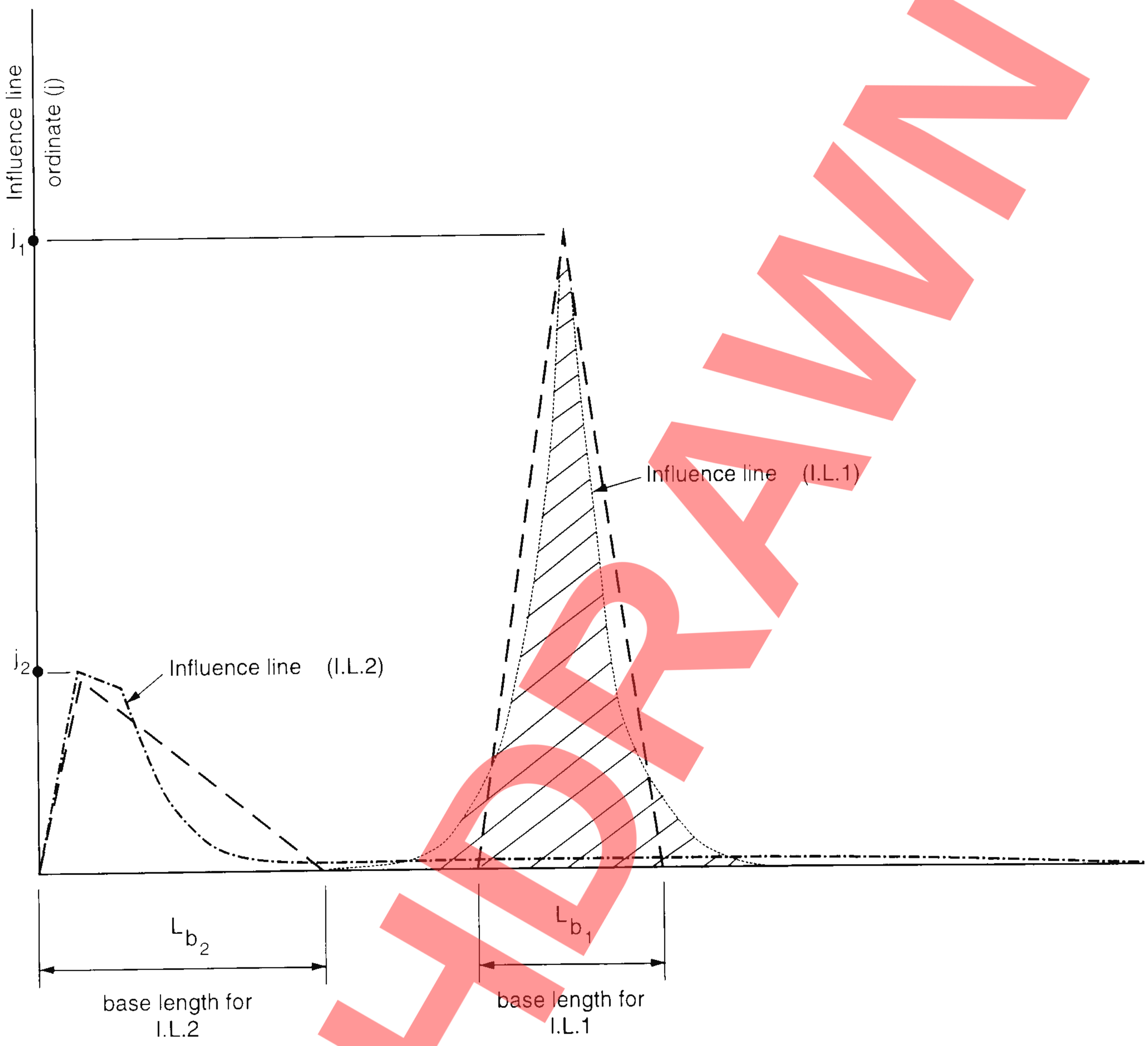


Figure 10. Loading curve HA UDL (Not to scale)



Evaluation of base lengths

Consider influence line 1: Area under I.L.1 = U_1 (shaded);

Value of maximum ordinate = j_1:

Effective base length, $L_{b_1} = \dfrac{2 \times U_1}{j_1}$

A similar evaluation applies for influence line 2

Figure 11. Base lengths for highly cusped influence lines

6.2.5 **Single nominal wheel load alternative to UDL and KEL.** One 100 kN wheel, placed on the carriageway and uniformly distributed over a circular contact area assuming an effective pressure of 1.1 N/mm^2 (ie 340mm diameter), shall be considered.

Alternatively, a square contact area may be assumed, using the same effective pressure (ie 300mm side).

6.2.6 **Dispersal.** Dispersal of the single nominal wheel load at a spread-to-depth ratio of 1 horizontally to 2 vertically through asphalt and similar surfacing may be assumed, where it is considered that this may take place.

Dispersal through structural concrete slabs may be taken at a spread-to-depth ratio of 1 horizontally to 1 vertically down to the neutral axis.

6.2.7 **Design HA loading.** For design HA load considered alone, γ_{fL} shall be taken as follows:

	For the ultimate limit state	For the serviceability limit state
For combinations 1	1.50	1.20
For combinations 2 & 3	1.25	1.00

Where HA loading is coexistent with HB loading (see 6.4.2), γ_{fL}, as specified in 6.3.4, shall be applied to HA loading.

6.3 **Type HB loading.** For all public highway bridges in Great Britain, the minimum number of units of type HB loading that shall normally be considered is 30, but his number may be increased up to 45 if so directed by the appropriate authority.

6.3.1 **Nominal HB loading.** Figure 12 shows the plan and axle arrangement for one unit of nominal HB loading. One unit shall be taken as equal to 10 kN per axle (ie 2.5 kN per wheel).

The overall length of the HB vehicle shall be taken as 10, 15, 20, 25 or 30 m for inner axle spacings of 6, 11, 16, 21 or 26 m respectively, and the effects of the most severe of these cases shall be adopted. The overall width shall be taken as 3.5m. The longitudinal axis of the HB vehicle shall be taken as parallel with the lane markings.

6.3.2 **Contact area.** Nominal HB wheel loads shall be assumed to be uniformly distributed over a circular contact area, assuming an effective pressure of 1.1 N/mm^2.

Alternatively, a square contact area may be assumed, using the same effective pressure.

6.3.3 **Dispersal.** Dispersal of HB wheel loads at a spread-to-depth ratio of 1 horizontally to 2 vertically through asphalt and similar surfacing may be assumed, where it is considered that this may take place.

Dispersal through structural concrete slabs may be taken at a spread-to-depth ratio of 1 horizontally to 1 vertically down to the neutral axis.

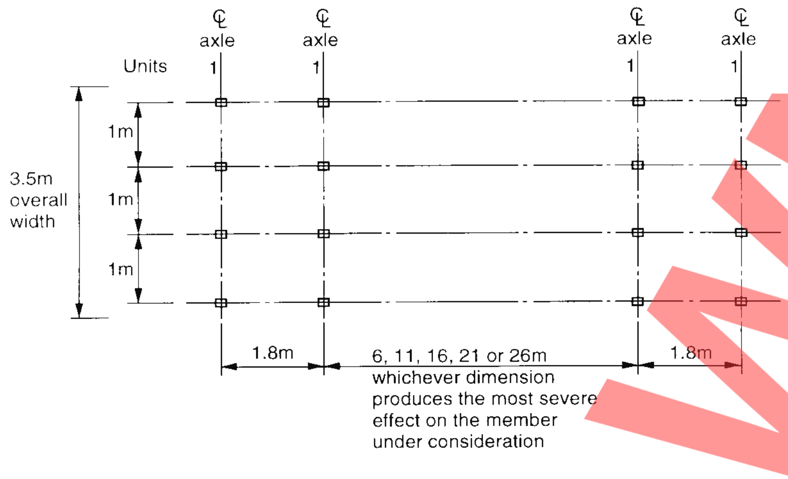


Figure 12. Dimensions of HB vehicle

6.3.4 **Design HB loading.** For design HB load, γ_{fL} shall be taken as follows:

	For the ultimate limit state	For the serviceability limit state
For combination 1	1.30	1.10
For combinations 2 & 3	1.10	1.00

6.4 Application of types HA and HB loading

6.4.1 **Type HA loading.** Type HA UDL determined for the appropriate loaded length (see Note under table 13) and type HA KEL loads shall be applied to each notional lane in the appropriate parts of the influence line for the element or member under consideration*. The lane loadings specified in 6.4.1.1 are interchangeable between the notional lanes and a notional lane or lanes may be left unloaded if this causes the most severe effect on the member or element under consideration. The KEL shall be applied at one point only in the loaded length of each notional lane.

Where the point under consideration has a different influence line for the loading in each lane, the appropriate loaded length for each lane will vary and the lane loadings shall be determined individually.

The lane factors given in 6.4.1.1 shall be applied except where otherwise specified by the appropriate authority.

6.4.1.1 HA Lane Factors. The HA UDL and KEL shall be multiplied by the appropriate factors from table 14 before being applied to the notional lanes indicated.

Where the carriageway has a single notional lane as specified in 3.2.9.3.2, the HA UDL and KEL applied to that lane shall be multiplied by the appropriate first lane factor for a notional lane width of 2.50m. The loading on the remainder of the carriageway width shall be taken as 5kN/m

*In consideration of local (not global) effects, where deviations from planarity may be critical, the application of the knife edge without the UDL immediately adjacent to it may have a more severe effect than with the UDL present.

Table 14. HA lane factors

Loaded Length L (m)	First lane factor β_1	Second lane factor β_2	Third lane factor β_3	Fourth & subsequent lane factor β_n
$0 < L \leq 20$	α_1	α_1	0.6	0.6 α_1
$20 < L \leq 40$	α_2	α_2	0.6	0.6 α_2
$40 < L \leq 50$	1.0	1.0	0.6	0.6
$50 < L \leq 112$ $N < 6$	1.0	$\frac{7.1}{\sqrt{L}}$	0.6	0.6
$50 < L \leq 112$ $N \geq 6$	1.0	1.0	0.6	0.6
$L > 112$ $N < 6$	1.0	0.67	0.6	0.6
$L > 112$ $N \geq 6$	1.0	1.0	0.6	0.6

NOTE 1. $\alpha_1 = 0.274\, b_L$ and cannot exceed 1.0
$\alpha_2 = 0.0137\, [b_L\, (40-L) + 3.65\, (L-20)]$

where b_L is the notional lane width (m)

NOTE 2. N shall be used to determine which set of HA lane factors is to be applied for loaded lengths in excess of 50m. The value of N is to be taken as the total number of notional lanes on the bridge (this shall include all the lanes for dual carriageway roads) except that for a bridge carrying one-way traffic only, the value of N shall be taken as twice the number of notional lanes on the bridge.

6.4.1.2 **Multilevel structures.** Where multilevel superstructures are carried on common substructure members (as, eg columns of a multilevel interchange) the most severe effect at the point under consideration shall be determined from type HA loading applied in accordance with 6.4.1. The number of notional lanes to be considered shall be the total number of lanes, irrespective of their level, which contribute to the load effect at that point.

6.4.1.3 **Transverse cantilever slabs, slabs supported on all four sides and slabs spanning transversely.** HA UDL and KEL shall be replaced by the arrangement of HB loading given in 6.4.3.1.

NOTE: Slabs shall be deemed to cover plates.

6.4.1.4 **Combined effects.** Where elements of a structure can sustain the effects of live load in two ways, ie as elements in themselves and also as parts of the main structure (eg the top flange of a box girder functioning as a deck plate), the element shall be proportioned to resist the combined effects of the appropriate loading specified in 6.4.2.

6.4.1.5 **Knife edge load (KEL).** The KEL shall be taken as acting as follows:

(a) On plates, right slabs and skew slabs spanning or cantilevering longitudinally: in a direction which has the most severe effect. The KEL for each lane shall be considered as acting in a single line in that lane and having the same length as the width of the notional lane and the intensity set out in 6.4.1. As specified in 6.4.1, the KEL shall be applied at one point only in the loaded length.

(b) On longitudinal members and stringers: in a direction parallel to the supports.

(c) On piers, abutments and other members supporting the superstructure: on the deck, parallel to the line of the bearings.

(d) On cross members, including transverse cantilever brackets: in a direction in line with the span of the member.

6.4.1.6 **Single wheel load.** The HA wheel load is applied to members supporting small areas of roadway where the proportion of UDL and KEL that would otherwise be allocated to it is small.

6.4.2 **Types HA and HB loading combined.** Types HA and HB loading shall be combined and applied as follows:

(a) Type HA loading shall be applied to the notional lanes of the carriageway in accordance with 6.4.1, modified as given in (b) below.

(b) Type HB loading shall occupy any transverse position on the carriageway, either wholly within one notional lane or straddling two or more notional lanes.

Where the HB vehicle lies wholly within the notional lane (eg figure 13 (1)) or where the HB vehicle lies partially within a notional lane and the remaining width of the lane, measured from the side of the HB vehicle to the edge of the notional lane, is less than 2.5 metres (eg figure 13 (2)(a)), type HB loading is assumed to displace part of the HA loading in the lane or straddled lanes it occupies. No other live loading shall be considered for 25 metres in front of the leading axle to 25 metres behind the rear axle of the HB vehicle.

The remainder of the loaded length of the lane or lanes thus occupied by the HB vehicle shall be loaded with HA UDL only; HA KEL shall be omitted. The intensity of the HA UDL in these lanes shall be appropriate to the loaded length that includes the total length displaced by the type HB loading with the front and rear 25 metre clear spaces.

Where the HB vehicle lies partially within a notional lane and the remaining width of the lane, measured from the side of the HB vehicle to the far edge of the notional lane, is greater or equal to 2.5 metres (eg figure 13(2)(b)), the HA UDL loading in that lane shall remain but shall be multiplied by an appropriate lane factor for a notional lane width of 2.5 metres irrespective of the actual lane width; the HA KEL shall be omitted.

Only one HB vehicle shall be considered on any one superstructure or on any substructure supporting two or more superstructures.

Figure 13 illustrates typical configurations of type HA loading in combination with type HB loading.

6.4.3 **Highway loading on transverse cantilever slabs, slabs supported on all four sides, slabs spanning transversely and central reserves.** Type HA loading shall be applied to the elements specified in 6.4.3.1 and 6.4.3.2.

6.4.3.1 **Transverse cantilever slabs, slabs supported on all four sides and slabs spanning transversely.** These elements shall be so proportioned as to resist the effects of the appropriate number of units of type HB loading occupying any transverse position in the carriageway or placed in one notional lane in combination with 30 units of type HB loading placed in one other notional lane. Proper consideration shall be given to transverse joints of transverse cantilever slabs and to the edges of these slabs because of the limitations of distribution*.

