BS 5400-2:
2006

Steel, concrete and composite bridges —

Part 2: Specification for loads

ICS 93.040

British Standards

Committees responsible for this British Standard

The preparation of this British Standard was entrusted by Technical Committee B/525, Building and civil engineering structures, to Subcommittee B/525/10, Bridges, upon which the following bodies were represented:

Association of Consulting Engineers
British Cement Association
British Construction Steelwork Association Ltd.
British Precast Concrete Federation Ltd.
Concrete Society
County Surveyors' Society
Department for Transport — Highways Agency
Institution of Civil Engineers
Institution of Structural Engineers
Network Rail
RSSB — Rail Safety and Standards Board
Steel Construction Institute
UK NTC
UK Steel Association
Welding Institute
Co-opted members

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Contents

Foreword

This British Standard has been prepared by Subcommittee B/525/10. It supersedes BS 5400-2:1978, which is now withdrawn.

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 for 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;*
- *Part 10C: Charts for classification of details for fatigue.*

BSI committee CSB 59/1 reviewed BS 5400-2:1978 (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. These changes were not incorporated into BS 5400-2:1978.

Since publication of Amendment No. 1, Subcommittee B/525/10 has identified several changes that are necessary to bring the loading requirements up to date, in addition to revision of the HA loading curve. For example, the new loading code for wind (BS 6399-2) has been published in the United Kingdom and further advances have been made in wind engineering.

This has led to the need to update 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 changes 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 changes have been made:

- — updating certain aspects of highway bridge loading:
  - — horizontal dynamic loading due to crowds on foot/cycle track bridges;
  - — vehicle collision loads on supports 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;
  - — load effects for bridges with more than two 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.

This new edition of BS 5400-2 does not reflect a full review or revision of the standard. Its primary purpose is to produce a standard including the revisions to BS 5400-2:1978 incorporated in Highways Agency standard BD 37/01, pending introduction of the Eurocodes. It incorporates only the changes listed above plus other editorial improvements.

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

Compliance with a British Standard does not of itself confer immunity from legal obligations.

Summary of pages

This document comprises a front cover, an inside front cover, pages i to v, a blank page, pages 1 to 88, an inside back cover and a back cover.

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1 Scope

1.1 General

This Standard specifies the loading to be used for the design of highway and railway bridges and associated structures.

Additional requirements for the design of highway structures are included in the Design Manual for Roads and Bridges, published by The Stationery Office Ltd. Any additional requirements are to be agreed with the relevant authority.

1.2 Documents comprising this British Standard

This specification for loads forms a part of a series of standards that deal with the design, materials and workmanship of steel, concrete and composite bridges (see Foreword).

1.3 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 for highway, railway and foot/cycle track bridges in the United Kingdom. Where different loading standards apply, modifications might be necessary.

1.4 Wind and temperature

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

2 Normative references

The following referenced documents are indispensable for the application of this document. For dated references, only the reference cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

BS 648:1964, *Schedule of weights of building materials.*

BS 5400-9.1:1983, *Steel, concrete and composite bridges — Part 1: Bridge bearings — Section 9.1: Code of practice for design of bridge bearings.*

BS 8004:1986, *Code of practice for foundations.*

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 Terms and definitions

For the purposes of this Part of BS 5400, the following terms and 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
stress resultants and deformations in the structure arising from its response to loads (as defined in **3.2.1**)

3.2.2
dead load
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, mains, ducts, miscellaneous furniture, etc.

3.2.3
superimposed dead load
weight of all materials forming loads on the structure that are not structural 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, e.g. lurching, nosing, centrifugal, longitudinal, skidding and collision loads

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
algebraic sum of the adverse and relieving effects

3.2.7
dispersal
spread of load through surfacing, fill, etc.

3.2.8
distribution
sharing of load between directly loaded members and other members not directly loaded as a consequence of the stiffness of intervening connecting members as, for example, 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 purpose 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 0.6 m or more than 1.0 m 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 marking or edge marking.

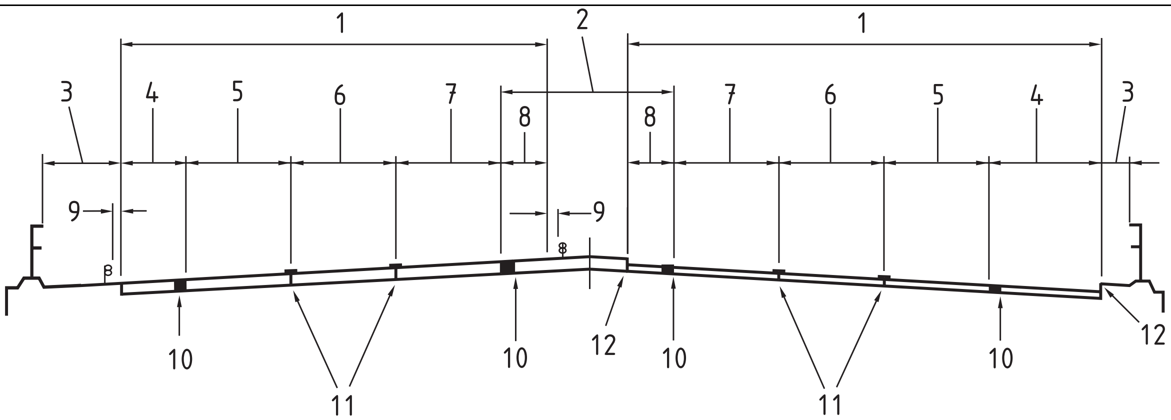
NOTE For ease of use, the definition of "carriageway" given in this Standard differs from that given in BS 6100-2.

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

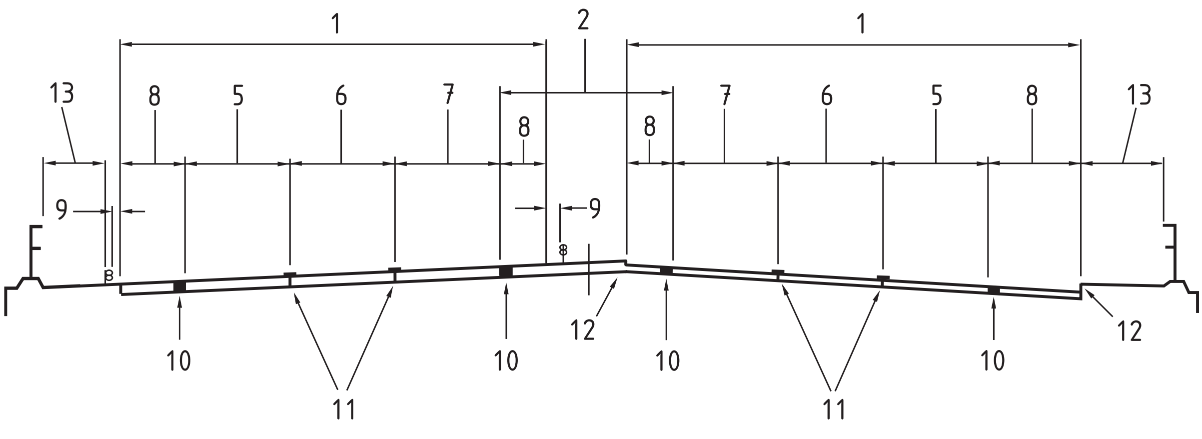
3.2.9.3
notional lanes
notional parts of the carriageway used solely for the purposes of applying the specified live loads, measured in a direction at right angles to the line of the raised kerbs, lane marking or edge marking

3.2.9.3.1
carriageway widths of 5.00 m or more
notional lanes shall be taken to be not less than 2.50 m wide. Where the number of notional lanes exceeds two, their individual widths shall be not more than 3.65 m. The carriageway shall be divided into an integral number of notional lanes having 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



i) Motorway



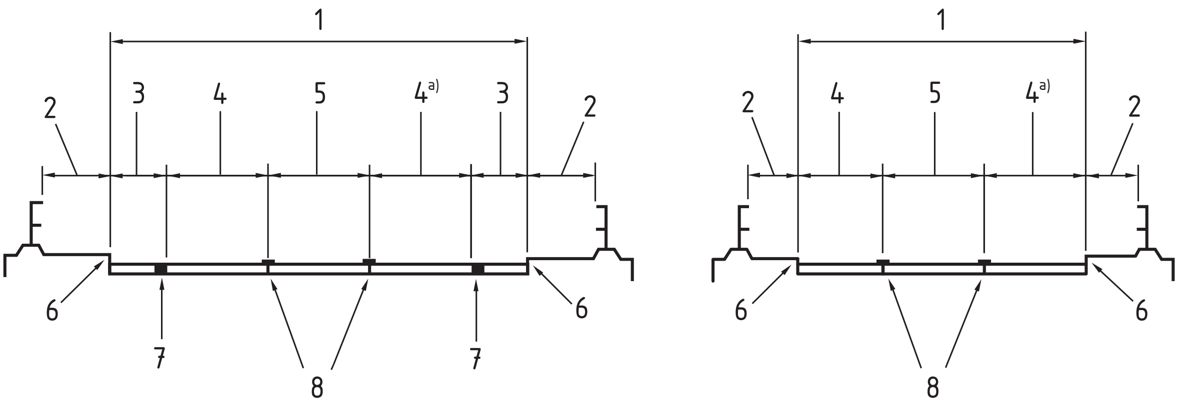
ii) All-purpose road

Key

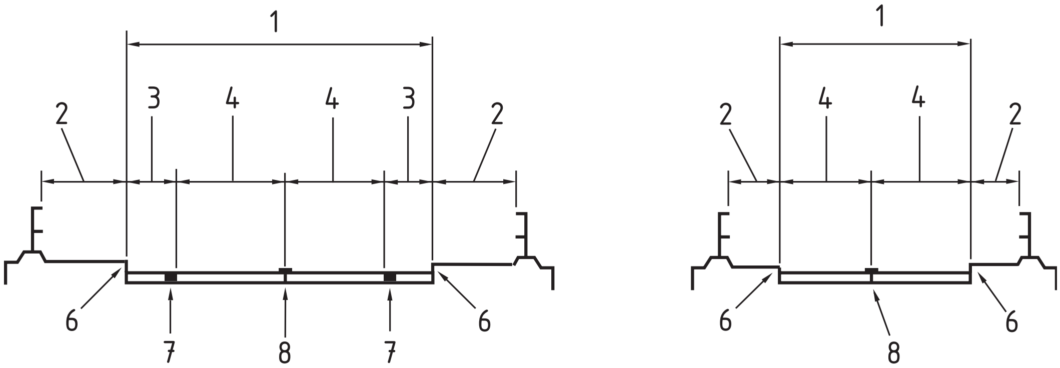
1 Carriageway for the purpose of **3.2.9**
2 Central reserve – will be split on separate superstructures
3 Verge
4 Hard shoulder
5 Inside traffic lane
6 Middle traffic lane
7 Outside traffic lane
8 Hard strip
9 Setback
10 Edge marking
11 Lane marks
12 Raised kerb
13 Footway

a) Superstructures: dual carriageway

Figure 1 — Highway carriageway and traffic lanes



i) Single 3-lane carriageway



ii) Single 2-lane carriageway

Key

1 Carriageway for the purpose of **3.2.9**	4 Inside traffic lane	7 Edge marking
2 Footway or verge	5 Middle traffic lane	8 Lane marking
3 Hard strip	6 Raised kerb	

[a] 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 1a).

b) Bridge carrying a single carriageway

Figure 1 — Highway carriageway and traffic lanes (*continued*)

3.2.9.3.2

carriageway widths of less than 5.00 m

The carriageway shall be taken to have one notional lane with a width of 2.50 m. The loading on the remainder of the carriageway width shall 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
in a bridge, 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 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 derive 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 Annex 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_a	altitude factor
S_b, S_b'	bridge and terrain factors
S_c, S_c'	hourly speed factors
S_d	direction factor
S_g	gust factor
S_h, S_h'	topography factors
S_m	hourly mean speed 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 Annex 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_{f1}, 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 ($\gamma_{f1} \times \gamma_{f2}$)
δ	logarithmic decrement of decay of vibration
Δ, Δ_s	altitude above mean sea level
Δ_T	base of topography

η	shielding factor
α	angle of wind (see **5.3.5**)
Ψ	dynamic response factor (see **B.2.6**)
Ψ	average slope of ground (see Figure 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 distributions 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 shall be multiplied by the approximate 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 shall also be multiplied by γ_{f3} to obtain the design load effects for design of concrete bridges conforming to Part 4 of this standard. For Parts 3 and 5 of this standard, the design load carrying capacity or stiffness shall be divided by γ_{f3}. 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 to meet the desired deflection, camber and drainage characteristics of the structure are given in Parts 3, 4 and 5 of this standard. Deformation requirements for railway bridges are specified in **8.8**.