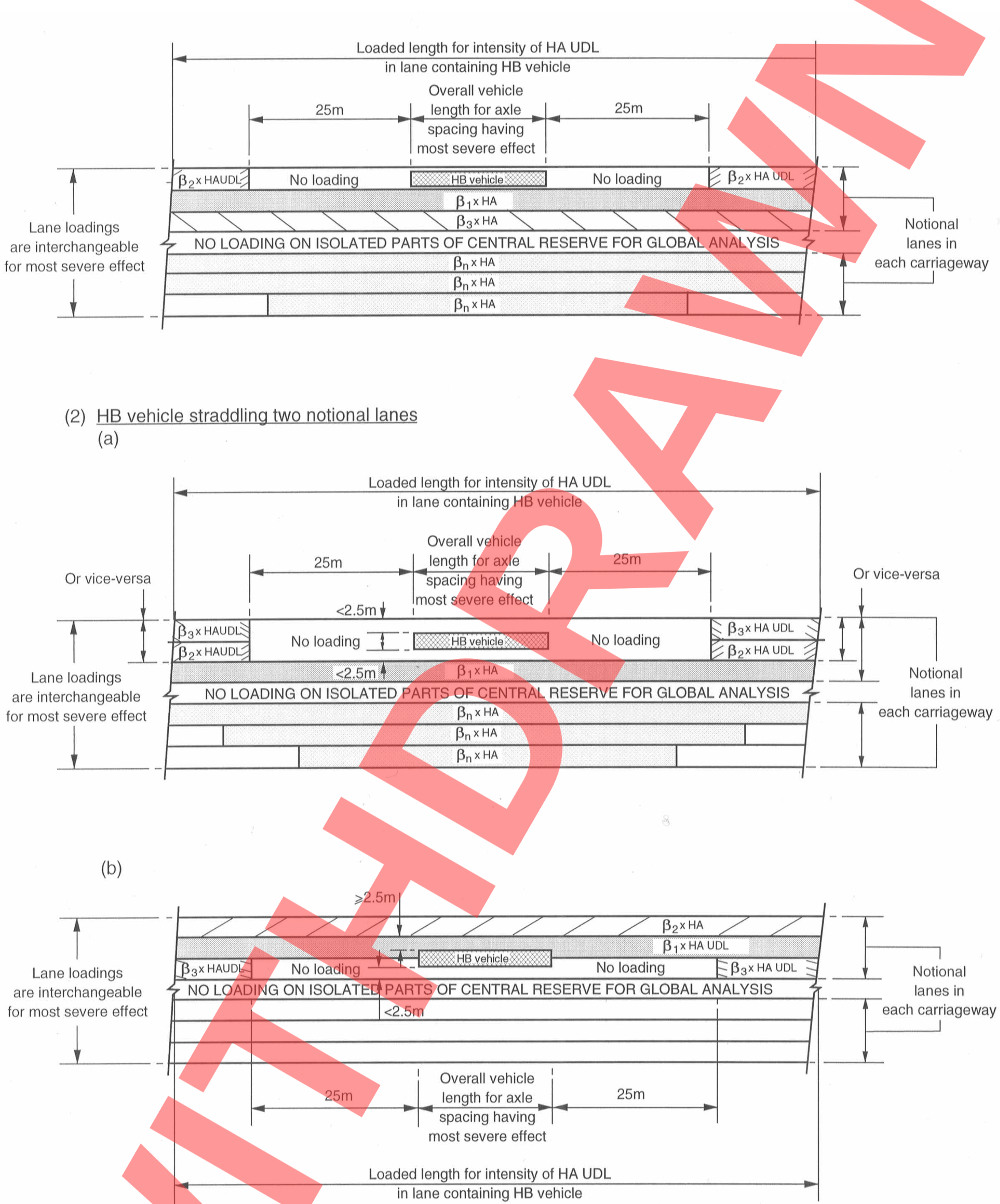
This does not apply to members supporting these elements.

6.4.3.2 **Central reserves.** On dual carriageways the portion of the central reserve isolated from the rest of the carriageway either by a raised kerb or by safety fences is not required to be loaded with live load in considering the overall design of the structure, but it shall be capable of supporting 30 units of HB loading.

6.5 **Standard footway and cycle track loading.** The live load on highway bridges due to pedestrian traffic shall be treated as uniformly distributed over footways and cycle tracks. For elements supporting footways or cycle tracks, the intensity of pedestrian live load shall vary according to loaded length and any expectation of exceptional crowds. Reductions in pedestrian live load intensity may be made for elements supporting highway traffic lanes as well as footways or cycle tracks. Reductions may also be made where the footway (or footway and cycle track together) has a width exceeding two metres.

* This is the only exception to the rule that not more than one HB vehicle shall be considered to act on a structure. The 30 unit vehicle is to be regarded as a substitute for HA loading for these elements only.

(1) HB vehicle within one notional lane



NOTE:

1. See 6.4.1.1 for the value of the HA lane factor (β) to be taken for each lane.
2. The overall length and width of the HB vehicle shall be as specified in 6.3.1.
3. Unless otherwise stated, type HA loading includes both uniformly distributed loading (UDL) and knife edge loading (KEL).
4. See 6.4.1 for loaded length to be taken in each lane.

Figure 13. Type HA and HB highway loading in combination.

6.5.1 **Nominal pedestrian live load**

6.5.1.1 **Elements supporting footways or cycle tracks only.** The nominal pedestrian live load on elements supporting footways and cycle tracks only shall be as follows:

(a) for loaded lengths of 36 m and under, a uniformly distributed live load of 5.0 kN/m^2.

(b) for loaded lengths in excess of 36m, k x 5.0 kN/m^2 where k is the

$$\frac{\text{nominal HA UDL for appropriate loaded length (in kN/m)} \times 10}{L+270}$$

where L is the loaded length (in m).

Where the footway (or footway and cycle track together) has a width exceeding 2m these intensities may be further reduced by 15% on the first metre in excess of 2m and by 30% on the second metre in excess of 2m. No further reduction for widths exceeding 4m shall be made. These intensities may be averaged and applied as a uniform intensity over the full width of the footway or cycle track.

Special consideration shall be given to the intensity of the pedestrian live load to be adopted on loaded lengths in excess of 36m where exceptional crowds may be expected. Such loading shall be agreed with the appropriate authority.

6.5.1.2 **Elements supporting footways or cycle tracks and a carriageway.** The nominal pedestrian live load on elements supporting carriageway loading as well as footway or cycle track loading shall be taken as 0.8 of the value specified in 6.5.1.1 (a) or (b) as appropriate, except for loaded lengths in excess of 400m or where crowd loading is expected.

Where the footway (or footway and cycle track together) has a width exceeding 2m these intensities may be further reduced by 15% on the first metre in excess of 2m and by 30% on the second metre in excess of 2m. No further reduction for widths exceeding 4m shall be made. These intensities may be averaged and applied as a uniform intensity over the full width of footway or cycle track.

Where a main structural member supports two or more notional traffic lanes, the footways/cycle track loading to be carried by the main member may be reduced to the following:

On footways: 0.5 of the value given in 6.5.1.1 (a) and (b) as appropriate.

On cycle tracks: 0.2 of the value given in 6.5.1.1 (a) and (b) as appropriate.

Where a highway bridge has two footways and a load combination is considered such that only one footway is loaded, the reductions in the intensity of footway loading specified in this clause shall not be applied.

Where crowd loading is expected or where loaded lengths are in excess of 400m, special consideration shall be given to the intensity of pedestrian live loading to be adopted. This shall be agreed with the appropriate authority.

Special consideration shall also be given to structures where there is a possibility of crowds using cycle tracks which could coincide with exceptionally heavy highway carriageway loading.

6.5.2 **Live load combination.** The nominal pedestrian live load specified in 6.5.1.2 shall be considered in combination with the normal primary live load on the carriageway derived and applied in accordance with 6.4.

6.5.3 **Design load.** For the pedestrian live load on footways and cycle tracks γ_{fL} shall be taken as follows:

	For the ultimate limit state	For the serviceability limit state
For combination 1	1.50	1.00
For combinations 2 & 3	1.25	1.00

For primary live load on the carriageway, γ_{fL} shall be taken as specified in 6.2.7 and 6.3.4.

6.6 **Accidental wheel loading.** The elements of the structure supporting outer verges, footways or cycle tracks which are not protected from vehicular traffic by an effective barrier, shall be designed to sustain local effects of the nominal accidental wheel loading.

6.6.1 **Nominal accidental wheel loading.** The accidental wheel loading having the plan, axle and wheel load arrangement shown in figure 14 shall be selected and located in the position which produces the most adverse effect on the elements. Where the application of any wheel or wheels has a relieving effect, it or they shall be ignored.

6.6.2 **Contact area.** Nominal accidental wheel loads shall be assumed to be uniformly distributed over a circular contact area, assuming an effective pressure of 1.1 N/mm^2. Alternatively, a square contact area may be assumed, using the same effective pressure.

6.6.3 **Dispersal.** Dispersal of accidental wheel loads at a spread-to-depth ratio of 1 horizontally to 2 vertically through asphalt and similar surfacing may be assumed, where it is considered that this may take place.

Dispersal through structural concrete slabs may be taken at a spread-to-depth ratio of 1 horizontally to 1 vertically down to the neutral axis.

6.6.4 **Live load combination.** Accidental wheel loading need not be considered in combinations 2 and 3. No other primary live load is required to be considered on the bridge.

6.6.5 **Design load.** For accidental wheel loading γ_{fL} shall be taken as follows:

	For the ultimate limit state	For the serviceability limit state
For combination 1	1.50	1.20

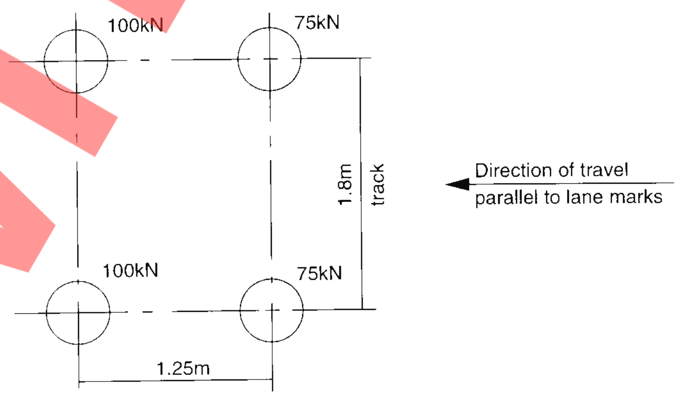


Figure 14. Accidental wheel loading

6.7 **Loads due to vehicle collision with parapets.*** The local effects of vehicle collision with parapets shall be considered in the design of elements of the structure supporting parapets by application of the loads given in 6.7.1. In addition, the global effects of vehicle collision with high level of containment parapets shall be considered in the design of the bridge superstructures, bearings, substructures and retaining walls and wing walls by application of loads given in 6.7.2. The global effects of vehicle collision with other types of parapets need not be considered.

6.7.1 **Loads due to vehicle collision with parapets for determining local effects.**

6.7.1.1 **Nominal loads.** In the design of the elements of the structure supporting parapets, the following loads shall be regarded as the nominal load effects to be applied to these elements according to the parapet type and construction.

For concrete parapets (high and normal levels of containment):

The calculated ultimate design moment of resistance and the calculated ultimate design shear resistance of a 4.5m length of parapet at the parapet base applied uniformly over any 4.5m length of supporting element.

For metal parapets (high, normal and low levels of containment):

(a) The calculated ultimate design moment of resistance of a parapet post applied at each base of up to three adjacent posts and

(b) the lesser of the following:

(i) the calculated ultimate design moment of resistance of a parapet post divided by the height of the centroid of the lowest effective longitudinal member above the base of the parapet applied at each base of up to any three adjacent parapet posts;

(ii) the calculated ultimate design shear resistance of a parapet post applied at each base of up to any three adjacent parapet posts.

In the case of all high level of containment parapets, an additional single vertical load of 175 kN shall be applied uniformly over length of 3m. at the top of the front face of the parapet. The loaded length shall be in that position which will produce the most severe effect on the member under consideration.

6.7.1.2 **Associated nominal primary live load.** The accidental wheel loading specified in 6.6 shall be considered to act with the loads due to vehicle collision with parapets.

6.7.1.3 **Load combination.** Loads due to vehicle collision with parapets for determining local effects shall be considered in combination 4 only, and need not be taken as coexistent with other secondary live loads.

* This subclause refers to the load effects resulting from a collision with a parapet, locally on the structural elements in the vicinity of the parapet supports and globally on bridge superstructures, bearings, and substructures and retaining walls and wing walls. Rules for the design of highway parapets in the United Kingdom including requirements for high level of containment parapets are set out in the appropriate Standard given in the Design Manual for Roads and Bridges (DMRB).

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6.7.1.4 **Design load.** For determining local effects on elements supporting the parapet, γ_{fl} factors to be applied to the nominal load due to vehicle collision with the parapet and the associated nominal primary live load shall be taken as follows:

	For the ultimate limit state		For the serviceability limit state	
	Low and normal levels of containment	High level of containment	Low and normal levels of containment	High level of containment
For load due to vehicle collision with parapet.	1.50	1.40	1.20	1.15
For associated primary live load	1.30	1.30	1.10	1.10

6.7.2 **Loads due to vehicle collision with high level of containment parapets for determining global effects.**

6.7.2.1 **Nominal loads.** In the design of bridge superstructures, bearings, substructures, retaining walls and wing walls, the following nominal impact loads shall be applied at the top of the traffic face of high level of containment parapets only.

(a) a single horizontal transverse load of 500 kN;

(b) a single horizontal longitudinal load of 100 kN;

(c) a single vertical load of 175 kN.

The loads shall be applied uniformly over a length of 3 m measured along the line of the parapet. The loaded length shall be in that position which will produce the most severe effect on the part of the structure under consideration.

6.7.2.2 **Associated nominal primary live load.** Type HA and the accidental wheel loading, shall be considered to act with the load due to vehicle collision on high level of containment parapets. The type HA and the accidental wheel loading shall be applied in accordance with 6.4 and 6.6.1, respectively and such that they will have the most severe effect on the member under consideration. They may be applied either separately or in combination.

6.7.2.3 **Load combination.** Loads due to vehicle collision with high level of containment parapets for determining global effects shall be considered in combination 4 only, and need not be taken as coexistent with other secondary live loads.

6.7.2.4 **Design load.** The load due to vehicle collision with high level of containment parapets for determining global effects on bridge superstructures, substructures, non elastomeric bearings, retaining walls and wing walls need only be considered at the ultimate limit state. In the case of elastomeric bearings however, the load due to vehicle collision with high level of containment parapet for determining global effects should only be considered at the serviceability limit state. The γ_{fL} values to be applied to the nominal load due to vehicle collision with high level of containment parapets and the associated nominal primary live load shall be taken as follows:

	Massive structures	Light* structures
For loads due to vehicle collision with parapets:	For the ultimate limit state	
On bridge superstructure and non-elastomeric bearings	1.25	1.4*
On bridge superstructures and wing and retaining walls	1.0	1.4*
For associated primary live loads	1.25	1.25

	Massive structures	Light structures
For loads due to vehicle collision with parapets:	For the serviceability limit state	
On elastomeric bearings	1.0	1.0
For associated primary live loads	1.00	1.00

6.8 **Vehicle collision loads on highway bridge supports and superstructures.** The design of highway bridges for vehicle collision loads shall be in accordance with the appropriate Standard given in the DMRB. The following Clauses 6.8.1 to 6.8.6 may still be applicable in certain circumstances where agreed with the appropriate authority, see also 7.2.

6.8.1 **Nominal loads on supports.** The nominal loads are given in table 15 together with their direction and height of application, and shall be considered as acting horizontally on bridge supports. All of the loads given in table 15 shall be applied concurrently. The loads shall be considered to be transmitted from the safety fence provided at the supports with residual loads acting above the safety fence.

Table 15. Collision loads on supports of bridges over highways

	Load normal to the carriageway below	Load parallel to the carriageway below	Point of application on bridge support
Load transmitted from safety fence	kN 150	kN 50	Any one bracket attachment point or for free-standing fences, any one point 0.75m above carriageway level
Residual load above safety fence	100	100	At the most severe point between 1m and 3m above carriageway level

*NOTE: The γ_{fL} value of 1.4 shall only be used for small and light structures (such as some wing walls cantilevered off abutments, low light retaining walls, very short span bridge decks) where the attenuation of the collision loads is unlikely to occur. For other structures, account may be taken of the dynamic nature of the force and its interaction with the mass of the structure by application of the reduced γ_{fL} values given above.

6.8.2 **Nominal load on superstructures.** A single nominal load of 50 kN shall be considered to act as a point load on the bridge superstructure in any direction between the horizonal and the vertical. The load shall be applied to the bridge soffit, thus precluding a downward vertical application. Given that the plane of the soffit may follow a superelevated or non-planar form, the load can have an outward or inward application.

6.8.3 **Associated nominal primary live load.** No primary live load is required to be considered on the bridge.

6.8.4 **Load combination.** Vehicle collision loads on supports and on superstructures shall be considered separately, in combination 4 only, and need not be taken as coexistent with other secondary live loads.

6.8.5 **Design load.** For all elements excepting elastomeric bearings, the effects due to vehicle collision loads on supports and on superstructures need only be considered at the ultimate limit state. The γ_{fL} to be applied to the nominal loads shall have a value of 1.50.

For elastomeric bearings, the effects due to vehicle collision loads on supports and on superstructures should be only considered at the serviceability limit state. The γ_{fL} to be applied to the nominal loads shall have a value of 1.0.