4.2 Loads to be considered

The loads to be considered in different load combinations, together with the specified values of γ_{fL}, are set out in the appropriate clauses and summarized 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 of 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.

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

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

[a] 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.
[b] γ_{fL} may be reduced to 1.2 and 1.0 for the ULS and SLS respectively, subject to approval of the relevant authority (see **5.2.2.1**).
[c] Accidental wheel loading shall not be considered as acting with any other primary live loads.

ULS: ultimate limit state SLS: serviceability limit state

Table 1 — Loads to be taken in each combination with appropriate γ_{fL} (*continued*)

Clause number	Load		Limit state	γ_{fL} to be considered in combination				
				1	2	3	4	5
Note to loads under 6.7.1, 6.7.2, 6.8, 6.9, 6.10, 6.11: Each secondary live load shall be considered separately together with other combination 4 loads as appropriate								
6.7.1	Loads due to vehicle collision with parapets and associated primary live load:	Local effects: parapet load						
		low and normal containment	ULS				1.50	
			SLS				1.20	
		high containment	ULS				1.40	
			SLS				1.15	
		associated primary live load:	ULS				1.30	
		low, normal & high containment	SLS				1.10	
6.7.2		Global effects: parapet load						
		1. Massive structures						
		a) Bridge superstructures and non-elastomeric bearings	ULS				1.25	
		b) Bridge substructures and wing and retaining walls	ULS				1.00	
		c) Elastomeric bearings	SLS				1.00	
		2. Light structures:						
		a) Bridge superstructures and non-elastomeric bearings	ULS				1.40	
		b) Bridge substructures and wind and retaining walls	ULS				1.40	
		c) Elastomeric bearings	SLS				1.00	
		Associated primary live load: Massive and light structures:						
		a) Bridge superstructures, non-elastomeric bearings, bridge substructures and wing and retaining walls	ULS				1.25	
		b) Elastomeric bearings	SLS				1.00	
6.8 **8.6**	Vehicle collision loads on bridge supports and superstructures:	Effects on all elements excepting elastomeric bearings	ULS				1.50	
		Effects on elastomeric bearings	SLS				1.00	
6.9	Centrifugal load and associated primary live load		ULS				1.50	
			SLS				1.00	
6.10	Longitudinal load:	HA and associated primary live load	ULS				1.25	
			SLS				1.00	
		HB associated primary live load	ULS				1.10	
			SLS				1.00	
6.11	Accidental skidding load and associated primary live load		ULS				1.25	
			SLS				1.00	
7	Foot/cycle track bridges:	live load and effects due to parapet load	ULS	1.50	1.25	1.25		
			SLS	1.00	1.00	1.00		
		vehicle collision loads on supports and superstructures[d]	ULS				1.50	
8	Railway bridges:	type RU (including SW/0) and RL, primary and secondary live loading	ULS	1.40	1.20	1.20		
			SLS	1.10	1.00	1.00		

[d] 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.

4.3.2 ***Transient loads***

For the purposes of this standard, all loads other than permanent loads shall be considered to be 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 Combination 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 summarized 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***

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**).

Combination 4 does not apply to railway bridges except for vehicle collision loading on bridge supports and superstructures (see Table 1 and **8.6**).

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[1].

4.5 Application of loads

Each element and structure shall be examined under the effects of loads that can co-exist in each combination.

4.5.1 ***Selection to cause most adverse effect***[2]

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 can 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.

[1] Where a member is required to resist the loads due to temperature restraint within the structure and to resist 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.

[2] It is expected that experience in the use of this standard will enable users to identify those load cases and combinations which govern design provisions, and it is only those load cases and combinations which need to be established for use in practice.

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 and **5.3.2.6**.

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 (i.e. γ_{fL} for the ultimate limit state multiplied by the effects of the nominal loads).

4.6.2 ***Removal of loads***

The requirements specified in **4.5.2** relating to the possible 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 specified in this standard.

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.

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 account, 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 specified 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, e.g. 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

except as specified in **5.2.2.1** and **5.2.2.2** (Note also the requirements of **4.5.2**).

NOTE 1 The term "deck surfacing" includes carriageway surfacing and track ballast. The maximum thickness of carriageway surfacing and track ballast to be considered as deck surfacing shall be in accordance with the requirements of the relevant authority.

NOTE 2 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 relevant 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 specified 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 200 m span and for footbridges up to 30 m 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 20 m or less in span, 10 m 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 value 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 calculated 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 that 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 10 m 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 10 m above sea level.

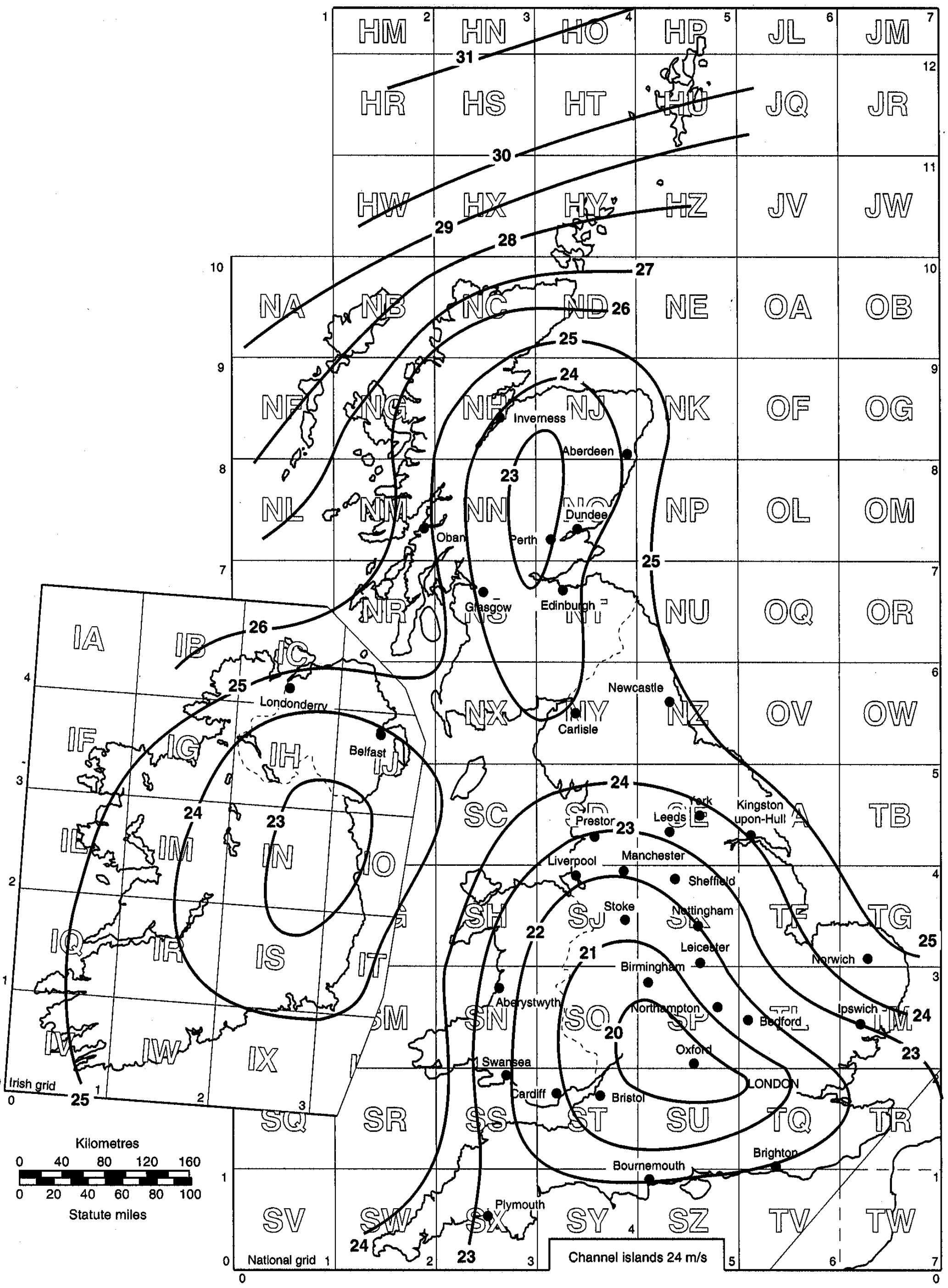


Figure 2 — Basic wind speed V_b in m/s (copyright BRE)

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 relevant 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 Annex E. Where a particular erection will be completed in a short period, S_p shall be combined with a seasonal factor also obtained from Annex E.

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 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 $\phi = 0°$ to $\phi = 330°$ in 30° intervals. (The wind direction is defined in the conventional manner: an east wind is a wind direction of $\phi = 90°$ 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	0°	0.78
	30°	0.73
	60°	0.73
E	90°	0.74
	120°	0.73
	150°	0.80
S	180°	0.85
	210°	0.93
	240°	1.00
W	270°	0.99
	300°	0.91
	330°	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 1 km in the wind direction providing the nearest edge of water is closer than 1 km 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 = 5 m 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 3 km 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 Annex 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 Vr 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

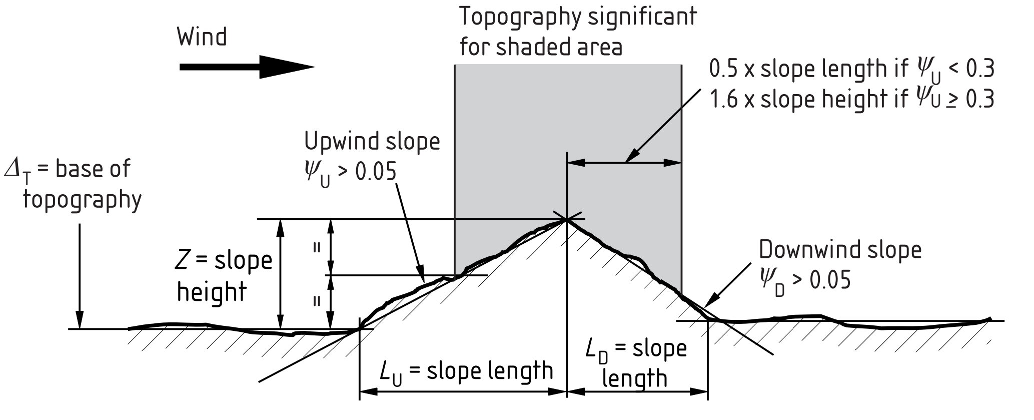
$S_c = S_c' K_F$

S_c' is the hourly speed factor (see **5.3.2.4.1**)

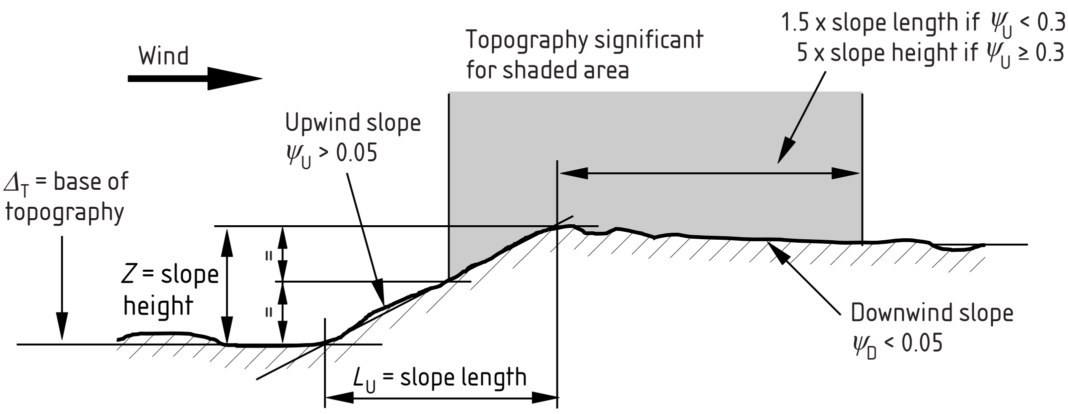
K_F is the fetch correction factor (see **5.3.2.3.1**)

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 F.1).



a) Hill and ridge (upwind slope > 0.05; downwind slope > 0.05)



b) Escarpment (0.3 > upwind slope > 0.05; downwind < 0.05) and cliff (upwind > 0.3; downwind < 0.05)

L_D Length of downwind slope

L_U Length of upwind slope

Z Slope height

Ψ_D Downwind slope Z/L_D

Ψ_U Upwind slope Z/L_U

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
For notes, see after Table 5.							

Table 3 — Values of terrain and bridge factor S_b', hourly speed factor S_c' and fetch correction factor, K_F (*continued*)

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
For notes, see after Table 5.						

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		
For notes, see after Table 5.			

Table 5 — Hourly mean reduction factor T_c for bridges in towns

Height above ground m	Distance from edge of town in upwind direction 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 Table 3, Table 4 and Table 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 Table 3, Table 4 and Table 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 3 km 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:

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

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

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:

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

$0.613V_r^2$ 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**, **5.3.3.4**, **5.3.3.5** and **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.

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 1 m, 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 1 m, 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 1.25 m above the footway) and the parts of the windward parapet more than 1.25 m 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 fances*

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**, **5.3.3.2.2**, **5.3.3.2.3**, **5.3.3.2.4** and **5.3.3.2.5** requirements are specified for discrete beams or girders before deck construction or other infilling (e.g. 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 combined structure shall be taken as 1.5 × 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 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*

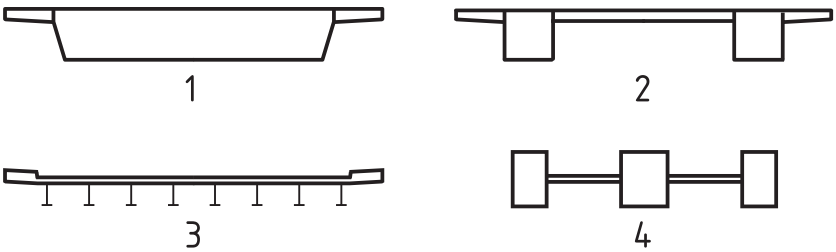
C_D for each girder shall be taken as 2.2 without any allowance for shielding. Where the combined girders are required to be considered, C_D for the combined structure shall be taken as $2(1 + c/20d)$, but not more than 4, where *c* is the distance centre to centre of adjacent girders, and *d* is the 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 Figure 4b). See also Notes 5 and 6 of Figure 5.



Key

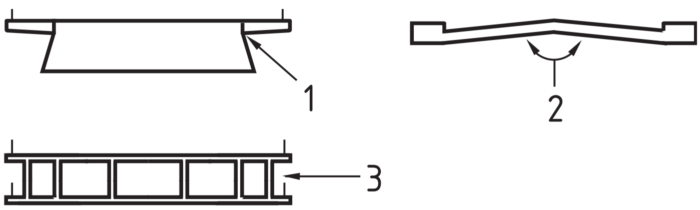
1 Single box or slab — sloping or vertical sides

2 Twin or multiple boxes — sloping or vertical sides

3 Multiple beams or girders

4 Through bridges — box or plate girders — deck at any position vertically

a) Typical superstructures to which Figure 5 applies



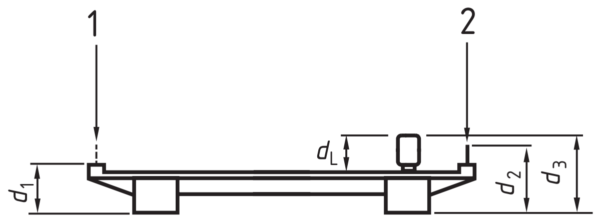
Key

1 Re-entrant angle

2 Re-entrant angle < 175° in soffit of slab

3 Open section on windward face

b) Typical superstructures that require wind tunnel tests



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

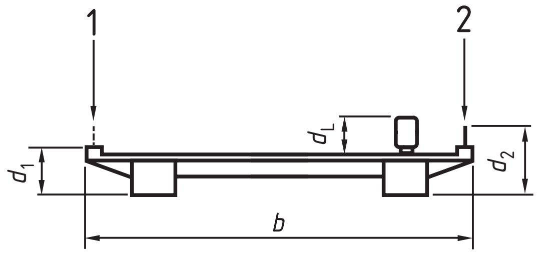
Key

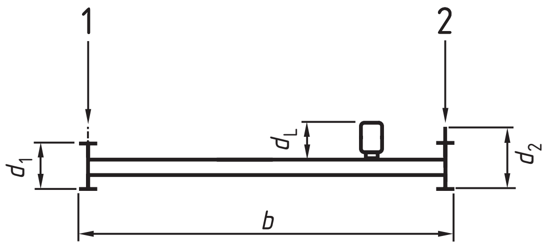
1 Open parapet

2 Solid parapet

c) Depth d to be used in deriving A_1

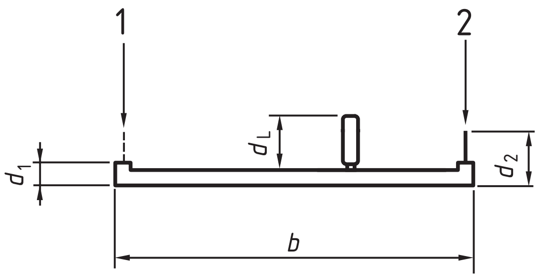
Figure 4 — Typical superstructures to which Figure 5 applies; those that require wind tunnel tests and depth d to be used for deriving A_1 and C_D

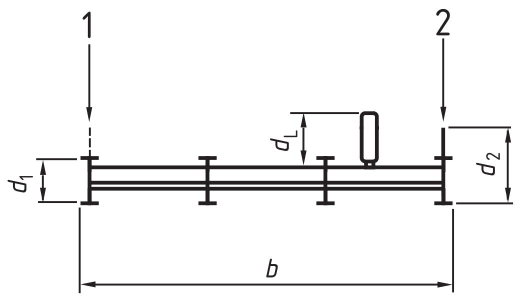




Parapet	Superstructures without live load	Superstructures with live load
Open	$d = d_1$	$d = d_L$
Solid	$d = d_2$	$d = d_L$

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





Parapet	Superstructures without live load	Superstructures with live load
Open	$d = d_1$	$d = d$
Solid	$d = d_2$	$d = d_L$

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

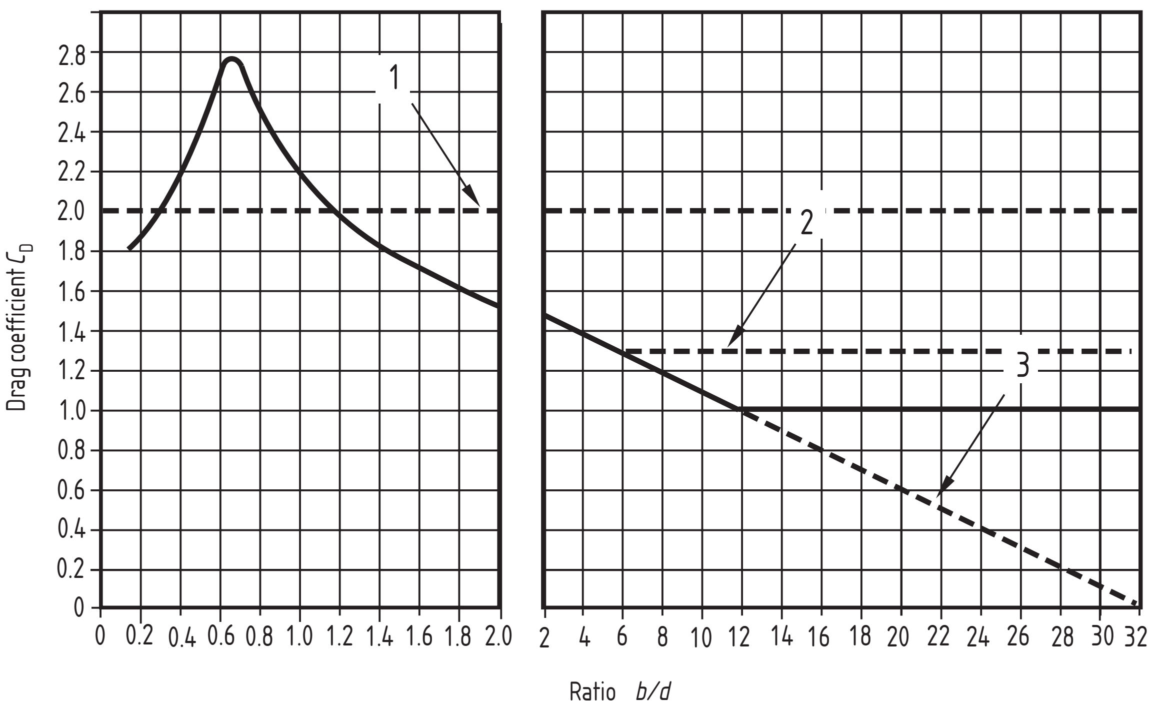
Key

1 Open parapet

2 Solid parapet

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 (*continued*)



Key

1 Minimum co-efficient for foot/cycle track bridges

2 Minimum co-efficient for decks supported by I sections or by more than 4 beams or box girders

3 See Note 7

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 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 (i.e. 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 a 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 1 m, 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 for 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 derived C_D. The load on the leeward structure is generally opposite in sign to that of the windward superstructure.

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

Figure 5 — Drag coefficient C_D for superstructures with solid elevation

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 < 6 m^2/s	$dV_d \geq 6\ m^2/s$
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 trusses 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 windload 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**, **5.3.4.2**, and **5.3.4.3**.

Table 8 — Drag coefficient C_D for parapets and safety fences

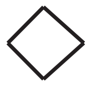
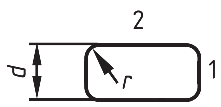
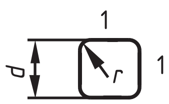
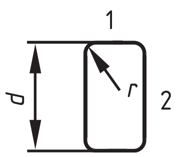
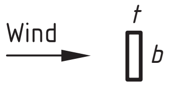
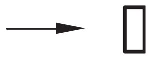
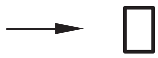
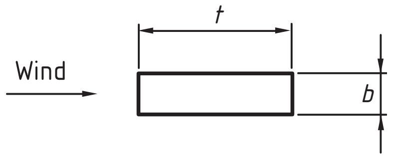
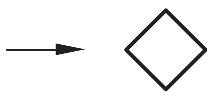
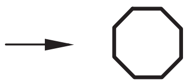
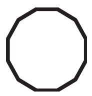
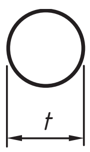
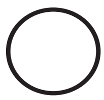
 (where V_d in m/s and d in m) NOTE On relieving areas use V_r instead of V_d	Circular sections	$dV_d < 6$ $dV_d \geq 6$	1.2 0.7
	Flat members with rectangular corners, crash barrier rails and solid parapets		2.2
	Square members diagonal to wind		1.5
	Circular stranded cables		1.2
	Rectangular members with circular corners $r > d/12$		1.1[a]
	Square members with circular corners $r > d/12$		1.5[a]
	Rectangular members with circular corners $r > d/24$		2.1[a]
[a] For sections with intermediate proportions, C_D may be obtained by interpolation.			