6.8.6 **Bridges crossing railway track, canals or navigable water.** Collision loading on bridges over railways, canals or navigable water shall be as agreed with the appropriate authority.

6.9 **Centrifugal loads.** On highway bridges carrying carriageways with horizontal radius' of curvature less than 1000m, centrifugal loads shall be applied in any two notional lanes in each carriageway at 50m centres. If the carriageway consists of one notional lane only, centrifugal loads shall be applied at 50m centres in that lane.

6.9.1 **Nominal centrifugal load.** A nominal centrifugal load F_c shall be taken as:

$$F_c = \frac{40000}{r + 150} \text{ kN}$$

where r is the radius of curvature of the lane (in m). A nominal centrifugal load shall be considered to act as a point load, acting in a radial direction at the surface of the carriageway and parallel to it.

6.9.2 **Associated nominal primary live load.** With each centrifugal load there shall also be considered a vertical live load of 400 kN, distributed over the notional lane for a length of 6m.

6.9.3 **Load combination.** Centrifugal loads shall be considered in combination 4 only and need not be taken as coexistent with other secondary live loads.

6.9.4 **Design load.** For the centrifugal loads and primary live loads, γ_{fL} shall be taken as follows:

For the ultimate limit state	For the serviceability limit state
1.50	1.00

6.10 **Longitudinal load.** The longitudinal load resulting from traction or braking of vehicles shall be taken as the more severe design load resulting from 6.10.1, 6.10.2 and 6.10.5, applied at the road surface and parallel to it in one notional lane only.

6.10.1 **Nominal load for type HA.** The nominal load for HA shall be 8kN/m of loaded length plus 250 kN, subject to a maximum of 750 kN, applied to an area one notional lane wide x the loaded length.

6.10.2 **Nominal load for type HB.** The nominal load for HB shall be 25% of the total nominal HB load adopted, applied as equally distributed between the eight wheels of 2 axles of the vehicle, 1.8m apart (see 6.3).

6.10.3 **Associated nominal primary live load.** Type HA or HB load, applied in accordance with 6.4, shall be considered to act with longitudinal load as appropriate.

6.10.4 **Load combination**. Longitudinal load shall be considered in combination 4 only and need not be taken as coexistent with other secondary live loads.

6.10.5 **Design load.** For the longitudinal and primary live load γ_{fL} shall be taken as follows:

	For the ultimate limit state	For the serviceability limit state
For HA load	1.25	1.00
For HB load	1.10	1.00

6.11 **Accidental load due to skidding.** On straight and curved bridges a single point load shall be considered in one notional lane only, acting in any direction on and parallel to, the surface of the highway.

6.11.1 **Nominal load.** The nominal load shall be taken as 300 kN.

6.11.2 **Associated nominal primary live load.** Type HA loading, applied in accordance with 6.4.1, shall be considered to act with the accidental skidding load.

6.11.3 **Load combination.** Accidental load due to skidding shall be considered in combination 4 only, and need not be taken as coexistent with other secondary live loads.

6.11.4 **Design load.** For the skidding and primary live load γ_{fL} shall be taken as follows:

For the ultimate limit state	For the serviceability limit state
1.25	1.00

6.12 **Loading for fatigue investigations.** For loading for fatigue investigations, see Part 10 of this standard.

6.13 **Dynamic loading on highway bridges.** The effects of vibration due to live load are not normally required to be considered. However, special consideration shall be given to dynamically sensitive structures.

7. FOOT/CYCLE TRACK BRIDGE LIVE LOADS

7.1 **Standard foot/cycle track bridge loading.** The live load due to pedestrian traffic on bridges supporting footways and cycle tracks only shall be treated as uniformly distributed. The intensity of pedestrian live load shall vary according to loaded length and any expectation of exceptional crowds.

7.1.1 **Nominal pedestrian live load.** The nominal pedestrian live load on foot/cycle track bridges shall be as follows:

(a) for loaded lengths of 36m and under, a uniformly distributed live load of 5.0 kN/m^2;

(b) for loaded lengths in excess of 36m, k x 5.0 kN/m^2 where k is the

$$\frac{\text{nominal HA UDL for appropriate loaded length (in kN/m)} \times 10}{L+270}$$

where L is the loaded length (in m).

Special consideration shall be given to the intensity of the live load to be adopted on loaded lengths in excess of 36m where exceptional crowds may be expected (as for example, where a footbridge services a sports stadium). Consideration shall also be given to horizontal dynamic loading due to crowds when the fundamental horizontal natural frequency of the loaded bridge is less than 1.5 Hz (see Appendix B). Such loading shall be agreed with the appropriate authority.

7.1.2 **Effects due to horizontal loading on pedestrian parapets.** In the design of the elements of the structure supporting pedestrian parapets*, the nominal load shall be taken as 1.4 kN/m length applied at the top of the parapet and acting horizontally. This loading shall be considered to act with the nominal pedestrian live load given in 7.1.1.

7.1.3 **Design load.** For the live load on foot/cycle track bridges and for the load on pedestrian parapets, γ_{fL} shall be taken as follows:

	For the ultimate limit state	For the serviceability limit state
For combination 1	1.50	1.00
For combinations 2 & 3	1.25	1.00

7.2 **Vehicle collision loads on foot/cycle track bridge supports and superstructures.** The design of foot/cycle track bridge supports for vehicle collision loads shall be in accordance with the appropriate Standard given in the DMRB. Clauses 6.8.1 and 6.8.3 to 6.8.6 may still be applicable in certain circumstances where agreed with the appropriate authority. For foot/cycle track bridges with minimum new headroom clearance of 5.7m, vehicle collision loads on superstructures need not be applied. For foot/cycle track bridges with minimum new headroom clearance of less than 5.7m the vehicle collision loads on superstructures given in Clause 6.8.2 shall not be applied and the impact requirements shall be obtained from the appropriate authority.

7.3 **Vibration serviceability.** Consideration shall be given to vibration that can be induced in foot/cycle track bridges by resonance with the movement of users and by deliberately induced vibration. The structure shall be deemed to be satisfactory where its response as calculated in appendix B complies with the limitations specified therein.

* Rules for the design of pedestrian parapets in the United Kingdom are set out in the appropriate Standard given in the DMRB.

8. RAILWAY BRIDGE LIVE LOAD

8.1 **General.** Standard railway loading consists of two types, RU and RL.

RU loading allows for all combinations of vehicles currently running or projected to run on railways in the Continent of Europe, including the United Kingdom, and is to be adopted for the design of bridges carrying main line railways of 1.4m gauge and above.

RL loading is reduced loading for use only on passenger rapid transit railway systems on lines where main line locomotives and rolling stock do not operate.

The derivation of standard railway loadings is given in appendix D.

Nominal primary and associated secondary live loads are as given in 8.2.

8.2 **Nominal loads**

8.2.1 **Load Models**

8.2.1.1 **Type RU loading**. Nominal type RU loading consists of four 250 kN concentrated loads preceded, and followed, by a uniformly distributed load of 80 kN/m. The arrangement of this loading is as shown in figure 15.

8.2.1.2 **Type SW/0 loading**. Nominal type SW/0 loading consists of two uniformly distributed loads of 133 kN/m, each 15m long and separated by a distance of 5.3m. The arrangement of this loading is as shown in figure 15.

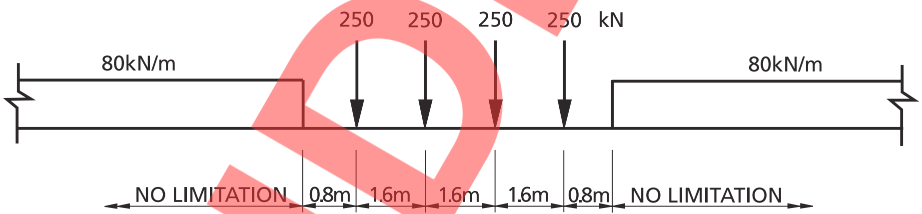


Figure 15(a) Type RU loading

Note: See 8.2.3 for additions to this loading for dynamic effects

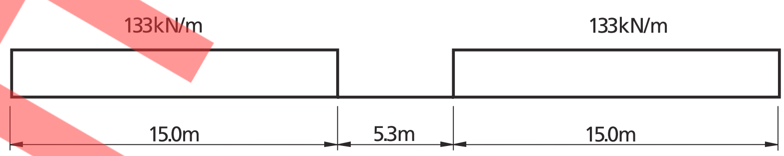
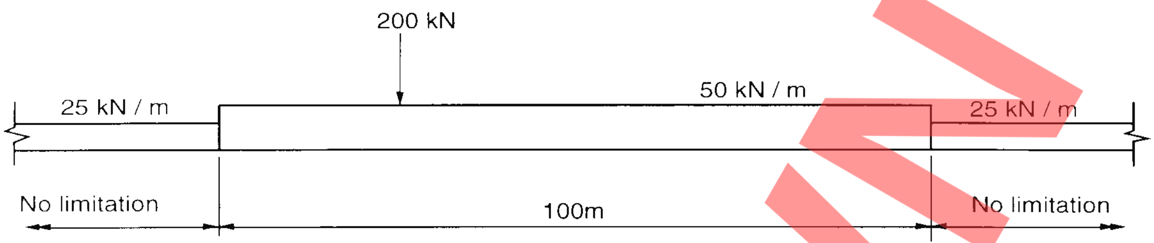


Figure 15(b) Type SW/0 loading

Note: See 8.2.3 for additions to this loading for dynamic effects

Figure 15. Type RU loading and Type SW/0 loading



NOTE: See 8.2.3 for additions to this loading for dynamic effects.

Figure 16. Type RL loading

8.2.2 **Type RL loading.** Nominal type RL loading consists of a single 200 kN concentrated load coupled with a uniformly distributed load of 50 kN/m for loaded lengths up to 100m. For loaded lengths in excess of 100m the distributed nominal load shall be 50 kN/m for the first 100m and shall be reduced to 25 kN/m for lengths in excess of 100m, as shown in figure 16.

Alternatively, two concentrated nominal loads, one of 300 kN and the other of 150 kN, spaced at 2.4m intervals along the track, shall be used on deck elements where this gives a more severe condition. These two concentrated loads shall be deemed to include dynamic effects.

8.2.3 **Dynamic effects.** The standard railway loadings specified in 8.2.1 and 8.2.2 (except the 300 kN and 150 kN concentrated alternative RL loading) are equivalent static loadings and shall be multiplied by appropriate dynamic factors to allow for impact, oscillation and other dynamic effects including those caused by track and wheel irregularities. The dynamic factors given in 8.2.3.1 and 8.2.3.2 shall be adopted, provided that maintenance of track and rolling stock is kept to a reasonable standard.

8.2.3.1 **Type RU and SW/0 loading**. Except as otherwise specified, the dynamic factor for RU and SW/0 loading applies to all types of track and shall be as given in table 16.

Table 16. Dynamic factors for type RU loading

Dimension L (m)	Dynamic factor for evaluating	
	bending moment	shear
up to 3.6	2.00	1.67
from 3.6 to 67	$0.73 + \dfrac{2.16}{(L)^{0.5} - 0.2}$	$0.82 + \dfrac{1.44}{(L)^{0.5} - 0.2}$
over 67	1.00	1.00

In deriving the dynamic factor, L is taken as the length (in m) of the influence line for deflection of the element under consideration. For unsymmetrical influence lines, L is twice the distance between the point at which the greatest ordinate occurs and the nearest end point of the influence line. In the case of floor members, 3m should be added to the length of the influence line as an allowance for load distribution through track.

The values given in table 17 may be used, where appropriate.

These dynamic factors are applicable to full RU and SW/0 loading, where the deflection of the bridge is within limits set out in UIC Leaflet 776-3R* (in Figure 1 of which the expression for δ_u for spans between 20 and 100 metres shall be corrected to read $\delta_u = 0.56L^{1.184}$.

For speeds greater than 200 km/h a special dynamic study shall be carried out to determine the dynamic effects in accordance with the requirements of the relevant authority.

8.2.3.2 **Type RL loading.** The dynamic factor for RL loading, when evaluating moments and shears, shall be taken as 1.20, except for unballasted tracks where, for rail bearers and single-track cross girders, the dynamic factor shall be increased to 1.40.

The dynamic factor applied to temporary works may be reduced to unity when rail traffic speeds are limited to not more than 25 km/h.

8.2.4 **Dispersal of concentrated loads.** Concentrated loads applied to the rail will be distributed both longitudinally by the continuous rail to more than one sleeper, and transversely over a certain area of deck by the sleeper and ballast.

It may be assumed that only two-thirds of a concentrated load applied to one sleeper will be transmitted to the bridge deck by that sleeper, and that the remaining one-third will be transmitted equally to the adjacent sleeper on either side.

Where the ballast depth is at least 200mm, it may be assumed that only half of a concentrated load applied to one sleeper will be transmitted to the bridge deck by that sleeper, and that the remaining half will be transmitted equally to the adjacent sleeper on either side.

Table 17. Dimension *L* used in calculating the dynamic factor for RU loading.

	Dimension *L*
Main girders:	
simply supported	span
continuous	For 2, 3, 4, 5 and more spans 1.2, 1.3, 1.4, 1.5 x mean span, but at least the greatest span
portal frames and arches	½ span
Floor members:	
simply supported rail bearers	cross girder spacing plus 3m
cross girders loaded by simply supported rail bearers	Twice the spacing of cross girders plus 3m
end cross girders or trimmers	4m
cross girders loaded by continuous deck elements and any elements in a continuous deck system	The lesser of the span of the main girders and twice the main girder spacing

*Leaflet 776-3R (1989 Edition) published by UIC, 16 rue Jean Rey, F, 75015, Paris, France.

The load acting on the sleeper under each rail may be assumed to be distributed uniformly over the ballast at the level on the underside of the sleeper for a distance of 800 mm symmetrically about the centre line of the rail or to twice the distance from the centre line of the rail to the nearer end of the sleeper, whichever is the lesser. Dispersal of this load through the ballast onto the supporting structure shall be taken at 5° to the vertical.

The distribution of concentrated loads applied to a track not supported on ballast shall be calculated on the basis of the relative stiffnesses of the rail, its support on the bridge deck and the bridge deck itself.

In designing the supporting structure for the loads transmitted from the sleepers, distributed as set out above, any further distribution arising from the type of construction of the deck may be taken into account.

8.2.5 **Deck plates and similar local elements.** Irrespective of the calculated distribution of axle loads, all deck plates and similar local elements shall be designed to support a nominal load of 250 kN for RU loading and 168 kN for RL loading at any point of support of a rail. These loads shall be deemed to include all allowances for dynamic effects and lurching.

8.2.6 **Application of standard loadings.**

8.2.6.1 **Type RL Loading.** RL loading shall be applied to each and every track as specified in 4.4. Any number of lengths of the distributed load may be applied, but the total length of 50 kN/m intensity shall not exceed 100m on any track. The concentrated loads shall only be applied once per track for any point under consideration.

8.2.6.2 **Type RU Loading.** For bridges carrying one or two tracks, RU loading shall be applied to each track as specified in 4.4. For bridges carrying more than two tracks, RU loading shall be applied as specified by the relevant authority.

8.2.6.3 **Type SW/0 loading.** For continuous bridges, SW/0 loading shall be applied as specified in 4.4 as an additional and separate load case to the requirements of 8.2.6.2. SW/0 loading shall be applied in accordance with the requirements of the relevant authority.

There is no requirement for SW/0 loading to be considered in any fatigue check.

There is no requirement for SW/0 loading to be repeated along the length of a track. SW/0 loading shall be applied without curtailment, even though part(s) of the load model may have relieving effects.

8.2.7 **Lurching.** Lurching results from the temporary transfer of part of the live loading from one rail to another, the total track load remaining unaltered.

The dynamic factor applied to RU loading will take into account the effects of lurching, and the load to be considered acting on each rail shall be half the track load.

The dynamic factor applied to RL loading will not adequately take account of all lurching effects. To allow for this, 0.56 of the track load shall be considered acting on one rail concurrently with 0.44 of the track load on the other rail. This redistribution of load need only be taken into account on one track where members support two tracks. Lurching may be ignored in the case of elements that support load from more than two tracks.

8.2.8 **Nosing.** An allowance shall be made for lateral loads applied by trains to the track. This shall be taken as a single nominal load of 100 kN, acting horizontally in either direction at right angles to the track at rail level and at such a point in the span as to produce the maximum effect in the element under consideration.