Table 9 — Drag coefficient C_D for piers

Plan shape	$\frac{t}{b}$	C_D for pier height/breadth ratios of						
		1	2	4	6	10	20	40
	≤ ¼	1.3	1.4	1.5	1.6	1.7	1.9	2.1
	⅓ to ½	1.3	1.4	1.5	1.6	1.8	2.0	2.2
	⅔	1.3	1.4	1.5	1.6	1.8	2.0	2.2
	1	1.2	1.3	1.4	1.5	1.6	1.8	2.0
	1½	1.0	1.1	1.2	1.3	1.4	1.5	1.7
	2	0.8	0.9	1.0	1.1	1.2	1.3	1.4
	3	0.8	0.8	0.8	0.9	0.9	1.0	1.2
	≥ 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 $tV_d \geq 6\ m^2/s$		0.5	0.5	0.5	0.5	0.5	0.6	0.6
 Circle with smooth surface where $tV_d < 6\ m^2/s$. 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 $1 - \frac{1.5r}{b}$ 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 Table 3, Table 4 and Table 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), ηC_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.

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

$$P_v = qA_3C_L$$

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:

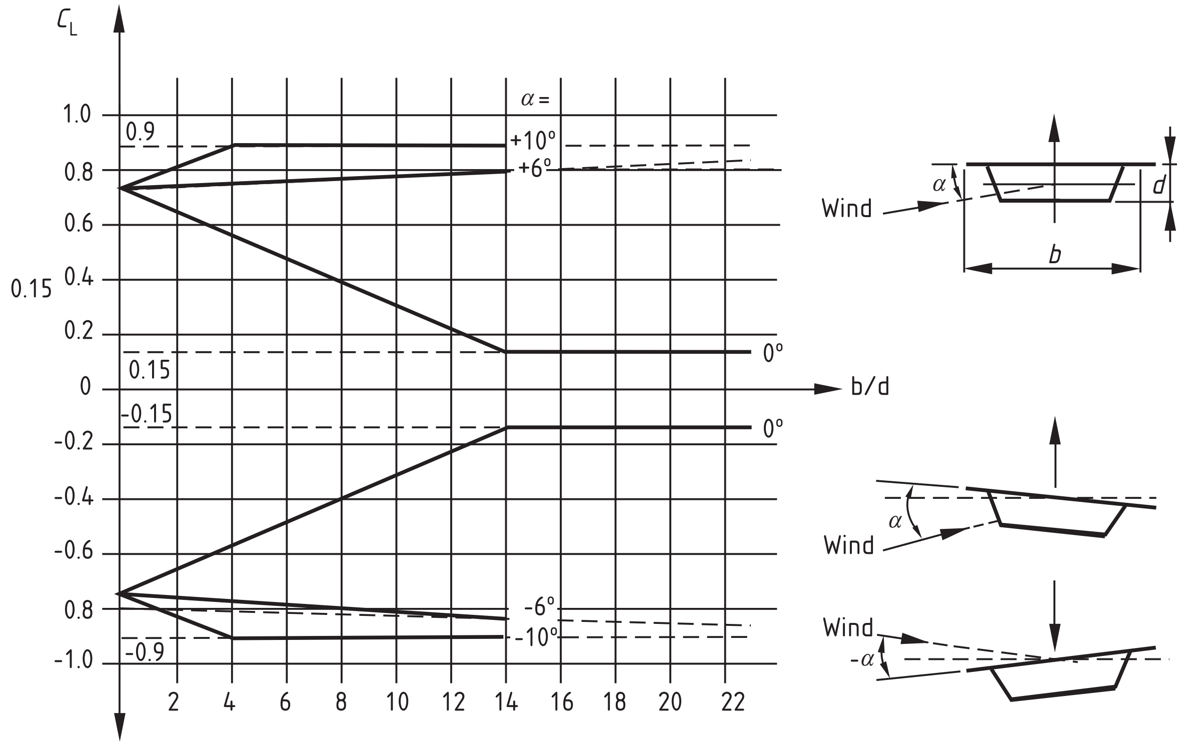
$$C_L = 0.75\left[1 - \frac{b}{20d}(1 - 0.2\alpha)\right]$$

but $0.15 < C_L < 0.90$

where

α is the sum of the angle of superelevation (in degrees) and the wind inclination to be considered (taken as a positive number in the above equation, irrespective of the inclination and superelevation) as given in Figure 6.

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



NOTE This figure applies to bridge types shown in Figure 4a).

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 6 kN/m of bridge;

— on railway bridges, not more than 12 kN/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 state and serviceability limit state 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 standard or as agreed with the relevant authority.

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 Annex 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 (e.g. portal frame, arch, flexible pier, elastomeric bearings) referred to as temperature restraint; and

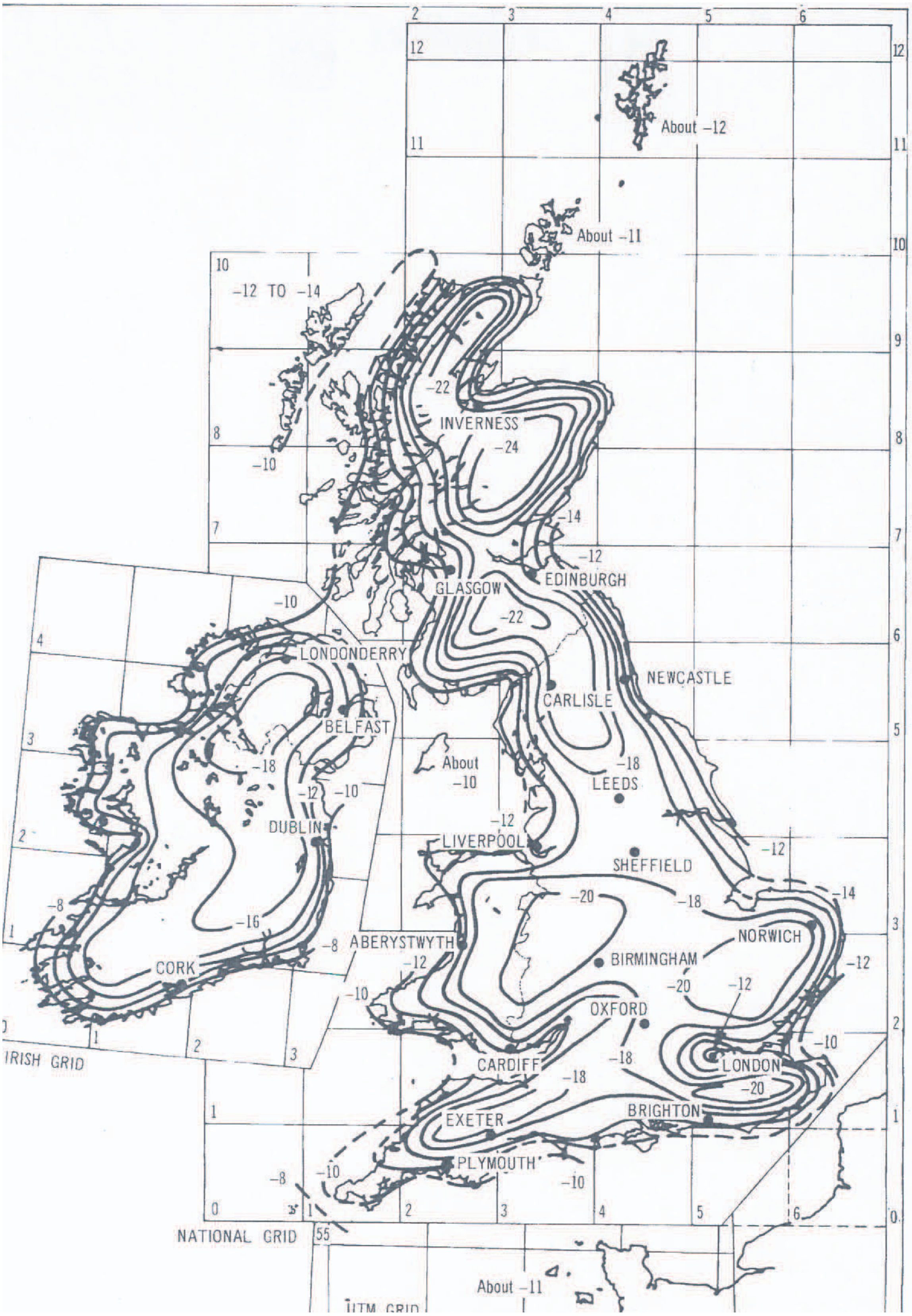
2) friction at roller or sliding bearings where the form of the structure permits associated expansion and contraction, referred to as frictional bearing restraint.

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 Figure 7 and Figure 8. These shade air temperatures are appropriate to mean sea level in open country and a 120-year return period.

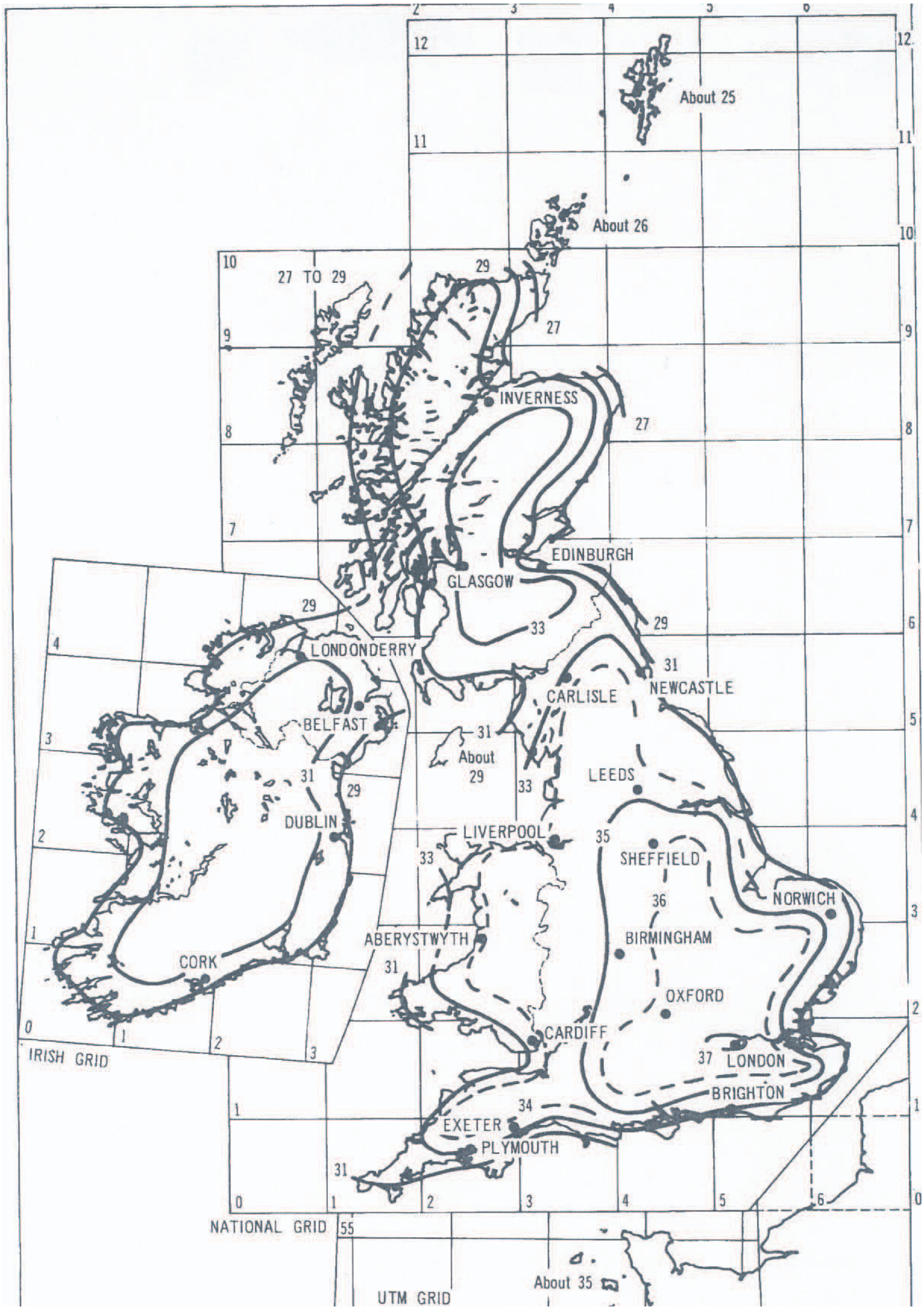
NOTE 1 It is only possible to relate the effective bridge temperature to the shade air temperature during a period of extreme environmental conditions.

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.



NOTE The isotherms are derived from Meteorological Office data.

Figure 7 — Isotherm 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)

5.4.2 ***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 Figure 7 and Figure 8.

For foot/cycle track bridges, subject to the agreement of the relevant 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 100 m height for minimum shade air temperatures and 1.0 °C per 100 m 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 maxcimum 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 Table 10 and Table 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 Table 10 and Table 11 assume depths of 40 mm for groups 1 and 2 and 100 mm 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.

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.

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

Table 12 — Adjustment to effective bridge temperature for deck surfacing

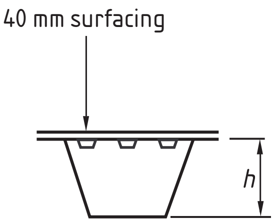
	Addition to minimum effective bridge temperature				Addition to maximum effective bridge temperature			
Deck surface	Group 1	Group 2	Group 3	Group 4	Group 1	Group 2	Group 3	Group 4
	°C	°C	°C	°C	°C	°C	°C	°C
Unsurfaced	0	0	–3	–1	+4	+2	0	0
Waterproofed[a]	0	0	–3	–1	+4	+2	+4	+2
40 mm surfacing[b]	0	0	–2	–1	0	0	+2	+1
100 mm surfacing[b]	N/A	N/A	0	0	N/A	N/A	0	0
200 mm surfacing[b]	N/A	N/A	+3	+1	N/A	N/A	–4	–2

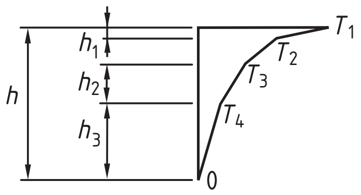
[a] Waterproofed deck values are conservative, assuming dark material; there may be some alleviations when light coloured waterproofing is used. Specialist advice should be sought if required.

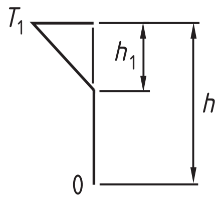
[b] Surfacing depths include waterproofing.

Group	Type of construction	Temperature difference (°C)	
		Positive temperature difference	**Reverse temperature differences**

1. Steel deck on steel box girders



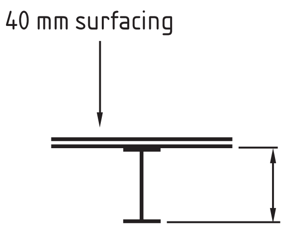


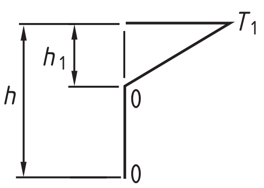


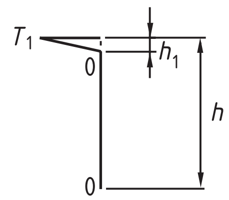
h_1 = 0.1 m T_1 = 24 °C
h_2 = 0.2 m T_2 = 14 °C
h_3 = 0.3 m T_3 = 8 °C
T_4 = 4 °C

T_1 = 6 °C h_1 = 0.5 m

2. Steel deck on steel truss or plate girders



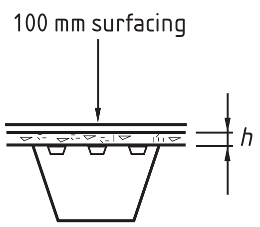


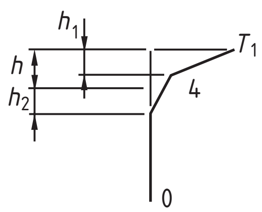


h_1 = 0.5 m T_1 = 21 °C

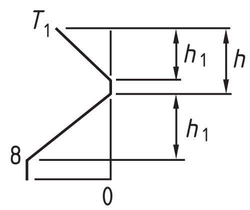
T_1 = 5 m h_1 = 0.1 m

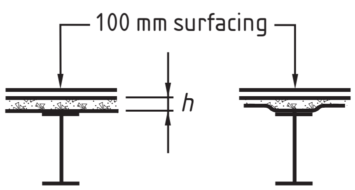
3. Concrete deck on steel box, truss or plate girders





$h_1 = 0.6h$
h_2 = 0.4 m





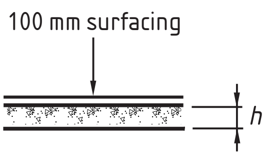
h	***T*₁**
m	°C
0.2	13
0.3	16

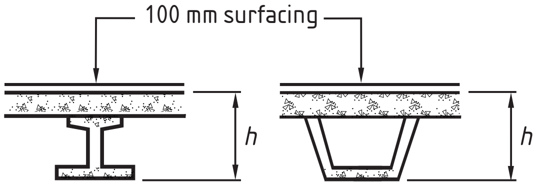
h	***T*₁**
m	°C
0.2	3.5
0.3	5.0

Figure 9 — Temperature difference for different types of construction

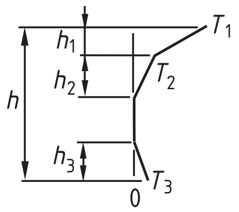
Group	Type of construction	Temperature difference (°C)	
		Positive temperature difference	Reverse temperature difference

4. **Concrete slab or concrete deck on concrete beams or box girders**





Positive temperature difference



$h_1 = 0.3\,h$ but ≤ 0.15 m

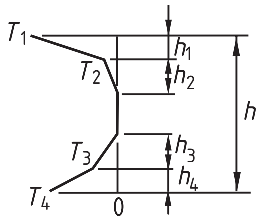
$h_2 = 0.3\,h$ but ≥ 0.10 m but ≤ 0.25 m

$h_3 = 0.3\,h$ but $\leq$ (0.1 m + surfacing depth in metres)

(for thin slabs, h_3 is limited by $h - h_1 - h_2$)

h m	***T***$_1$ °C	***T***$_2$ °C	***T***$_3$ °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

Reverse temperature difference



$h_1 = h_4 = 0.20\,h$ but ≤ 0.25 m

$h_2 = h_3 = 0.25\,h$ but ≤ 0.20 m

h m	***T***$_1$ °C	***T***$_2$ °C	***T***$_3$ °C	***T***$_4$ °C
≤ 0.2	2.0	0.5	0.5	1.5
0.4	4.4	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 (*continued*)

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 40 mm for groups 1 and 2 and 100 m for groups 3 and 4. For other depths of surfacing, different values will apply. Values for other thicknesses of surfacing are given in Annex 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 difference 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 Annex 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** and **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 5400-9.1:1983.

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 5400-9.1:1983.

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 needs to be provided for.

5.4.8.2 *Design load for temperature restraint*

For combination 3, γ_{fL} shall be taken as follows:

a) for the ultimate limit state: 1.30;

b) for the serviceability limit state: 1.00.

5.4.8.3 *Design load for frictional bearing restraint*

For combination 5, γ_{fL} shall be taken as follows:

a) for the ultimate limit state: 1.30;

b) for the serviceability limit state: 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:

a) for the ultimate limit state: 1.00;

b) for the serviceability limit state: 0.80.

5.5 Effects of shrinkage and creep, residual stresses, etc.

Where it is necessary to take into account the effects of shrinkage and 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 different 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 relevant 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:

a) For the ultimate limit state: 1.20;

b) For the serviceability limit state: 1.00.

5.7 Exceptional loads

Where other loads not specified in this standard are likely to be encountered, e.g. the effects of abnormal indivisible live loads (see Annex A), ship impact, 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 relevant authority.

5.7.1 ***Snow load***

Snow loading should be considered in accordance with local conditions; for those prevailing in Great Britain, this loading may generally be ignored in combinations 1 to 4 (see **4.4.1**, **4.4.2**, **4.4.3** and **4.4.4**), but there are circumstances, e.g. for opening bridges, covered 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 relevant 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 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 for 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 of 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: 10 kN/m^2;

b) for HB loading:

45 units: 20 kN/m^2

30 units: 12 kN/m^2

(intermediate values 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**, **5.9.2**, **5.9.3**, **5.9.4** and **5.9.5**.

For the serviceability limit state, nothing shall be done during erection that could cause damage to the permanent structure or 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/m^2 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 Annex 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 lengths are specified in **6.4.1**.

6.1.3 ***Distribution analysis of structure***

he 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 relevant 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 50 m 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 50 m but less than 1 600 m, the UDL shall be derived from the equation

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

where

L is the load length (in m)

W is the load per metre of notional lane (in kN).

For loaded lengths above 1 600 m, the UDL shall be agreed with the relevant 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.

6.2.2 ***Nominal knife edge load (KEL)***

The KEL per notional lane shall be taken as 120 kN.

6.2.3 ***Distributuion***

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**.

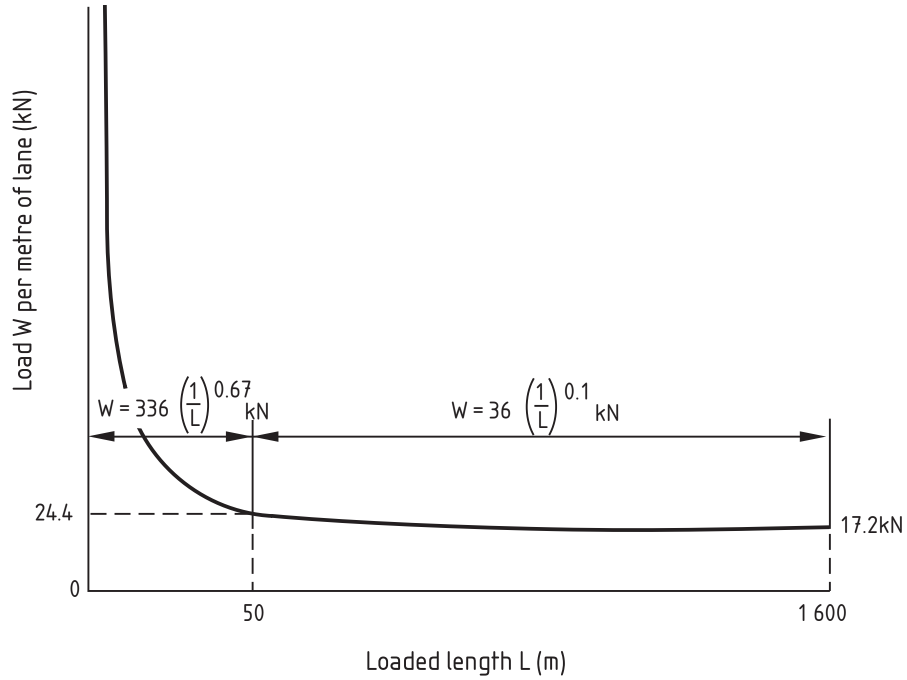


Figure 10 — Loading curve HA UDL (not to scale)

Table 13 — Type HA uniformly distributed load

Loaded length m	Load kN/m	Loaded length m	Load kN/m	Loaded length m	Load 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	1 050	18.0
35	31.0	190	21.3	1 130	17.8
38	29.4	220	21.0	1 210	17.7
41	27.9	250	20.7	1 300	17.6
44	26.6	280	20.5	1 400	17.4
47	25.5	310	20.3	1 500	17.3
50	24.4	340	20.1	1 600	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).