The vertical effects of this load on secondary elements such as rail bearers shall be considered. For elements supporting more than one track a single load, as specified, shall be deemed sufficient.

8.2.9 **Centrifugal load.** Where the track on a bridge is curved, allowance for centrifugal action of moving loads shall be made in designing the elements, all tracks on the structure being considered occupied. The nominal centrifugal load F_c, in kN, per track acting radially at a height of 1.8m above rail level shall be calculated from the following formula.

$$F_c = \frac{P(v_t + 10)^2 \text{ x } f}{127r}$$

where

P is the static equivalent uniformly distributed load for bending moment when designing for RU loading; for RL loading, a distributed load of 40 kN/m multiplied by *L* is deemed sufficient.

r is the radius of curvature (in m)

v_t is the greatest speed envisaged on the curve in question (in km/h)

f $$= 1 - \left[\frac{V_t - 120}{1000}\right] \text{x} \left[\frac{814}{V_t} + 1.75\right] \text{x} \left[1 - \sqrt{\frac{2.88}{L}}\right]$$

for *L* greater than 2.88m and v_t over 120 km/h

= unity for *L* less than 2.88 m or v_t less than 120km/h

L is the loaded length of the element being considered.

8.2.10 **Longitudinal loads**. Provision shall be made for the nominal loads due to traction and application of brakes as given in table 18. These loads shall be considered as acting at rail level in a direction parallel to the tracks. No addition for dynamic effects shall be made to the longitudinal loads calculated as specified in this subclause.

For bridges supporting ballasted track, up to one-third of the longitudinal loads may be assumed to be transmitted by the track to resistances outside the bridge structure, provided that no expansion switches or similar rail discontinuities are located on, or within, 18m of either end of the bridge.

Structures and elements carrying single tracks shall be designed to carry the larger of the two loads produced by traction and braking in either direction parallel to the track.

Where a structure or element carries two tracks, both tracks shall be considered as being occupied simultaneously. Where the tracks carry traffic in opposite directions, the load due to braking shall be applied to one track and the load due to traction to the other. Structures and elements carrying two tracks in the same direction shall be subjected to braking or traction on both tracks, whichever gives the greater effect. Consideration, however, may have to be given to braking and traction, acting in opposite directions, producing rotational effects.

Where elements carry more than two tracks, longitudinal loads shall be considered as applied simultaneously to two tracks only.

8.2.11 **Aerodynamic effects from passing trains.** Provision shall be made for the nominal loads due to aeodynamic effects from passing trains. The requirements shall be agreed with the relevant authority.

8.3 **Load combinations.** All loads that derive from rail traffic, including dynamic effects, lurching, nosing, centrifugal load and longitudinal loads, shall be considered in combinations 1, 2 and 3.

In assessing combination 3, the combined response of the structure and the track to the longitudinal loads set out in 8.2.10, the thermal effects in the combined structure and track system and other effects such as creep and shrinkage, where appropriate, shall be considered. The requirements shall be agreed with the relevant authority.

Table 18. Nominal longitudinal loads

<table>
<tr><th>Standard loading type</th><th>Load arising from</th><th>Loaded length (m)</th><th>Longitudinal load (kN)</th></tr>
<tr><td rowspan="9">RU</td><td rowspan="5">Traction (30% of load on driving wheels)</td><td>up to 3</td><td>150</td></tr>
<tr><td>from 3 to 5</td><td>225</td></tr>
<tr><td>from 5 to 7</td><td>300</td></tr>
<tr><td>from 7 to 25</td><td>24 (L - 7) + 300</td></tr>
<tr><td>over 25</td><td>750</td></tr>
<tr><td rowspan="4">Braking (25% of load on braked wheels)</td><td>up to 3</td><td>125</td></tr>
<tr><td>from 3 to 5</td><td>187</td></tr>
<tr><td>from 5 to 7</td><td>250</td></tr>
<tr><td>over 7</td><td>20 (L - 7) + 250</td></tr>
<tr><td rowspan="8">RL</td><td rowspan="5">Traction (30% of load on driving wheels)</td><td>up to 8</td><td>80</td></tr>
<tr><td>from 8 to 30</td><td>10 kN/m</td></tr>
<tr><td>from 30 to 60</td><td>300</td></tr>
<tr><td>from 60 to 100</td><td>5 kN/m</td></tr>
<tr><td>over 100</td><td>500</td></tr>
<tr><td rowspan="3">Braking (25% of load on braked wheels)</td><td>up to 8</td><td>64</td></tr>
<tr><td>from 8 to 100</td><td>8 kN/m</td></tr>
<tr><td>over 100</td><td>800</td></tr>
</table>

8.4 **Design loads.** For primary and secondary railway live loads γ_{fL} shall be taken as follows:

	For the ultimate limit state	For the serviceability limit state
Combination 1	1.40	1.10
Combination 2 and 3	1.20	1.00

8.5 **Derailment loads.** Railway bridges shall be so designed that they do not suffer excessive damage or become unstable in the event of a derailment. The following conditions shall be taken into consideration.

(a) For the serviceability limit state, derailed coaches or light wagons remaining close to the track shall cause no permanent damage.

(b) For the ultimate limit state, derailed locomotives or heavy wagons remaining close to the track shall not cause collapse of any major element, but local damage may be accepted.

(c) For overturning or instability, a locomotive and one following wagon balanced on the parapet shall not cause the structure as a whole to overturn, but other damage may be accepted.

Conditions (a), (b) and (c) are to be considered separately and their effects are not additive. Design loads applied in accordance with 8.5.1 and 8.5.2 for types RU and RL loading, respectively, may be deemed to comply with these requirements.

8.5.1 **Design load for RU loading.** The following equivalent static loads, with no addition for dynamic effects, shall be applied.

(a) For the serviceability limit state, either

(1) a pair of parallel vertical line loads of 20 kN/m each, 1.4m apart, parallel to the track and applied anywhere within 2 m either side of the track centre line; or

(2) an individual concentrated vertical load of 100 kN anywhere within the width limits specified in (1).

(b) For the ultimate limit state, eight individual concentrated vertical loads each of 180 kN, arranged on two lines 1.4 m apart, with each of the four loads 1.6m apart on line, applied anywhere on the deck.

(c) For overturning or instability, a single line vertical load of 80 kN/m applied along the parapet or outermost edge of the bridge, limited to a length of 20m anywhere along the span.

Loads specified in (a) and (b) shall be applied at the top surface of the ballast or other deck covering and may be assumed to disperse at 30° to the vertical onto the supporting structure.

8.5.2 **Design load for RL loading.** The following equivalent static loads, with no addition for dynamic effects, shall be applied.

(a) For the serviceability limit state, either

(1) a pair of parallel vertical line loads of 15 kN/m each, 1.4 m apart, parallel to the track and applied anywhere within 2 m either side of the track centre line (or within 1.4 m either side of the track centre line where the track includes a substantial centre rail for electric traction or other purposes); or

(2) an individual concentrated vertical load of 75 kN anywhere within the width limits specified in (1).

(b) For the ultimate limit state, four individual concentrated vertical loads each of 120 kN, arranged at the corners of a rectangle of length 2.0 m and width 1.4 m, applied anywhere on the deck.

(c) For overturning and instability, a single line vertical load of 30 kN/m, applied along the parapet or outermost edge of the bridge, limited to a length of 20 m anywhere along the span.

Loads specified in (a) and (b) shall be applied at the top surface of the ballast or other deck covering and may be assumed to disperse at 30° to the vertical onto the supporting structure.

8.6 **Collision load on supports of bridges over railways*.** The collision load on supports of bridges over railways shall be as agreed with the appropriate authorities.

8.7 **Loading for fatigue investigations.** All elements of bridges subject to railway loading shall be checked against the effects of fatigue caused by repeated cycles of live loading. The number of load cycles shall be based on a life expectancy of 120 years for bridges intended as permanent structures. The load factor to be used in all cases when considering fatigue is 1.0.

For RU and RL loading the 120-year load spectrum, which has been calculated from traffic forecasts for the types of line indicated, shall be in accordance with Part 10 of this standard.

8.8 **Deflection Requirements.** The deflection limits (vertical, lateral, longitudinal, rotational) for railway bridges shall be as specified by the relevant authority.

8.9 **Footway and cycle track loading on railway bridges.** The requirements of 6.5.1.1 and 6.5.1.2 shall apply to railway bridges except that where reference is made to notional traffic lanes in 6.5.1.2 this shall be taken as referring instead to railway tracks. The nominal pedestrian live load specified in 6.5.1.2 shall be considered in combination with the nominal primary live load on the railway track. To determine design loads, γ_{fL} to be applied to the nominal loads shall be as specified in 6.5.3 and 8.4 respectively.

*Requirements for the supports of bridges over highways and waterways are specified in 6.8.

APPENDIX A

BASIS OF HA AND HB HIGHWAY LOADING

Type HA loading has been revised to take into account the results of recent research into the factors affecting loading, the large increase in the numbers of heavy goods vehicles and a better understanding of the loading patterns on long span bridges. The type HA loading is the normal design loading for Great Britain where it adequately covers the effects of all permitted normal vehicles* other than those used for the carriage of abnormal indivisible loads.

For short loaded lengths, the main factors which influence the loading are impact, overloading and lateral bunching. Research has shown that the impact effect of an axle on highway bridges can be as high as 80% of the static axle weight and an allowance of this magnitude was made in deriving the HA loading. The impact factor was applied to the highest axle load and only included in the single vehicle loading case. The amount of overloading of axle and vehicle weights was determined from a number of roadside surveys. The overloading factor was taken as a constant for loaded lengths between 2 and 10m reducing linearly from 10m to unity at a loaded length of 60m, where, with up to seven vehicles in convoy it could reasonably be expected that any overloaded vehicles would be balanced by partially laden ones. Allowance has also been made for the case where more than one line of vehicles can squeeze into traffic lane. The factor was based on the ratio of the standard lane width, 3.65m, to the maximum permitted width of normal vehicles* which is 2.5m. The HA loading is therefore given in terms of a 3.65m standard lane width and corresponding compensating width factors have been provided to allow for the cases where the actual lane widths are less than the standard lane width. The loading derived after application of the factors was considered to represent the ultimate load from which nominal loads were obtained by dividing by 1.5. The loading has been derived for a single lane only, but it is assumed for short spans that if two adjacent lanes are loaded there is a reasonable chance that they will be equally loaded.

There has been a significant increase in the number of heavier vehicles within the overall heavy goods vehicle population since the loading specified in BS 153 and generally adopted in BS 5400: Part 2 was derived. This has led to the frequent occurrence of convoys consisting of closely spaced, heavy types of heavy goods vehicles which has resulted in higher loading effects than were originally envisaged. The maximum weights of normal commercial vehicles permitted in Great Britain have also increased but the effects of this have been limited by restrictions on axle weights and spacing.

For long loaded lengths, the main factors affecting the loading are the traffic flow rates, percentage of heavy vehicles in the flows, frequency of occurrence and duration of traffic jams and, the spacing of vehicles in a jam. These parameters were determined by studying the traffic patterns at several sites on trunk roads, by load surveys at other sites and, where the required data was unobtainable, by estimation. A statistical approach was adopted to derive characteristic loadings from which nominal loads where obtained. Sensitivity analyses were carried out to test the significance on the loading of some of the assumptions made.

HB loading requirements derive from the nature of exceptional industrial loads (eg electrical transformers, generators, pressure vessels, machine presses, etc.) likely to use the roads in the area.

*As defined in The Road Vehicles (Construction and Use) Regulations 1986 (S.I. 1986/1078) and subsequent amendments and The Road Vehicles (Authorised Weight) Regulations (S.I. 1998/3111) available from HMSO.

APPENDIX B

VIBRATION SERVICEABILITY REQUIREMENTS FOR FOOT AND CYCLE TRACK BRIDGES

B.1 **General.** For superstructures for which the fundamental natural frequency of vibration exceeds 5Hz for the unloaded bridge in the vertical direction and 1.5 Hz for the loaded bridge in the horizontal direction, the vibration serviceability requirement is deemed to be satisfied.

For superstructures where f_o is equal to, or less than 5 Hz, the maximum vertical acceleration of any part of the superstructure shall be limited to $0.5\sqrt{f_o}$ m/s^2. The maximum vertical acceleration shall be calculated in accordance with B.2 or B.3, as appropriate.

A method for determining the vertical fundamental frequency f_o is given in B.2.3.

Where the fundamental frequency of horizontal vibration is less than 1.5 Hz, special consideration shall be given to the possibility of excitation by pedestrians of lateral movements of unacceptable magnitude. Bridges having low mass and damping and expected to be used by crowds of people are particularly susceptable to such vibrations. The method for deriving maximum horizontal acceleration should be agreed with the appropriate authority.

B.2 **Simplified method for deriving maximum vertical acceleration.** This method is valid only for single span, or two-or-three-span continuous, symmetric superstructures, of constant cross section and supported on bearings that may be idealised as simple supports.

The maximum vertical acceleration a (in m/s^2) shall be taken as

$$a = 4\pi^2 f_o^{\,2} y_s k \psi$$

where

f_o is the fundamental natural frequency (in Hz) (see B.2.3).
y_s is the static deflection (in m) (see B.2.4.)
k is the configuration factor (see B.2.5)
ψ is the dynamic response factor (see B.2.6)

For values of f_o greater than 4 Hz the calculated maximum acceleration may be reduced by an amount varying linearly from zero reduction at 4 Hz to 70% reduction at 5 Hz.

B.2.1 **Modulus of elasticity**. In calculating the values of f_o and y_s, the short-term modulus of elasticity shall be used for concrete (see Parts 7 and 8 of this standard), and for steel as given in Part 6 of this standard.

B.2.2 **Second moment of area**. In calculating the values of f_o and y_s, the second moment of area for sections of discrete concrete members may be used on the entire uncracked concrete section ignoring the presence of reinforcement. The effects of shear lag need not be taken into account in steel and concrete bridges.

B.2.3 **Fundamental natural frequency f_o.** The fundamental natural frequency f_o is evaluated for the bridge including superimposed dead load but excluding pedestrian live loading and may be calculated from the following formula.

$$f_o = \frac{C^2}{2\pi l^2}\sqrt{\frac{EIg}{M}}$$

where

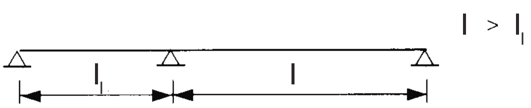
g is the acceleration due to gravity (in m/s^2)
l is the length of the main span (in m)
C is the configuration factor (see table 19)
E is the modulus of elasticity (in kN/m^2) (see B.2.1)
I is the second moment of area of the cross section at midspan (in m^4) (see B.2.2)
M is the weight per unit length of the full cross-section at midspan (in kN/m)

Midspan values of I and M shall be used only when there is no significant change in depth or weight of the bridge throughout the span. Where the value of I/M at the support exceeds twice, or is less than 0.8 times, the value at midspan, average values of I and M shall be used.

The stiffness of the parapets shall be included where they contribute to the overall flexural stiffness of the superstructure.

Values of C shall be obtained from table 19.

Table 19. Configuration factor C

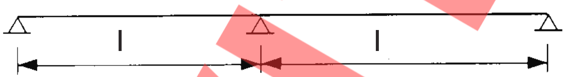
Bridge configuration	Ratio l_l/l	C
	–	π
	0.25 0.50 0.75 1.00	3.70 3.55 3.40 π
	0.25 0.50 0.75 1.00	4.20 3.90 3.60 π

For two-span and three-span continuous bridges, intermediate values of C may be obtained by linear interpolation.

B.2.4 **Static deflection y_s.** The static deflection y_s is taken at the midpoint of the main span for a vertical concentrated load of 0.7 kN applied at this point. For three-span superstructures, the centre span is taken as the main span.

B.2.5 **Configuration factor K.** Values of K shall be taken from table 20.

Table 20. Configuration factor K

Bridge configuration		K
	–	1.0
	–	0.7
	Ratio l_l/l 1.0 0.8 0.6 or less	 0.6 0.8 0.9

For three-span continuous bridges, intermediate values of K may be obtained by linear interpolation.