6.2.4 *Dispersal*

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

6.2.5 *Single nominal wheel load alternative to UDL and KEL*

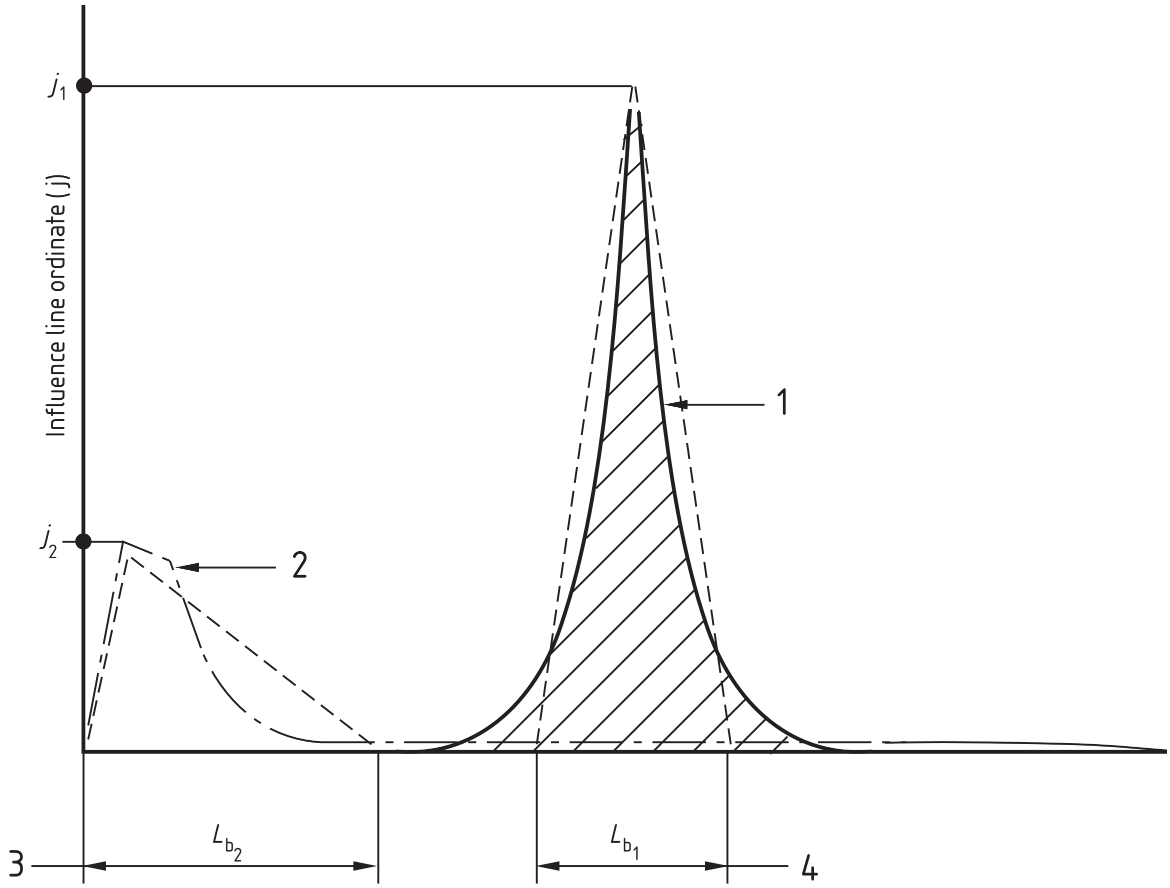
One 100 kN wheel, placed on the carriageway and uniform distribution over a circular contact area assuming an effective pressure of 1.1 N/mm^2 (i.e. 340 mm diameter), shall be considered.

Alternatively, a square contact area may be assumed, using the same effective pressure (i.e. 300 mm 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.



Key

1 Influence line IL1

2 Influence line IL2

3 Base length for IL2

4 Base length for IL1

Evaluation of base lengths

Consider influence line 1: Area under IL1 = U_1 (shaded);

Value of maximum ordinate = j_1:

Effective base length $L_{b1} = \dfrac{2U_1}{j_1}$

A similar evaluation applies for influence line 2.

Figure 11 — Base lengths for highly cusped influence lines

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 and 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.

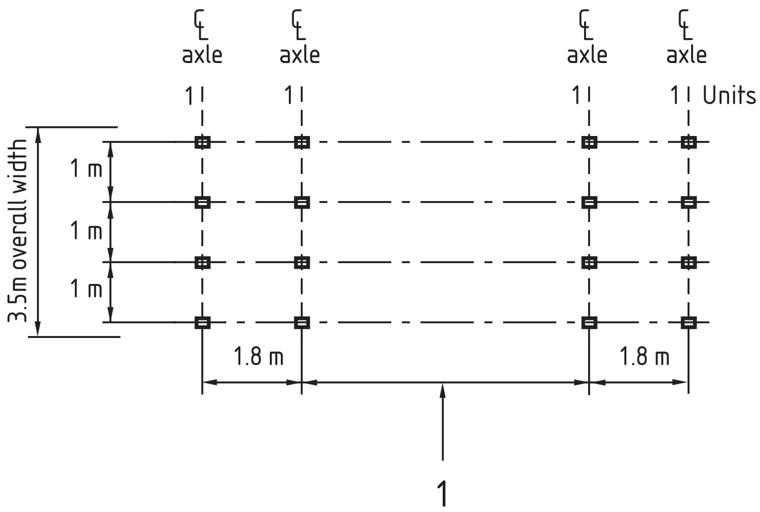
6.3 Type HB loading

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 this number may be increased up to 45 if so directed by the relevant 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 (i.e. 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 3.5 m. The longitudinal axis of the HB vehicle shall be taken as parallel with the lane markings.



Key

1 Inner axle spacing — 6, 11, 16, 21 or 26 m, whichever dimension produces the most severe effect on the member under consideration.

Figure 12 — Dimensions of HB vehicle

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.

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 combinations 1	1.30	1.10
For combinations 2 and 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[3]. 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 relevant 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 2.50 m. The loading on the remainder of the carriageway width shall be taken as 5 kN/m^2.

6.4.1.2 *Multilevel structures*

Where multilevel superstructures are carried on common substructure members (as, e.g. 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, i.e. as elements in themselves and also as parts of the main structure (e.g. 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**.

[3] 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 and subsequent lane factor β_n
$0 < L \leq 20$	α_1	α_1	0.6	$0.6\alpha_1$
$20 < L \leq 40$	α_2	α_2	0.6	$0.6\alpha_2$
$40 < L \leq 50$	1.0	1.0	0.6	0.6
$50 < L \leq 112$ $N < 6$	1.0	$7.1/\sqrt{L}$	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 $\alpha_1 = 0.274b_L$ 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 50 m. The value of N shall 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.

6.4.1.5 *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 be otherwise 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 (e.g. 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 m (e.g. 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 m in front of the leading axle to 25 m 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 m clear spaces.

Where the HB vehicle lies partially within the 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 m (e.g. 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 m 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 HB 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, 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[4]. 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*

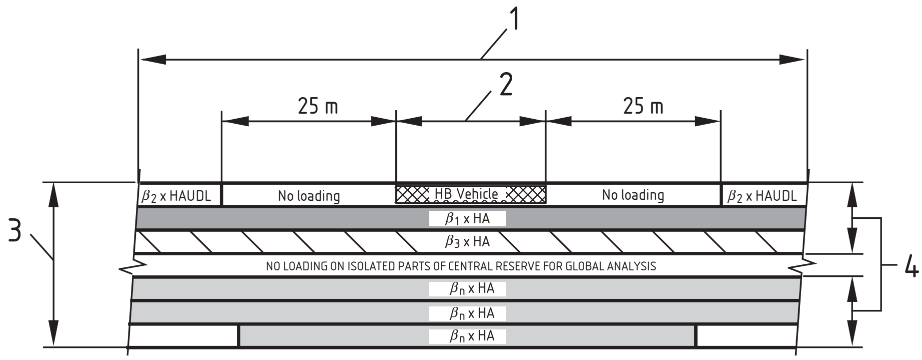
On dual carriageways the portion of the central reserve isolated from the rest of the carriageway either by a raised kerb or by safety fences is not required to be loaded with live load in considering the overall design of the structure, but it shall be capable of supporting 30 units of HB loading.

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 (footway and cycle track together) has a width exceeding two metres.

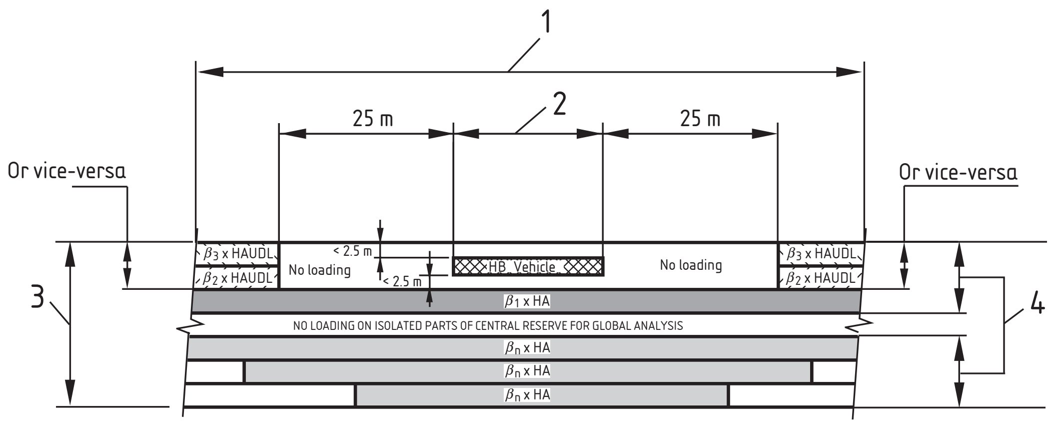
[4] This is the only exception to the rule that not more than one HB vehicle shall be considered to act on a structure. The 30 unit vehicle is to be regarded as a substitute for HA loading for these elements only.

1) HB vehicle within one notional lane

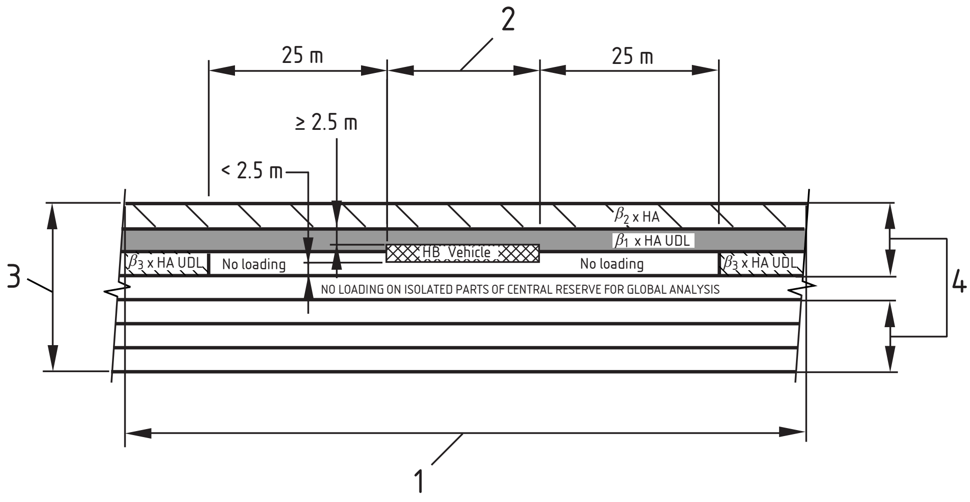


2) HB vehicle straddling two notional lanes

a)



b)



Key

1 Loaded length for intensity of HA UDl in lane containing HB vehicle

2 Overall vehicle length for axle spacing having most severe effect

3 Lane loadings are interchangeable for most severe effect

4 Notional lanes in each carriageway

NOTE 1 See **6.4.1.1** for the value of the HA lane factor (β) to be taken for each lane.