B.2.6 **Dynamic response factor ψ.** Values of ψ are given in figure 17. In the absence of more precise information, the values of δ (the logarithmic decrement of the decay of vibration due to structural damping) given in table 21 should be used.

Table 21. Logarithmic decrement of decay of vibration δ

Bridge superstructure	δ
Steel with asphalt or epoxy surfacing	0.03
Composite steel/concrete	0.04
Prestressed and reinforced concrete	0.05

B.3 **General method for deriving maximum vertical acceleration.** For superstructures other than those specified in B.2, the maximum vertical acceleration should be calculated assuming that the dynamic loading applied by a pedestrian can be represented by a pulsating point load F, moving across the main span of the superstructure at a constant speed v_t as follows:

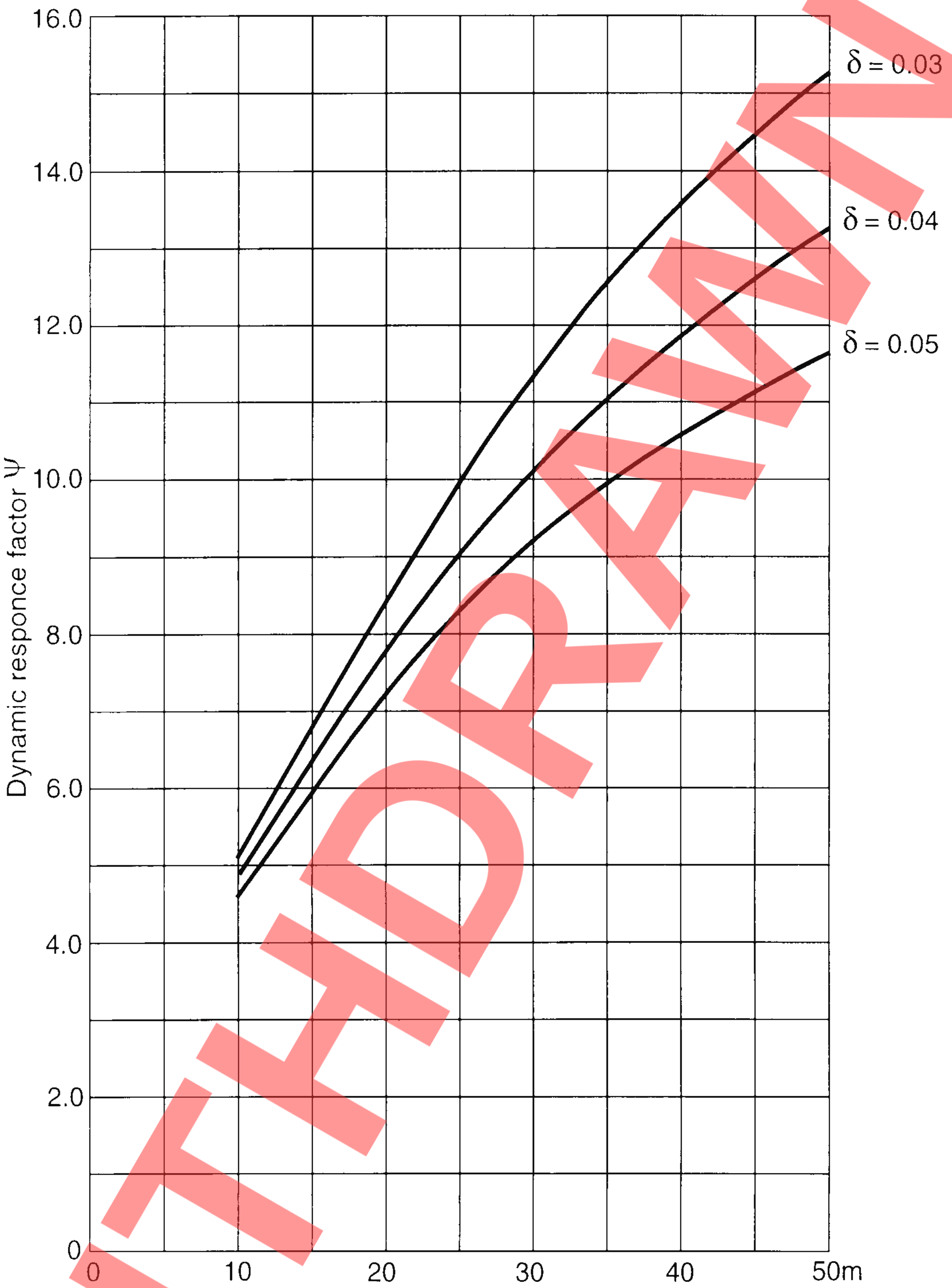
$F = 180 \sin 2\pi f_o T$ (in N), where T is the time (in s)

$v_t = 0.9 f_o$ (in m/s)

For values of f_o greater than 4 Hz, the calculated maximum acceleration may be reduced by an amount varying linearly from zero reduction at 4 Hz to 70% reduction at 5 Hz.

B.4 **Damage from forced vibration.** Consideration should be given to the possibility of permanent damage to a superstructure by a group of pedestrians deliberately causing resonant oscillations of the superstructure. As a general precaution, therefore, the bearings should be of robust construction with adequate provision to resist upward or lateral movement.

For prestressed concrete construction, resonant oscillation may result in a reversal of up to 10% of the static live load bending moment. Providing that sufficient unstressed reinforcement is available to prevent gross cracking, no further consideration need be given to this effect.



Main span 1

NOTE 1. Main span 1 is shown in table 20.
NOTE 2. Values of δ for different types of construction are given in table 21.

Figure 17. Dynamic reponse factor ψ

APPENDIX C

TEMPERATURE DIFFERENCES T FOR VARIOUS SURFACING DEPTHS

The values of T given in figure 9 are for 40mm surfacing depths for groups 1 and 2 and 100mm surfacing depths for groups 3 and 4. For other depths of surfacing, the values given in tables 22 to 24 may be used. In Tables 22 to 24 all surfacing depths include waterproofing thickness and values for waterproofed decks are conservative, assuming dark material. There may be some alleviation when light coloured waterproofing is used. Specialist advice should be sought if advantage of this is required. These values are based on the temperature difference curves developed under various studies by TRL and are generally appropriate and conservative for design purposes and allow for typical construction configurations, surfacing materials or colour. In situations where temperature effects are critical specialist advice should be sought if required.

Table 22a Values of T for group 1

Surface thickness	Postive temperature difference °C				Reverse temperature difference °C
mm	T_1	T_2	T_3	T_4	T_1
Unsurfaced	30	16	6	3	8
20	27	15	9	5	6
40	24	14	8	4	6

Table 22b Values of T for group 2

Surface thickness	Positive temperature difference °C	Reverse temperature difference °C
mm	T_1	T_1
Unsurfaced	25	6
20	23	5
40	21	5

Table 23 Values of T for group 3

Depth of slab (h)	Surfacing thickness	Positive temperature difference T_1	Reverse temperature difference T_1
m	mm	°C	°C
0.2	unsurfaced	16.5	5.9
	waterproofed	23.0	5.9
	50	18.0	4.4
	100	13.0	3.5
	150	10.5	2.3
	200	8.5	1.6
0.3	unsurfaced	18.5	9.0
	waterproofed	26.5	9.0
	50	20.5	6.8
	100	16.0	5.0
	150	12.5	3.7
	200	10.0	2.7

Table 24. Values of T for group 4

Depth of slab (h)	Surfacing thickness	Positive temperature difference			Reverse temperature difference			
		T_1	T_2	T_3	T_1	T_2	T_3	T_4
m	mm	°C	°C	°C	°C	°C	°C	°C
<0.2	unsurfaced	12.0	5.0	0.1	4.7	1.7	0.0	0.7
	waterproofed	19.5	8.5	0.0	4.7	1.7	0.0	0.7
	50	13.2	4.9	0.3	3.1	1.0	0.2	1.2
	100	8.5	3.5	0.5	2.0	0.5	0.5	1.5
	150	5.6	2.5	0.2	1.1	0.3	0.7	1.7
	200	3.7	2.0	0.5	0.5	0.2	1.0	1.8
0.4	unsurfaced	15.2	4.4	1.2	9.0	3.5	0.4	2.9
	waterproofed	23.6	6.5	1.0	9.0	3.5	0.4	2.9
	50	17.2	4.6	1.4	6.4	2.3	0.6	3.2
	100	12.0	3.0	1.5	4.5	1.4	1.0	3.5
	150	8.5	2.0	1.2	3.2	0.9	1.4	3.8
	200	6.2	1.3	1.0	2.2	0.5	1.9	4.0
0.6	unsurfaced	15.2	4.0	1.4	11.8	4.0	0.9	4.6
	waterproofed	23.6	6.0	1.4	11.8	4.0	0.9	4.6
	50	17.6	4.0	1.8	8.7	2.7	1.2	4.9
	100	13.0	3.0	2.0	6.5	1.8	1.5	5.0
	150	9.7	2.2	1.7	4.9	1.1	1.7	5.1
	200	7.2	1.5	1.5	3.6	0.6	1.9	5.1
0.8	unsurfaced	15.4	4.0	2.0	12.8	3.3	0.9	5.6
	waterproofed	23.6	5.0	1.4	12.8	3.3	0.9	5.6
	50	17.8	4.0	2.1	9.8	2.4	1.2	5.8
	100	13.5	3.0	2.5	7.6	1.7	1.5	6.0
	150	10.0	2.5	2.0	5.8	1.3	1.7	6.2
	200	7.5	2.1	1.5	4.5	1.0	1.9	6.0
1.0	unsurfaced	15.4	4.0	2.0	13.4	3.0	0.9	6.4
	waterproofed	23.6	5.0	1.4	13.4	3.0	0.9	6.4
	50	17.8	4.0	2.1	10.3	2.1	1.2	6.3
	100	13.5	3.0	2.5	8.0	1.5	1.5	6.3
	150	10.0	2.5	2.0	6.2	1.1	1.7	6.2
	200	7.5	2.1	1.5	4.8	0.9	1.9	5.8
>1.5	unsurfaced	15.4	4.5	2.0	13.7	1.0	0.6	6.7
	waterproofed	23.6	5.0	1.4	13.7	1.0	0.6	6.7
	50	17.8	4.0	2.1	10.6	0.7	0.8	6.6
	100	13.5	3.0	2.5	8.4	0.5	1.0	6.5
	150	10.0	2.5	2.0	6.5	0.4	1.1	6.2
	200	7.5	2.1	1.5	5.0	0.3	1.2	5.6

Temperatures for the calculation of loads and/or load effects

1. Maximum temperatures:

(a) Determine the maximum effective bridge temperature from figure 8 and table 11. For the purposes of this example, let its value be X°C.

(b) Determine the positive temperature difference distribution through the superstructure from figure 9.

(c) Assume that the temperature differences which form this distribution are actual temperatures.

(d) Using the assumed actual temperatures derived in (c), the geometry of the superstructure, and Appendix 1 of TRRL Report LR 765, calculate the effective bridge temperature. For the purposes of this example, let its value be Y°C.

(e) Add (X-Y) °C to all the assumed actual temperatures derived in (c). These are now the temperatures which co-exist with the maximum effective bridge temperature and positive temperature difference distribution determined from (a) and (b) respectively, and which are to be used for the calculation of loads and/or load effects.

2. Minimum temperatures:

Proceed as for the calculation of maximum temperatures, but use figure 7, table 10 and the reversed temperature difference distributions shown in figure 9. In step (c) regard the assumed actual temperatures to be NEGATIVE. In step (e) add (X-Y) °C to all the assumed actual negative temperatures derived in step (c). These are now the temperatures which co-exist with the minimum effective bridge temperature and reversed temperature difference distribution determined from steps (a) and (b) respectively, and which are to be used for the calculation of loads and/or load effects.

APPENDIX D

Derivation of RU and RL railway loadings

D.1 **RU loading.** The loading given in 8.2.1 has been derived by a Committee of the International Union of Railways to cover present and anticipated future loading on railways in Great Britain and on the Continent of Europe. Motive power now tends to be diesel and electric rather than steam, and this produces axle loads and arrangements for locomotives that are similar to those used for bogie freight vehicles, freight vehicles often being heavier than locomotives. In addition to the normal train loading, which can be represented quite well by a uniformly distributed load of 8 t/m, railway bridges are occasionally subject to exceptionally heavy abnormal loads. At short loaded lengths it is necessary to introduce heavier concentrated loads to simulate individual axles and to produce high end shears. Certain vehicles exceed RU static loading at certain spans, particularly in shear, but these excesses are acceptable because dynamic factors applied to RU loading assume high speeds whereas those occasional heavy loads run at much lower speeds.

The concentrated and distributed loads have been approximately converted into equivalent loads measured in kN when applying RU loading in this British Standard.

Figure 18 shows diagrams of two locomotives and several wagons all of which, when forming part of a train, are covered by RU loading. Double heading of the locomotives has been allowed for in RU loading.

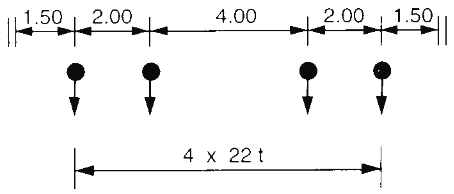
The allowances for dynamic effects for RU loading given in 8.2.3.1 have been calculated so that, in combination with that loading, they cover the effects of slow moving heavy, and fast moving light, vehicles. Exceptional vehicles are assumed to move at speeds not exceeding 80 km/h, heavy wagons at speeds up to 120 km/h, passenger trains at speeds up to 200 km/h.

The formulae for the dynamic effects are not to be used to calculate dynamic effects for a particular train on a particular bridge. Appropriate methods for this can be found by reference to a recommendation published by the International Union of Railways (UIC), Paris*

Similar combinations of vehicle weight and speed have to be considered in the calculation of centrifugal loads. The factor f given in 8.2.9 allows for the reduction in vehicle weight with increasing speed above certain limits. The greatest envisaged speed is that which is possible for the alignment as determined by the physical conditions at the site of the bridge.

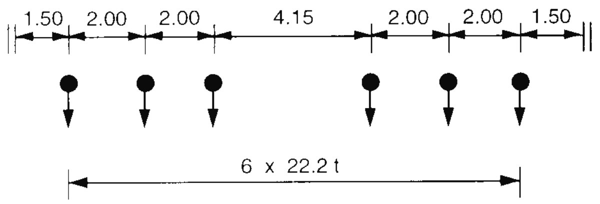
*Leaflet 776-1R, published by UIC, 16 rue Jean-Rey F, 75015 Paris, France

B-B LOCOMOTIVE



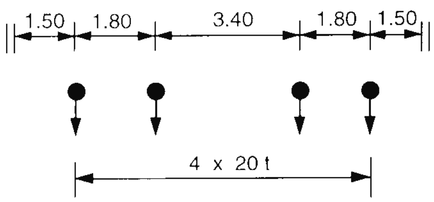
To be considered
Double headed

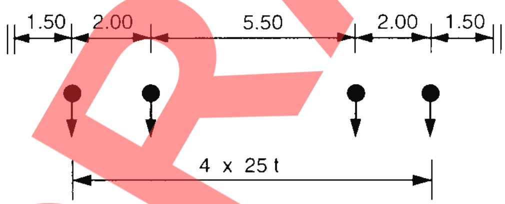
C-C LOCOMOTIVE



To be considered
Double headed

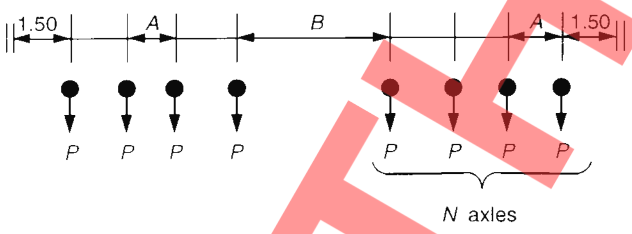
WAGONS





EXCEPTIONAL WAGONS

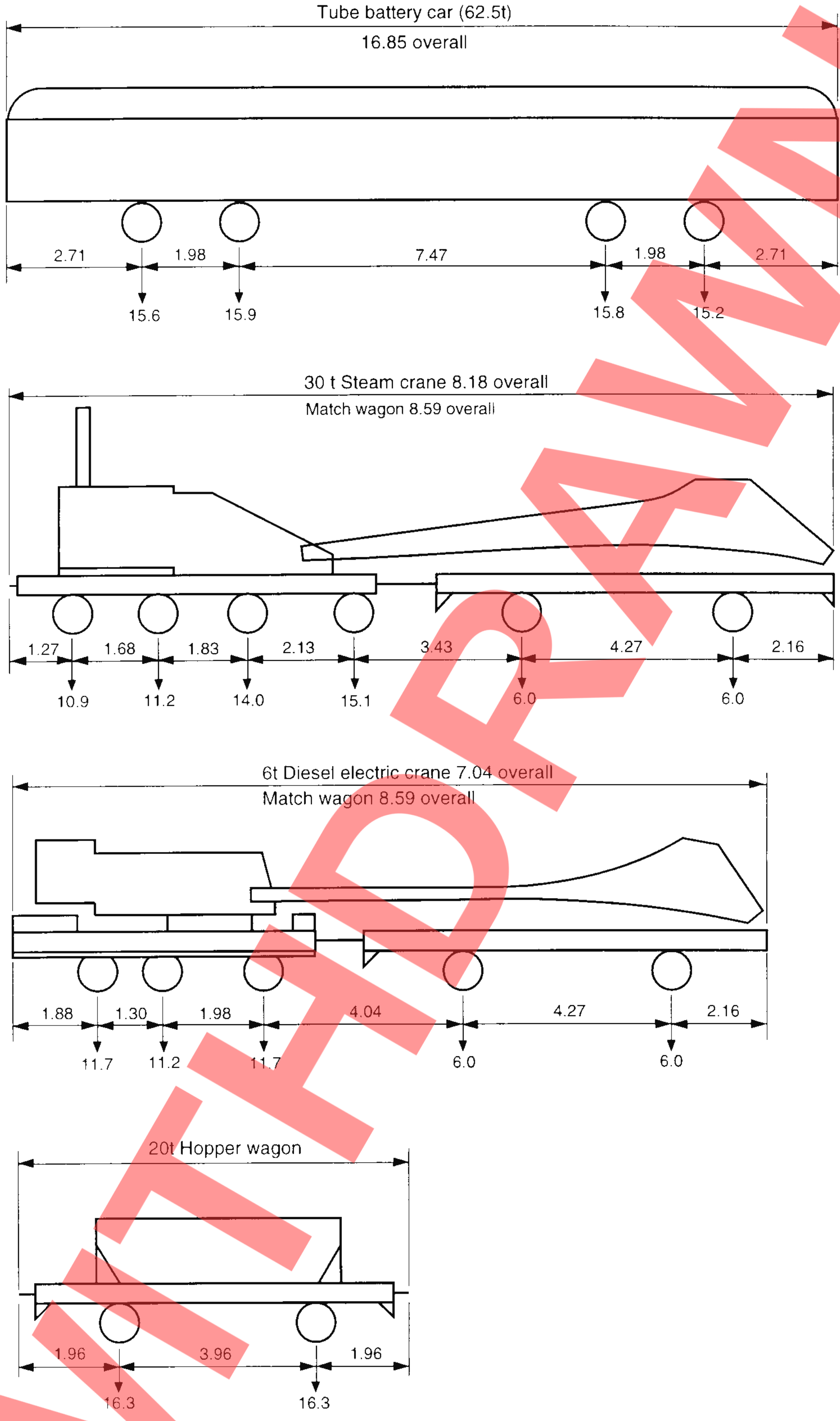
N is the number of axles in each end
2*N* is the total number of axles
t = metric tonnes



P Weight per axle (t)	A in metres, number of axles N						
	3	4	5	6	8	10	14
16				1.25	1.45	1.55	1.60
17				1.35	1.60	1.65	1.70
18			1.25	1.45	1.75	1.75	1.80
19			1.35	1.65	1.85	1.85	1.90
20	1.25	1.25	1.45	1.80	1.95	2.00	2.00
21	1.30	1.30	1.70	2.00	2.10	2.10	2.15
22	1.40	1.40	1.90	2.15	2.20	2.20	2.25
	B in metres						
No variation	6.00	6.00	7.00	8.00	8.00	10.00	10.00

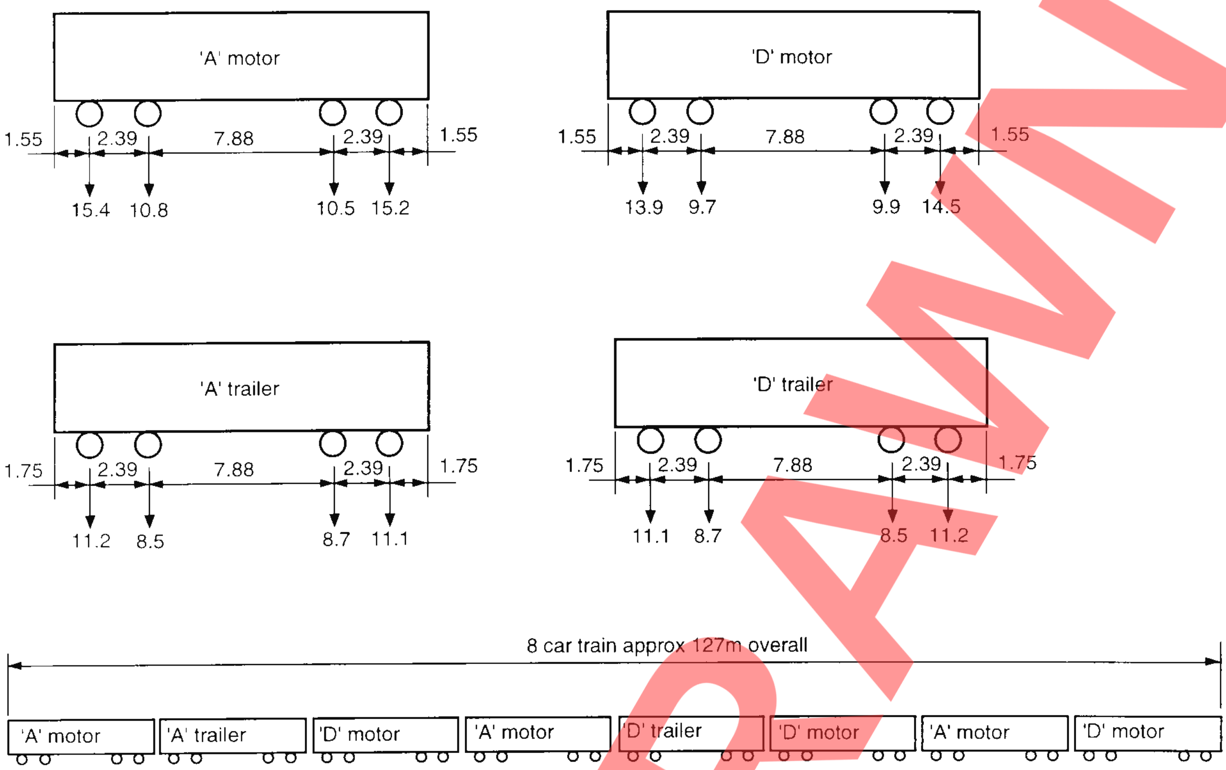
Note. Dimensions in metres.

Figure 18. Wagons and locomotives covered by RU loading

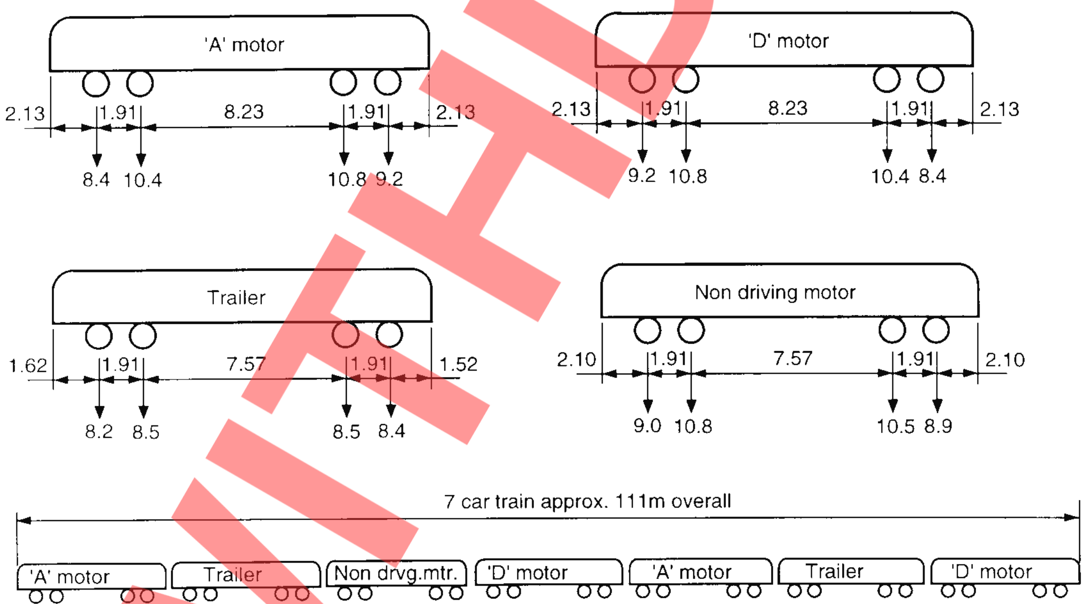


Dimensions are in metres. Axle loads are in tonnes

Figure 19. Works trains vehicles covered by RL loading



Heaviest surface stock



Heaviest tube stock

Dimensions are in metres. Axle loads are in tonnes

Figure 20. Passenger vehicles covered by RL loading

D.2 **RL loading.** The loading specified in 8.2.2 has been derived by the London Transport Executive to cover present and anticipated loading on lines that only carry rapid transit passenger stock and light engineers' works trains. This loading should not be used for lines carrying 'main line' locomotives or stock. Details are included in this appendix to allow other rapid transit passenger authorities to compare their actual loading where standard track of 1.432 m gauge is used but where rolling stock and locomotives are lighter than on the main line UIC railways.

RL loading covers the following conditions, which are illustrated in figures 19 and 20.

(a) Works trains. This constitutes locomotives, cranes and wagons used for maintenance purposes. Locomotives are usually of the battery car type although very occasionally diesel shunters may be used. Rolling stock hauled includes a 30t steam crane, 6t diesel cranes, 20t hopper wagons and bolster wagons. The heaviest train would comprise loaded hopper wagons hauled by battery cars.

(b) Passenger trains. A variety of stock of different ages, loadings and load gauges is used on surface and tube lines.

The dynamic factor has been kept to a relatively low constant, irrespective of span, because the heavier loads, which determine the static load state, arise from works trains which only travel at a maximum speed of about 32 km/h. The faster passenger trains produce lighter axle loads and a greater margin is therefore available for dynamic effects.

Loading tests carried out in the field on selected bridges produced the following conclusions.

(a) Main girders

(1) Works trains produce stresses about 20% higher than static stresses.

(2) Passenger trains produce stresses about 30% higher than static stresses.

(b) Cross girders and rail bearers (away from rail joints). All types of train produce stresses about 30% higher than static stresses.

(c) Cross girders and rail bearers at rail joints

(1) With no ballast, one member carrying all the joint effect (e.g. rail bearer or cross girder immediately under joint with no distribution effects), all trains can produce an increase over static stress of up to 27% for each l0km/h of speed.

(2) With no ballast, but with some distribution effects (e.g. cross girder with continuous rail bearers or heavy timbers above), all trains can produce an increase over static stress of up to 20% for each 10 km/h of speed.

(3) With ballasted track, the rail joint effect is considerably reduced, depending on the standard and uniformity of compaction of ballast beneath the sleepers. The maximum increase in poorly maintained track is about 12% for each 10 km/h of speed.

The equivalent static loading is over generous for short loaded lengths. However, it is short members that are most severely affected by the rail joint effect and, by allowing the slight possibility of a small overstress under ballasted rail joints, it has been found possible to adopt a constant dynamic factor of 1.2 to be applied to the equivalent static loading.

For the design of bridges consisting of independently acting linear members, the effects of trains are adequately covered by the effects of the basic RL loading system. Recent trends, however, are towards the inclusion of plate elements as principal deck members, and here the load representation is inadequate. A reinforced concrete slab deck between steel main girders, for example, will distribute concentrated loads over a significant length of the main girders and in consequence suffers longitudinal stresses from bending, shear and torsion.

To cater for this consideration, a check loading bogie has been introduced. This should be used only on deck structures to check the ability of the deck to distribute the load adequately. To allow for dynamic effects, an addition of 12% per 10 km/h of speed has been made to the heaviest axle, assumed to be at a rail joint, and an additional 30% has been made to the other axle of the bogie.

D.3 **Use of tables 25 to 28 when designing for RU loading**

D.3.1 **Simply supported main girders and rail bearers.** Bending moments in simply supported girders are to be determined using the total equivalent uniformly distributed load given in the tables for the span of the girder, assuming a parabolic bending moment diagram.

End shears and support reactions for such girders shall be taken from the tables giving end shear forces.

Shear forces at points other than the end shall be determined by using the static shear force from table 26 for a span equal to that of the length of shear influence line for the points under consideration. The static shear thus calculated shall be multiplied by the appropriate ratio (figure 21) and the result shall be multiplied by the dynamic factor for shear in which *L* is taken to be the span of the girder.

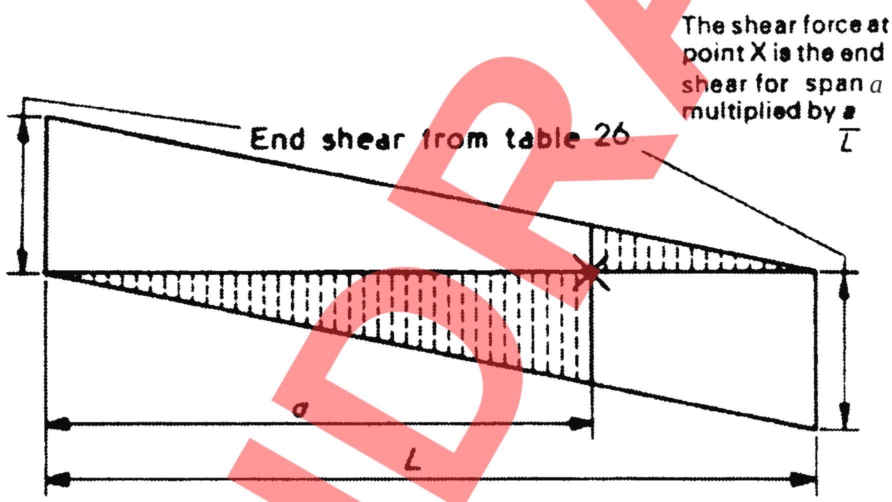


Figure 21. Shear force determination

D.3.2 **Cross girders loaded through simply supported rail bearers.** The cross girders shall be designed to carry two concentrated point loads for each track. Each of these loads is to be taken as one-quarter of the equivalent uniformly distributed load for bending moments shown in table 25 for a span equal to twice the cross girder spacing, multiplied by the appropriate dynamic factor.