NOTE 2 The overall length and width of the HB vehicle shall be as specified in **6.3.1**.

NOTE 3 Unless otherwise stated, type HA loading includes both uniformly distributed loading (UDL) and knife edge loading (KEL).

NOTE 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 36 m, $k \times 5.0$ kN/m^2 where k is the

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

where L is the loaded length (in m).

Where the footway (or footway and cycle track together) has a width exceeding 2 m these intensities may be further reduced by 15 % on the first metre in excess of 2 m and by 30 % on any additional width in excess of 3 m. 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 36 m where exceptional crowds may be expected. Such loading shall be agreed with the relevant 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 400 m or where crowd loading is expected.

Where the footway (or footway and cycle track together) has a width exceeding 2 m these intensities may be further reduced by 15 % on the first metre in excess of 2 m and by 30 % on any additional width in excess of 3 m. These intensities may be averaged and applied as a uniform intensity over the full width of the 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 400 m, special consideration shall be given to the intensity of pedestrian live loading to be adopted. This shall be agreed with the relevant 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	1.10
For combinations 2 and 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 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, or such other loading as may be required by the relevant authority.

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 wheels 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 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

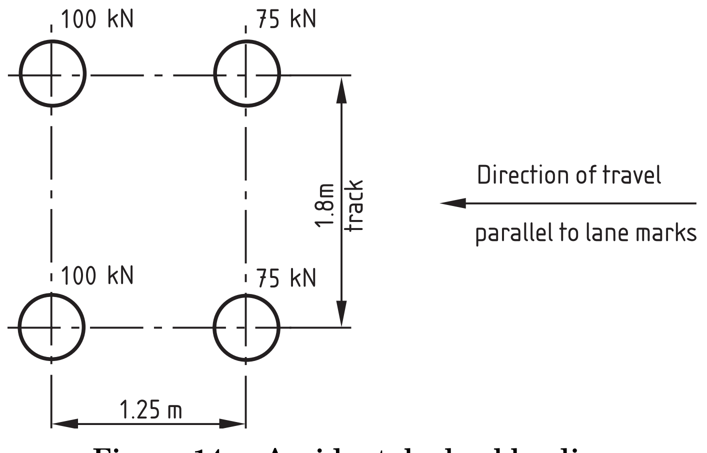


Figure 14 — Accidental wheel loading

6.7 Loads due to vehicle collision with parapets[5)]

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 4.5 m length of parapet at the parapet base applied uniformly over any 4.5 m length of supporting element.

For metal parapet (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 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 a length of 3 m 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.

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

5) 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.

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 shall 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 ultimate limit state	
On bridge superstructures and non-elastomeric bearings	1.25	1.4[a]
On bridge substructures and wing and retaining walls	1.0	1.4[a]
For associated primary live loads	1.25	1.25
	For serviceability limit state	
On elastomeric bearings	1.0	1.0
For associated primary live loads	1.0	1.0

[a] 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 Vehicle collision loads on bridge supports and superstructures over highways

The design of bridges for vehicle collision loads shall be in accordance with the requirements of the relevant authority. The following requirements in **6.8.1**, **6.8.2**, **6.8.3**, **6.8.4**, **6.8.5**, and **6.8.6** may still be applicable in certain circumstances where agreed with the relevant 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 kN	**Load parallel to the carriageway below** kN	**Point of application on bridge support**
Load transmitted from safety fence	150	50	Any one bracket attachment point or for free-standing fences, any one point 0.75 m above carriageway level
Residual load above safety fence	100	100	At the most severe point between 1 m and 3 m above carriageway level

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 horizontal 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 super elevated 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 shall only be 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 relevant authority.

6.9 Centrifugal loads

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

6.9.1 ***Nominal centrifugal load***

A nominal centrifugal load F_c (kN) shall be taken as:

$$F_c = \frac{40\ 000}{r + 150}$$

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 6 m.

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:

a) for ultimate limit state: 1.50

b) for the serviceability state: 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 8 kN/m of loaded length plus 250 kN, subject to a maximum of 750 kN, applied to an area one notional lane wide multiplied by 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 two axles of the vehicle, 1.8 m 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:

a) for the ultimate limit state: 1.25

b) for the serviceability state: 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 36 m and under, a uniformly distributed live load of 5.0 kN/m^2;

b) for loaded lengths in excess of 36 m, $k \times 5.0$ kN/m^2

where

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

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 36 m 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 Annex B). Such loading shall be agreed with the relevant authority.

Consideration shall be given to the requirements of carrying out appropriate testing in order to verify that the bridge is suitable for entry into public service and provisions for future installation of vibration reduction devices, such as dampers, if required.

7.1.2 ***Effects due to horizontal loading on pedestrian parapets***

The design of pedestrian parapets shall be in accordance with the appropriate standard.

7.1.3 ***Design load***

For the live load on foot/cycle tracks 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 and 3	1.25	1.00

7.2 Vehicle collision loads on foot/cycle track bridge supports and superstructures over highways

The design of foot/cycle track bridge supports for vehicle collision loads shall be in accordance with the requirements of the relevant authority. Subclauses **6.8.1**, **6.8.2**, **6.8.3**, **6.8.4**, **6.8.5** and **6.8.6** may still be applicable in certain circumstances where agreed with the relevant authority. For foot/cycle track bridges with minimum new headroom clearance of 5.7 m, vehicle collision loads on superstructures need not be applied. For foot/cycle track bridges with minimum new headroom clearance of less than 5.7 m the vehicle collision loads on superstructures given in **6.8.2** shall not be applied and the impact requirements shall be obtained from the relevant 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 Annex B conforms to the limitations specified therein.

8 Railway bridge live loads

8.1 General

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

RU loading (including SW/0 loading for continuous beams) allows for all combinations of normal 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 1.4 m 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.

Annex D gives the derivation of standard railway loadings together with tables of equivalent uniformly distributed loading for type RU loading (defined in **8.2.1.1**), which may be used for simply supported spans.

Design requirements for railway loading beyond the scope of Annex D shall be agreed with the relevant authority.

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 15 m long and separated by a distance of 5.3 m. The arrangement of this loading is as shown in Figure 15.

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 100 m. For loaded lengths in excess of 100 m the distributed nominal load shall be 50 kN/m for the first 100 m and shall be reduced to 25 kN/m for lengths in excess of 100 m, as shown in Figure 16.

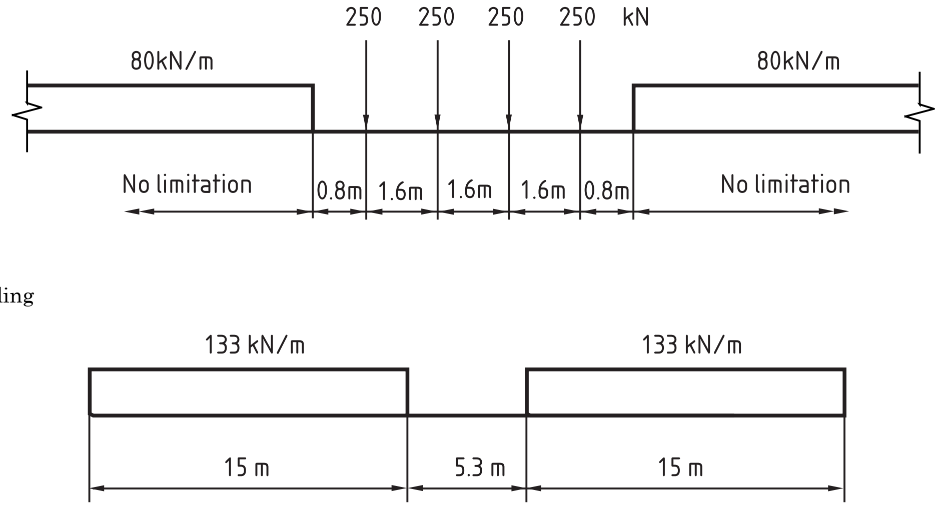
Alternatively, two concentrated nominal loads, one of 300 kN and the other of 150 kN, spaced at 2.4 m intervals along the track, shall be used on deck elements where this gives a more severe condition. These two concentrated loads shall be deemed to include dynamic effects.

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, unless otherwise specified by the relevant authority.

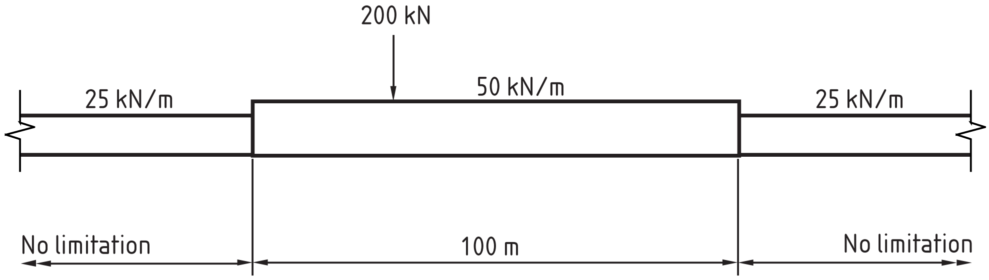
a) Type RU loading

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

8.2.3.1 *Type RU and SW/0 loading*

Except as otherwise specified, the dynamic factor for RU and SW/0 loadings 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	2.0	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, 3 m 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 the limits set out in UIC Leaflet 776-3R [2] (in Figure 1 of which the expression for δ_u for spans between 20 and 100 m 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.

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

	Dimension *L* m
Main girders:	
simply supported	span
continuous	for 2, 3, 4, 5 and more spans 1.2, 1.3, 1.4, 1.5 × mean span, but at least the greatest span
portal frames and arches	½ span
Floor members:	
simply supported rail bearers	cross girder spacing plus 3 m
cross girders loaded by simply supported rail bearers	twice the spacing of cross girders plus 3 m
end cross girders or trimmers	4 m
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

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 200 mm, 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.

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 **4.5**. Any number of lengths of the distributed load may be applied, but the total length of 50 kN/m intensity shall not exceed 100 m 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 and every track as specified in **4.5**. 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 elements of railway bridges carrying one or two tracks, SW/0 loading shall be applied as specified in **4.5** as an additional and separate load case to the requirements of **8.2.6.2**. For bridges carrying more than two tracks, SW/0 loading shall be applied as specified by 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.

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 unless otherwise specified by the relevant authority.

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. The nominal centrifugal load F_c, in kN, per track acting radially at a height of 1.8 m above rail level shall be calculated from the following formula.

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

where

- P is the static nominal vertical railway loading in accordance with **8.2.1** 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 $= 1 - \left[\frac{v_t - 120}{1000}\right] \times \left[\frac{814}{v_t} + 1.75\right] \times \left[1 - \sqrt{\frac{2.88}{L}}\right]$

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

  = unity for L less than 2.88 m or v_t less 120 km/h
- L is the loaded length of the element being considered.