Table 25. Equivalent uniformly distributed loads for bending moments for simply supported beams (static loading) under RU loading

Span	Load	Span	Load	Span	Load
m	kN	m	kN	m	kN
1.0	500	8.0	1 257	50.0	4 918
1.2	500	8.2	1 282	52.0	5 080
1.4	500	8.4	1 306	54.0	5 242
1.6	500	8.6	1 330	56.0	5 404
1.8	501	8.8	1 353	58.0	5 566
2.0	504	9.0	1 376	60.0	5 727
2.2	507	9.2	1 399	65.0	6 131
2.4	512	9.4	1 422	70.0	6 534
2.6	518	9.6	1 444	75.0	6 937
2.8	523	9.8	1 466	80.0	7 340
3.0	545	10.0	1 488	85.0	7 742
3.2	574	11.0	1 593	90.0	8 144
3.4	601	12.0	1 695	95.0	8 545
3.6	627	13.0	1 793	100.0	8 947
3.8	658	14.0	1 889	105.0	9 348
4.0	700	15.0	1 983	110.0	9 749
4.2	738	16.0	2 075	115.0	10 151
4.4	773	17.0	2 165	120.0	10 552
4.6	804	18.0	2 255	125.0	10 953
4.8	833	19.0	2 343	130.0	11 354
5.0	860	20.0	2 431	135.0	11 754
5.2	886	22.0	2 604	140.0	12 155
5.4	910	24.0	2 775	145.0	12 556
5.6	934	26.0	2 944	150.0	12 957
5.8	956	28.0	3 112	155.0	13 357
6.0	978	30.0	3 279	160.0	13 758
6.2	1 004	32.0	3 445	165.0	14 158
6.4	1 036	34.0	3 610	170.0	14 559
6.6	1 067	36.0	3 775	175.0	14 959
6.8	1 097	38.0	3 939	180.0	15 360
7.0	1 126	40.0	4 103	185.0	15 760
7.2	1 154	42.0	4 267	190.0	16 161
7.4	1 181	44.0	4 430	195.0	16 561
7.6	1 207	46.0	4 593	200.0	16 961
7.8	1 232	48.0	4 755		

Table 26. End shear forces for simply supported beams (static loading) under RU loading

Span	Force	Span	Force	Span	Force
m	kN	m	kN	m	kN
1.0	252	8.0	729	50.0	2 529
1.2	255	8.2	740	52.0	2 610
1.4	260	8.4	752	54.0	2 691
1.6	266	8.6	763	56.0	2 772
1.8	278	8.8	774	58.0	2 852
2.0	300	9.0	785	60.0	2 933
2.2	318	9.2	795	65.0	3 134
2.4	333	9.4	806	70.0	3 336
2.6	347	9.6	817	75.0	3 537
2.8	359	9.8	827	80.0	3 738
3.0	371	10.0	837	85.0	3 939
3.2	383	11.0	888	90.0	4 139
3.4	397	12.0	937	95.0	4 340
3.6	417	13.0	984	100.0	4 541
3.8	434	14.0	1 030	105.0	4 741
4.0	450	15.0	1 076	110.0	4 942
4.2	465	16.0	1 120	115.0	5 142
4.4	479	17.0	1 165	120.0	5 342
4.6	492	18.0	1 208	125.0	5 543
4.8	505	19.0	1 252	130.0	5 743
5.0	520	20.0	1 295	135.0	5 944
5.2	538	22.0	1 380	140.0	6 144
5.4	556	24.0	1 464	145.0	6 344
5.6	571	26.0	1 548	150.0	6 544
5.8	586	28.0	1 631	155.0	6 745
6.0	601	30.0	1 714	160.0	6 945
6.2	615	32.0	1 796	165.0	7 145
6.4	629	34.0	1 878	170.0	7 345
6.6	642	36.0	1 960	175.0	7 545
6.8	656	38.0	2 042	180.0	7 746
7.0	668	40.0	2 123	185.0	7 946
7.2	681	42.0	2 205	190.0	8 146
7.4	693	44.0	2 286	195.0	8 346
7.6	705	46.0	2 367	200.0	8 546
7.8	717	48.0	2 448		

Table 27. Equivalent uniformly distributed loads for bending moments for simply supported beams, including dynamic effects, under RU loading

Span	Load	Span	Load	Span	Load
m	kN	m	kN	m	kN
1.0	1 000	8.0	1 951	50.0	5 136
1.2	1 000	8.2	1 975	52.0	5 273
1.4	1 000	8.4	1 999	54.0	5 411
1.6	1 000	8.6	2 022	56.0	5 547
1.8	1 002	8.8	2 044	58.0	5 684
2.0	1 007	9.0	2 066	60.0	5 820
2.2	1 015	9.2	2 088	65.0	6 160
2.4	1 024	9.4	2 109	70.0	6 534
2.6	1 035	9.6	2 130	75.0	6 937
2.8	1 047	9.8	2 150	80.0	7 340
3.0	1 089	10.0	2 171	85.0	7 742
3.2	1 148	11.0	2 268	90.0	8 144
3.4	1 203	12.0	2 359	95.0	8 545
3.6	1 255	13.0	2 447	100.0	8 947
3.8	1 293	14.0	2 531	105.0	9 348
4.0	1 351	15.0	2 613	110.0	9 749
4.2	1 401	16.0	2 694	115.0	10 151
4.4	1 444	17.0	2 773	120.0	10 552
4.6	1 481	18.0	2 851	125.0	10 953
4.8	1 512	19.0	2 927	130.0	11 354
5.0	1 541	20.0	3 003	135.0	11 754
5.2	1 567	22.0	3 153	140.0	12 155
5.4	1 591	24.0	3 301	145.0	12 556
5.6	1 613	26.0	3 447	150.0	12 957
5.8	1 633	28.0	3 592	155.0	13 357
6.0	1 652	30.0	3 736	160.0	13 758
6.2	1 680	32.0	3 878	165.0	14 158
6.4	1 717	34.0	4 020	170.0	14 559
6.6	1 753	36.0	4 162	175.0	14 959
6.8	1 785	38.0	4 302	180.0	15 360
7.0	1 817	40.0	4 442	185.0	15 760
7.2	1 846	42.0	4 582	190.0	16 161
7.4	1 874	44.0	4 721	195.0	16 561
7.6	1 900	46.0	4 860	200.0	16 961
7.8	1 926	48.0	4 998		

Table 28. End shear forces for simply supported beams, including dynamic effects, under RU loading

Span	Force	Span	Force	Span	Force
m	kN	m	kN	m	kN
1.0	421	8.0	997	50.0	2 604
1.2	427	8.2	1 007	52.0	2 676
1.4	435	8.4	1 018	54.0	2 748
1.6	445	8.6	1 028	56.0	2 821
1.8	464	8.8	1 037	58.0	2 893
2.0	501	9.0	1 047	60.0	2 965
2.2	532	9.2	1 057	65.0	3 144
2.4	557	9.4	1 066	70.0	3 336
2.6	579	9.6	1 076	75.0	3 537
2.8	601	9.8	1 085	80.0	3 738
3.0	621	10.0	1 094	85.0	3 939
3.2	640	11.0	1 138	90.0	4 139
3.4	663	12.0	1 181	95.0	4 340
3.6	695	13.0	1 223	100.0	4 541
3.8	714	14.0	1 264	105.0	4 741
4.0	729	15.0	1 304	110.0	4 942
4.2	743	16.0	1 343	115.0	5 142
4.4	756	17.0	1 383	120.0	5 342
4.6	768	18.0	1 421	125.0	5 543
4.8	780	19.0	1 460	130.0	5 743
5.0	794	20.0	1 498	135.0	5 944
5.2	815	22.0	1 574	140.0	6 144
5.4	832	24.0	1 649	145.0	6 344
5.6	849	26.0	1 724	150.0	6 544
5.8	864	28.0	1 799	155.0	6 745
6.0	878	30.0	1 873	160.0	6 945
6.2	892	32.0	1 947	165.0	7 145
6.4	905	34.0	2 021	170.0	7 345
6.6	917	36.0	2 094	175.0	7 545
6.8	930	38.0	2 167	180.0	7 746
7.0	942	40.0	2 240	185.0	7 946
7.2	953	42.0	2 313	190.0	8 146
7.4	965	44.0	2 386	195.0	8 346
7.6	976	46.0	2 459	200.0	8 546
7.8	987	48.0	2 531		

APPENDIX E

Probability Factor S_p and Seasonal Factor

E.1 **Probability Factor, S_p**. The basic wind speed as defined in 5.3.2.2.1 has an annual probability of exceedance of Q = 0.02. To vary the basic wind speed for other annual probabilities of exceedance the basic wind speed should be multiplied by the probability factor S_p given by:

$$S_p = \sqrt{\left(\frac{\{5 - 1n(-1n[1-Q])\}}{\{5 - 1n(-1n0.98)\}}\right)}$$

where

Q is the required annual risk of exceedance.

This expression corresponds to a Fisher-Tippett Type 1 (FT1) model for dynamic pressure that has a characteristic product (mode/dispersion ratio) value of 5, which is valid for the U.K. climate only.

NOTE: S_p = 1.000 for Q = 0.02 corresponding to a mean recurrence interval of 50 years.

S_p = 1.048 for Q = 0.0083 corresponding to a mean recurrence interval of 120 years.

E.2 **Seasonal Factor, S_s**. For bridges which are expected to be exposed to the wind for specific sub-annual periods or when particular erection conditions need to be examined the seasonal factor S_s may be used to reduce the basic wind speeds while maintaining a risk of exceedance of Q = 0.02 in the stated period. The basic wind speed V_b to be used in 5.3.2.2.1 is then obtained as S_s times the wind speed obtained from Figure 2. The seasonal factor S_s may also be used in conjunction with the probability factor S_p for other risks of exceedance Q in the stated period. Values are given in Table 29.

Table 29. Values of seasonal factor S_s

<table>
<tr><th rowspan="2">Months</th><th colspan="7">Sub annual periods</th></tr>
<tr><th>1 month</th><th colspan="2">2 months</th><th colspan="4">4 months</th></tr>
<tr><td>Jan</td><td>0.98</td><td rowspan="2">0.98</td><td></td><td rowspan="4">0.98</td><td></td><td rowspan="2"></td><td rowspan="3"></td></tr>
<tr><td>Feb</td><td>0.83</td><td rowspan="2">0.86</td><td rowspan="4">0.87</td></tr>
<tr><td>Mar</td><td>0.82</td><td rowspan="2">0.83</td><td rowspan="4">0.83</td></tr>
<tr><td>Apr</td><td>0.75</td><td rowspan="2">0.75</td><td rowspan="4">0.76</td></tr>
<tr><td>May</td><td>0.69</td><td rowspan="2">0.71</td><td rowspan="4">0.73</td></tr>
<tr><td>Jun</td><td>0.66</td><td rowspan="2">0.67</td><td rowspan="4">0.83</td></tr>
<tr><td>Jul</td><td>0.62</td><td rowspan="2">0.71</td><td rowspan="4">0.86</td></tr>
<tr><td>Aug</td><td>0.71</td><td rowspan="2">0.82</td><td rowspan="4">0.90</td></tr>
<tr><td>Sept</td><td>0.82</td><td rowspan="2">0.85</td><td rowspan="4">0.96</td></tr>
<tr><td>Oct</td><td>0.82</td><td rowspan="2">0.89</td><td rowspan="4">1.00</td></tr>
<tr><td>Nov</td><td>0.88</td><td rowspan="2">0.95</td><td rowspan="4">1.00</td></tr>
<tr><td>Dec</td><td>0.94</td><td rowspan="2">1.00</td><td rowspan="4">1.00</td></tr>
<tr><td>Jan</td><td>0.98</td><td rowspan="2">0.98</td><td rowspan="3"></td></tr>
<tr><td>Feb</td><td>0.83</td><td rowspan="2">0.86</td><td rowspan="2"></td></tr>
<tr><td>Mar</td><td>0.82</td><td></td><td></td></tr>
</table>

NOTE: The factor for the six month winter period October to March inclusive is 1.0, and for the six month summer period April to September inclusive is 0.84.

APPENDIX F

Topography Factor S_h'

F.1 **General**. The factor S_h' allows for local topographical features such as hills, valleys, cliffs, escarpments or ridges which can significantly affect the wind speed in their vicinity. Near the summits of hills, or the crests of cliffs, escarpments or ridges the wind is accelerated. In valleys or near the foot of cliffs, steep escarpments or ridges, the wind may be decelerated. Values of S_h should be derived for each wind direction considered and used in conjunction with the corresponding direction factor S_d.

F.2 **Topography Significance**. Where the average slope of the ground does not exceed $\psi = 0.05$ within a kilometre radius of the site, the terrain should be taken as level and the topography factor S_h' should be taken as unity. When the topography is defined as not significant as defined in Figure 3, the terrain may be taken as level and the topography factor S_h' may be taken as unity, but implementation of the topography factor will produce a more accurate assessment.

F.3 **Altitude**. Depending on whether the topography factor, S_h', is used care should be taken to ensure that the altitude factor S_a, used to determine the site wind speed V_s, is derived from the appropriate definition of altitude Δ in 5.3.2.2.3.

F.4 **Gust Speeds**. The value of S_h', for deriving the maximum wind gust speed where topography needs to be taken into account, should be taken as:

$$S_h' = \left[1 + S_h . \frac{S_c T_c}{S_b T_g}\right]$$

where

S_h is defined in F.6

S_c is defined in 5.3.2.4.1

T_c is defined in 5.3.2.4.2

S_b is defined in 5.3.2.3.1

T_g is defined in 5.3.2.3.2

F.5 **Hourly Mean Speeds**. The value of S_h' for deriving the hourly mean wind speed for relieving areas should be taken as:

$$S_h' = [1 + S_h]$$

where

S_h is defined in F.6.

F.6 **Topography Features**

F.6.1 **General.** In the vicinity of local topographic features the factor S_h is a function of the upwind slope and the position of the site relative to the summit or crest, and will be within the range of $0 < S_h < 0.6$. It should be noted that S_h will vary with height above ground level, from maximum near to the ground reducing to zero at higher levels, and with position from the crest, from maximum near the crest reducing to zero distant from the crest.

In certain steep-sided enclosed valleys wind speeds may be less than in level terrain. Before any reduction in wind speeds is considered specialist advice should be sought. For sites in complex topography specialist advice should be sought. Alternatively, use the maximum value $S_h = 0.6$ given by the method. Values of S_h may be derived from

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full scale measurements or numerical simulations.

Values of topography factor are confined in this method to the range of $0 < S_h < 0.6$ and apply only to the simple topographic features defined in Figure 22. In situations of multiple hills or ridges, this procedure is appropriate when applied to the single hill or ridge on which the site is situated.

F.6.2 **Derivation**. The topographic dimensions defined in Figure 22 are:

L_D the actual length of the downwind slope in the wind direction

L_u the actual length of the upwind slope in the wind direction

Z the effective height of the feature

ψ the upwind slope Z/L in the wind direction

L_e the effective length of the upwind slope, defined in Table 30

H the height (of the bridge) above local ground level

X the horizontal distance of the site from the crest

s the factor to be obtained from Figures 23 and 24.

The influence of any topography feature should be considered to extend $X = 15\ L_e$ upwind and $X = 2.5\ L_e$ downwind of the summit or crest of the feature. Outside this zone the topography factor should be taken as $S_h = 0$.

If the zone downwind from the crest of the feature is level ($\psi < 0.05$) for a distance exceeding L_e then the feature should be treated as an escarpment. If not, then the feature should be treated as a hill or ridge (see Figure 22). No differentiation is made in deriving S_h between a three-dimensional hill and a two-dimensional ridge.

F.6.3 **Undulating Terrain**. In undulating terrain it is often not possible to decide whether the local topography of the site is significant in terms of wind flow. In such cases the average level of the terrain upwind of the site for a distance of 5km should be taken as the base level from which to assess the height Z and the upwind slope ψ of the feature.

F.6.4 **Value of factor S_h**. Values of the factor S_h should be obtained from Table 30 using the appropriate values for the slope of the hill ψ, the effective length L_e and the factor s which should be determined from:

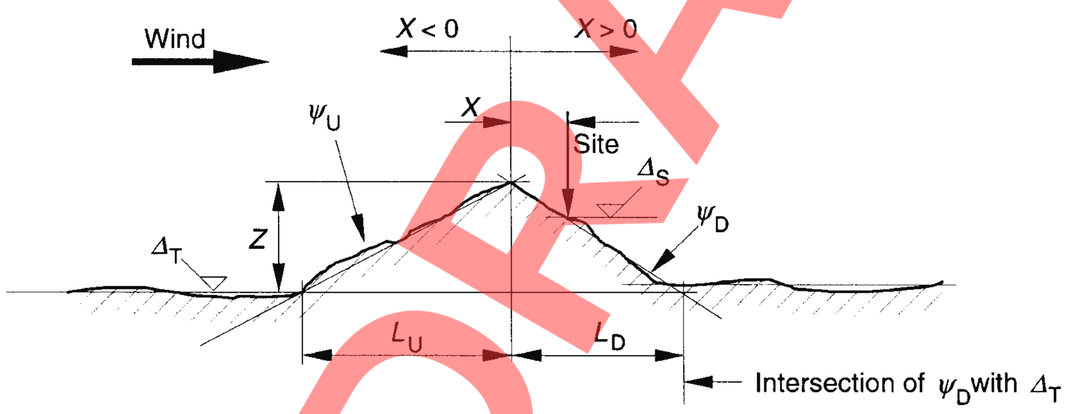
(a) Figure 23 for hills and ridges; or

(b) Figure 24 for cliffs and escarpments.

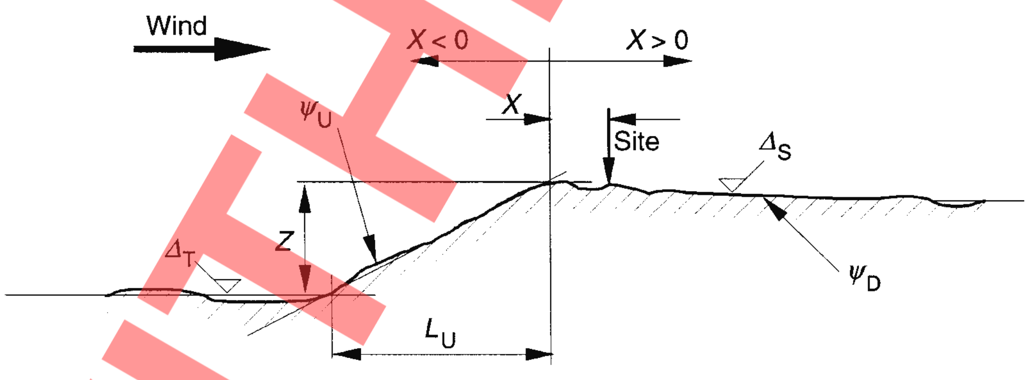
Where the downwind slope of a hill or ridge is greater than $\psi = 0.3$ there will be large regions of reduced acceleration or even shelter and it is not possible to give precise design rules to cater for these circumstances. Values of s from Figure 23 should be used as upper bound values.

Table 30. Values of L_e and S_h

Slope ($\psi = Z/L_u$)	Shallow ($0.05 < \psi < 0.3$)	Steep ($\psi > 0.3$)
Effective Length	$L_e = L_u$	$L_e = Z/0.3$
Topography Factor	$S_h = 2.0 \psi s$	$S_h = 0.6 s$



a) Hill and ridge ($\psi_U > 0.05$, $\psi_D > 0.05$)



b) Escarpment ($0.3 > \psi_U > 0.05$, $\psi_D < 0.05$) and cliff ($\psi_U > 0.3$, $\psi_D < 0.05$)

Key

- L_D Length of the downwind slope in the wind direction
- L_U Length of the upwind slope in the wind direction
- X Horizontal distance of the site from the crest
- Z Effective height of the feature
- Δ_S Site altitude in metres above mean sea level
- Δ_T Altitude of upwind base of topographic feature
- ψ_U Upwind slope Z/L_U in the wind direction
- ψ_D Downwind slope Z/L_D in the wind direction

Figure 22 - Definition of topographic dimensions

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a)

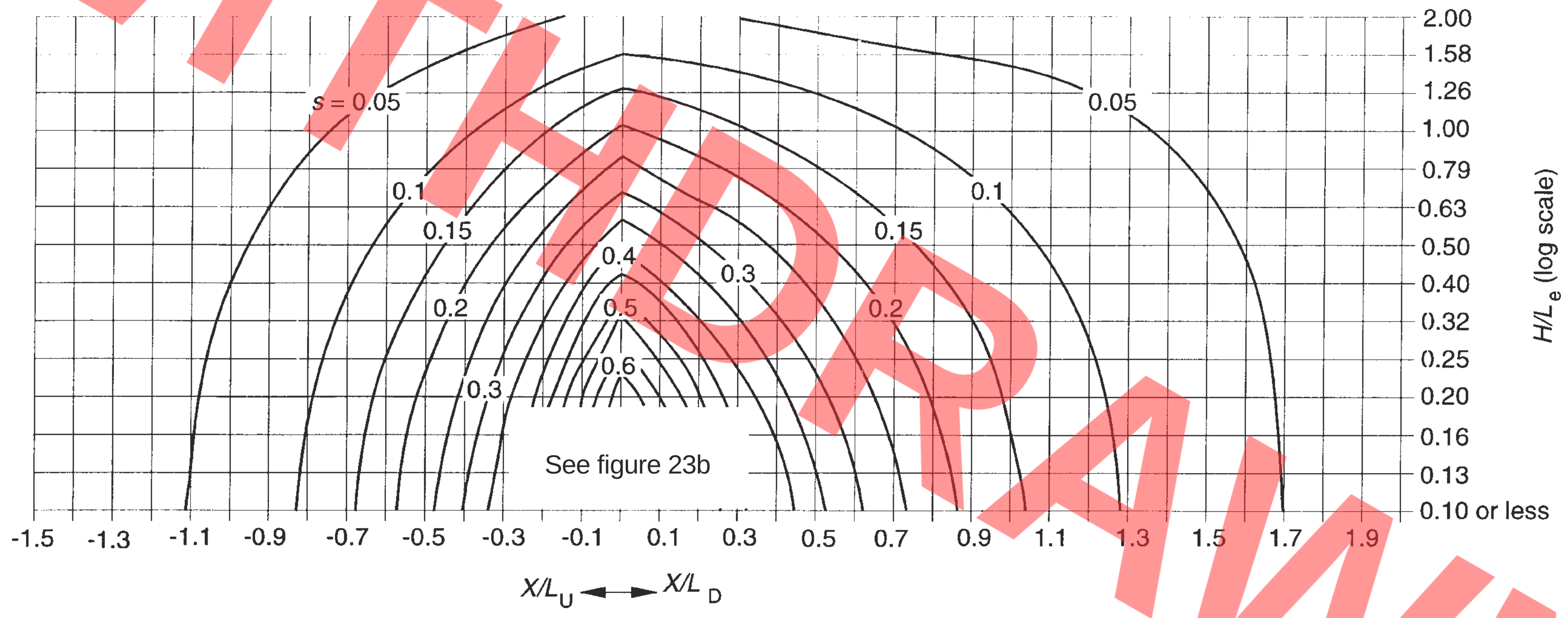

Figure 23a - Topographic location factors for hills and ridges

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b)

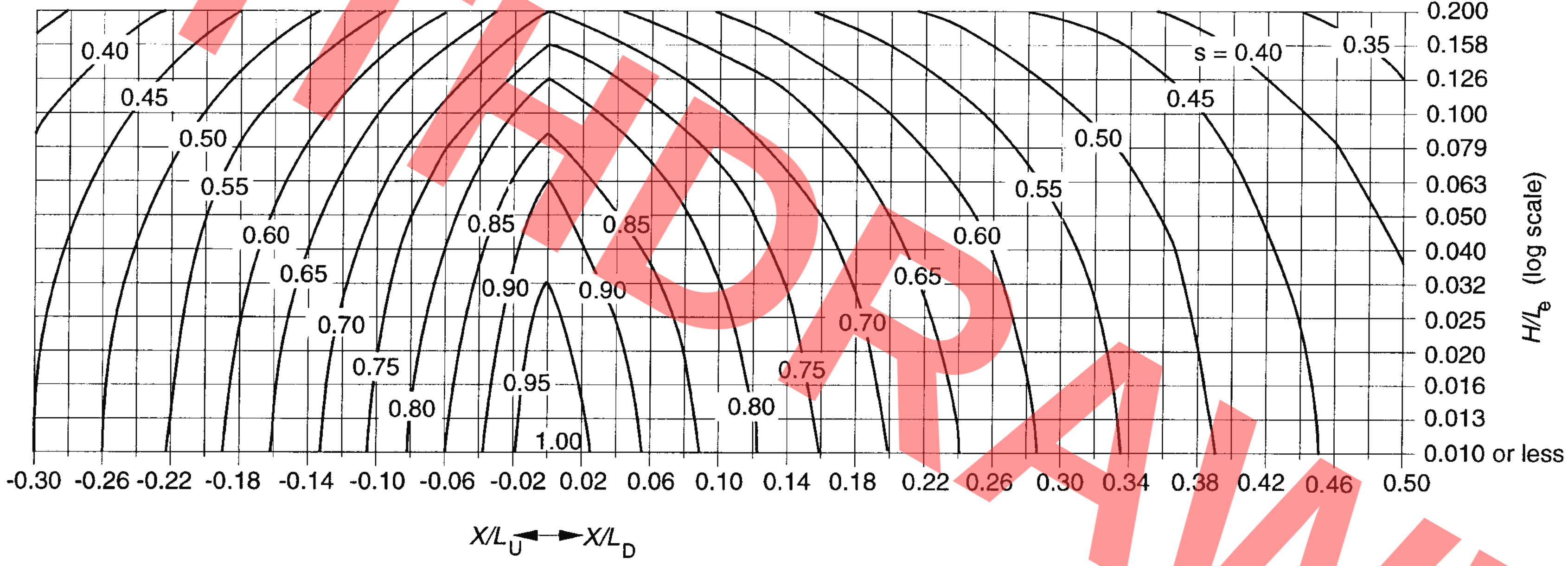

Figure 23b - Topographic location factors for hills and ridges

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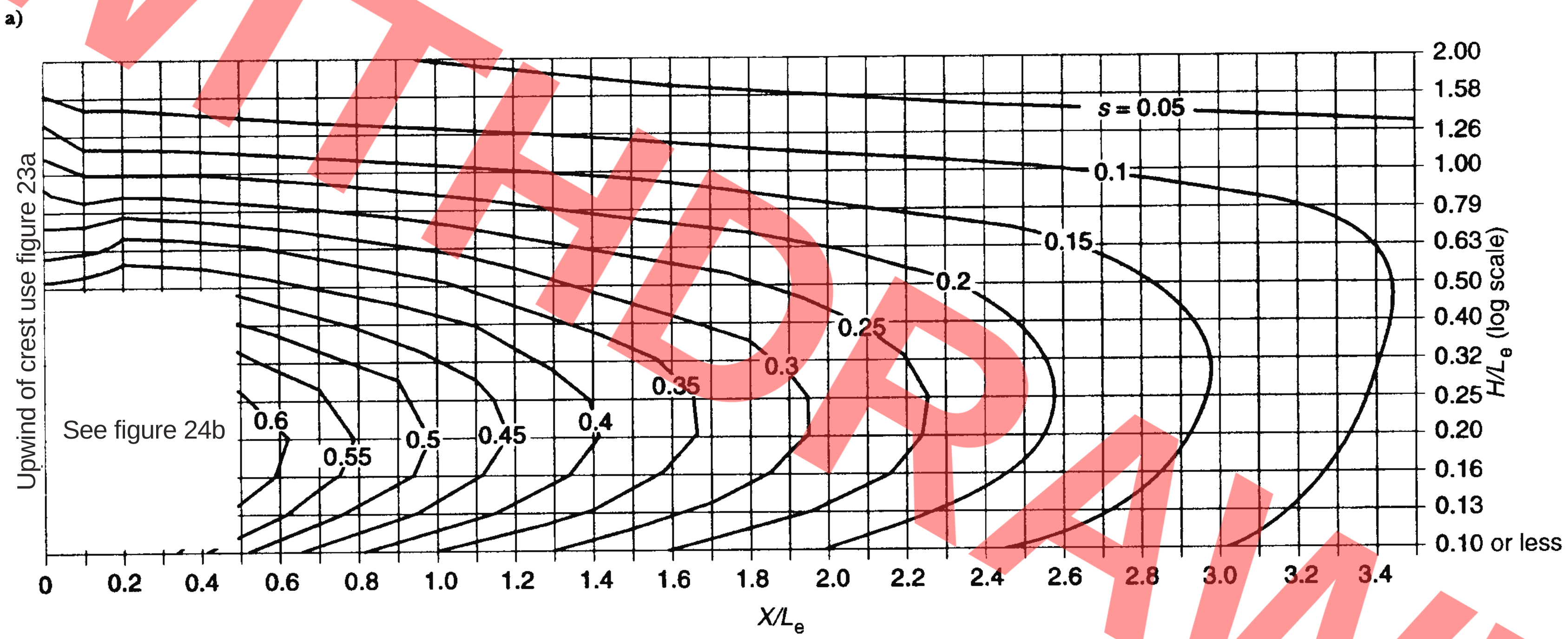

Figure 24a - Topographic location factors for cliffs and escarpments

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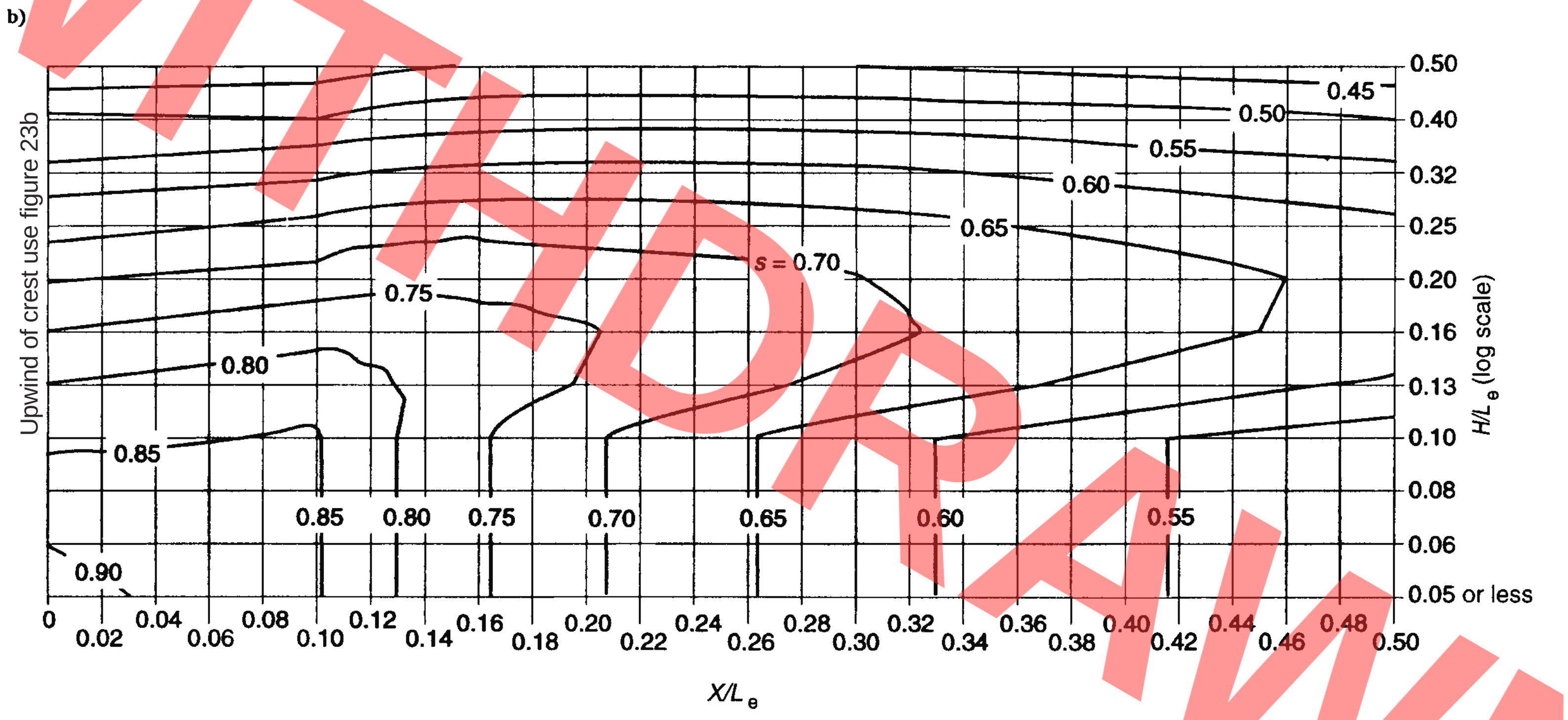

Figure 24b - Topographic location factors for cliffs and escarpments

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STANDARDS PUBLICATIONS REFERRED TO

BS 153	Steel girder bridges
BS 648	Schedule of weights of building materials
BS 6100	Glossary of building and civil engineering terms: Part 2: Civil Engineering
BS 8004	Foundations