The number of tracks and loaded lengths considered for centrifugal loading shall be consistent with the number assumed to be occupied for vertical loading. In addition, for a bridge located on a curve, the effects of cant of the tracks shall be considered, both with and without centrifugal force.

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 resisted by track outside the bridge structure, provided that no expansion switches or similar rail discontinuities are located on, or within, 18 m 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 elements 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, shall 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.

The longitudinal loading to be used in conjunction with SW/10 loading may be assumed to be the same as the longitudinal loading used in conjunction with RU loading as given in Table 18.

8.2.11 ***Aerodynamic effects from passing trains***

Provision shall be made for the nominal loads due to aerodynamic effects from passing trains. The dynamic response of the structure to aerodynamic effects shall also be taken into account. 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.

Table 18 — Nominal longitudinal loads

Standard loading type	Loading arising from	Loaded length m	Longitudinal load kN
RU and SW/10	Traction (30% of load on driving wheels)	up to 3	150
		from 3 to 5	225
		from 5 to 7	300
		from 7 to 25	$24(L - 7) + 300$
		over 25	750
	Braking (25 % of load on braked wheels)	up to 3	125
		from 3 to 5	187
		from 5 to 7	250
		over 7	$20(L - 7) + 250$
RL	Traction (30 % of load on driving wheels)	up to 8	80
		from 8 to 30	10 kN/m
		from 30 to 60	300
		from 60 to 100	5 kN/m
		over 100	500
	Braking (25 % of load on braked wheels)	up to 8	64
		from 8 to 100	8 kN/m
		over 100	800

In assessing combination 3, where appropriate, 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 also be considered. The requirements for checking combined response in the structure and track shall be agreed with the relevant authority.

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
For combination 1	1.40	1.10
For combinations 2 and 3	1.20	1.00

8.5 Derailment loads

Railway bridges shall be designed so 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) shall 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 conform to these requirements.

8.5.1 ***Design load for RU loading (including SW/0)***

The following equivalent static design 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, 1.4 m 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, each line with four loads 1.6 m 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 20 m anywhere along the span (see **4.6**).

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 design 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 (see **4.6**).

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 and superstructures of bridges over railways

The collision load on supports and superstructures of bridges over railways shall be as agreed with the relevant authorities.

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

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 Deformation requirements

The deformation limits (vertical, twist, 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 designs loads, γ_{fL} to be applied to the nominal loads shall be as specified in **6.5.3** and **8.4** respectively.

Annex A (normative)
Basis of HA and HB highway loading

A.1 Historical background to highway loading

Type HA loading has been revised to take into account the results of 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[6] 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 10 m reducing linearly from 10 m to unity at a loaded length of 60 m, 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 a traffic lane. The factor was based on the ratio of the standard lane width, 3.65 m, to the maximum permitted width of normal vehicles[6] which is 2.5 m.

The HA loading is therefore given in terms of a 3.65 m 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 5400-2:1978 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 the United Kingdom 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 (e.g. electrical transformers, generators, pressure vessels and machine presses) likely to use the roads in the area.

A.2 Design of highway structures subject to abnormal indivisible loads (AIL)

Unless specified by the relevant authority, the following conditions may be applied in addition to **5.7**. When designing 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 this standard;

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

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[6] 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 The Stationery Office Ltd.

Annex B (normative) Vibration serviceability requirements for foot and cycle track bridges

B.1 General

For superstructures for which the fundamental natural frequency f_0 of vibration exceeds 5 Hz 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 susceptible to such vibrations. The method for deriving maximum horizontal acceleration should be agreed with the relevant 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 idealized 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 based 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 deal load but excluding pedestrian live loading and may be calculated from the following equation.

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

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 B.1)

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**)

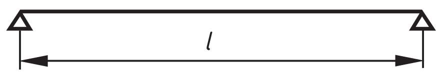
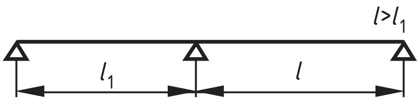
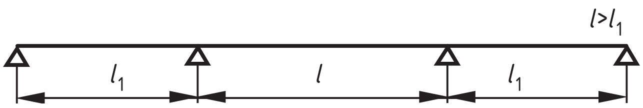
M is the weight per unit length of the full cross-section at midspan (in kN/m)

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 B.1.

Table B.1 — Configuration factor C

Bridge configuration	Ratio l_1/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.

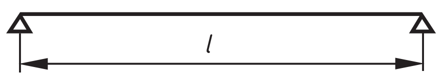
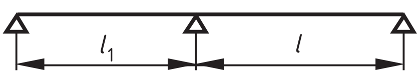
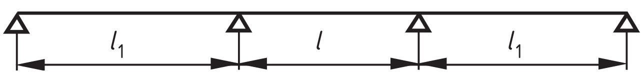
B.2.4 *Static deflection y_s*

The static deflection y_s is taken at the midpoint of the main span for 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 B.2.

Table B.2 — Configuration factor K

Bridge configuration	Ratio l_1/l	K
	—	1.0
	—	0.7
	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 B.1. 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 B.3 should be used.

Table B.3 — Logarithmic decrement of decay of vibration δ

Bridge supersxtructure	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)

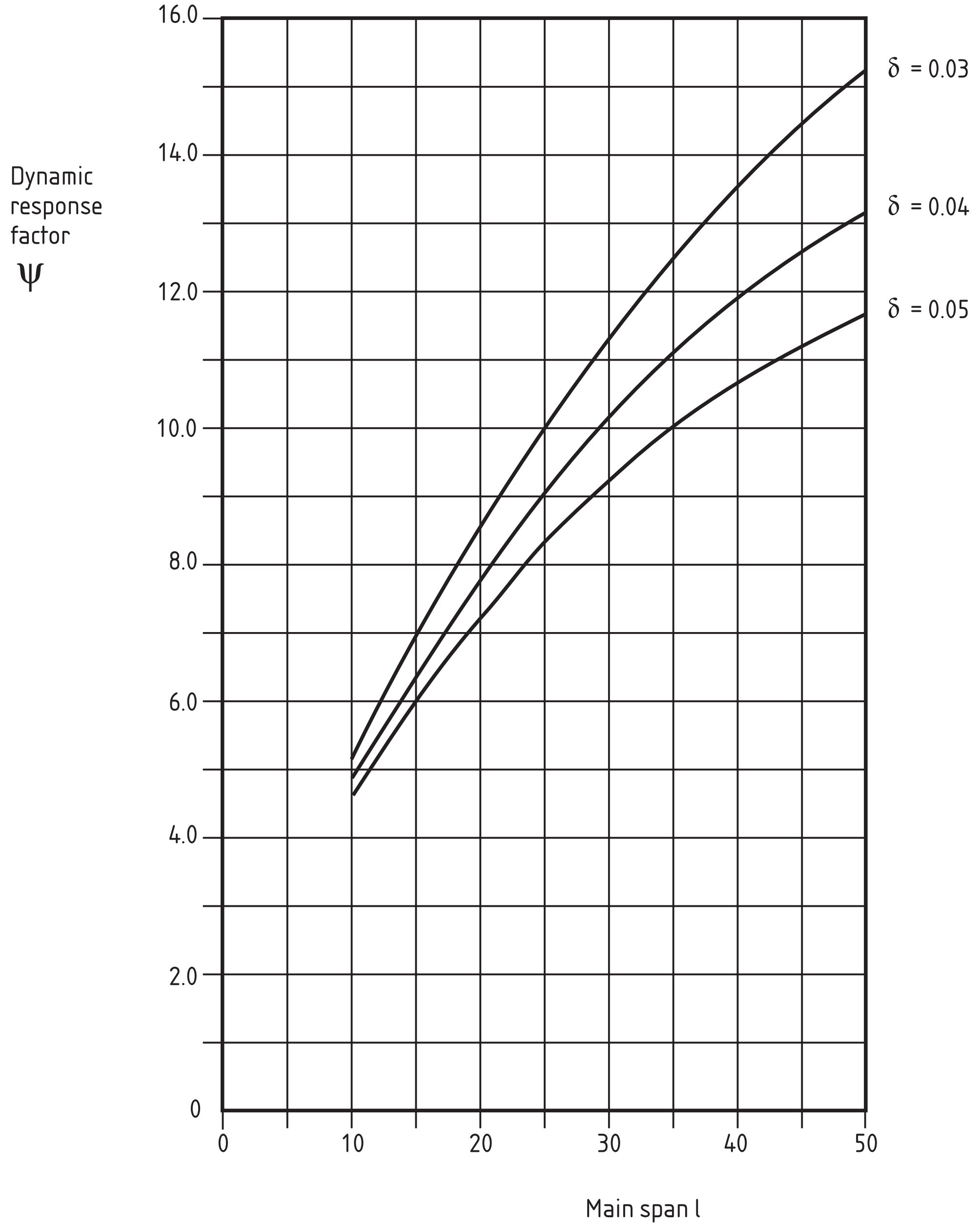
v_t = 0.9 f_o (in m/s)

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.



NOTE 1 Main span l is shown in Table B.1 and Table B.2.

NOTE 2 Values of δ for different types of construction are given in Table B.3.

Figure B.1 — Dynamic response factor Ψ

Annex C (normative)
Temperature differences *T* for various surfacing depths

The values of *T* given in Figure 9 are for 40 mm surfacing depths for groups 1 and 2 and 100 mm surfacing depths for groups 3 and 4. For other depths of surfacing, the values given Table C.1a), Table C.1b), Table C.2 and Table C.3 may be used. In Table C.1a), Table C.1b), Table C.2 and Table C.3 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 C.1a) — Values of *T* for group 1

Surfacing thickness mm	Positive temperature difference °C				Reverse temperature difference °C
	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 C.1b) — Values of *T* for group 2

Surfacing thickness mm	Positive temperature difference °C	Reverse temperature difference °C
	T_1	T_2
Unsurfaced	25	6
20	23	5
40	21	5

Table C.2 — Values of *T* for group 3

Depth of slab *h* m	Surfacing thickness mm	Positive temperature difference °C	Reverse temperature difference °C
		T_1	T_1
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 C.3 — Values of *T* for group 4

Depth of slab h m	Surfacing thickness mm	Positive temperature differences °C			Reverse temperature difference °C			
		T_1	T_2	T_3	T_1	T_2	T_3	T_4
≤ 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 purpose 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 reverse 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 c). These are now the temperatures which co-exist with the minimum effective bridge temperature and reverse 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.

Annex D (normative) 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 normal loading on railways in the United Kingdom and on the continent of Europe. Motive power is now 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.

Type RU loading, defined in **8.2.1.1** does not adequately cover the load effects of normal rail traffic on continuous beams. For continuous elements, it is necessary to take into account SW/0 loading as defined in **8.2.1.2**.

The concentrated and distributed loads have been approximately converted into equivalent loads measured in kN when applying RU loading in Table D.1, Table D.2, Table D.3 and Table D.4.

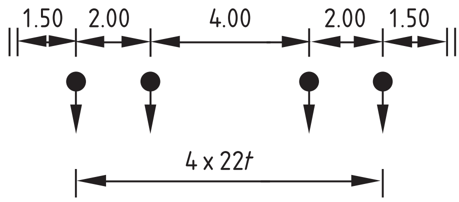
Figure D.1 shows diagrams of two locomotives and several wagons all of which, when forming part of a train, are covered by RU loading. Double heading of the locomotives has been allowed for in RU loading.

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 and 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 recommendations published by the International Union of Railways (UIC), Paris [3].

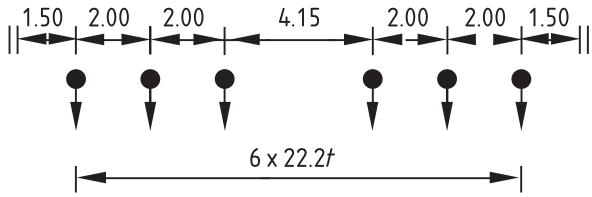
Similar combinations of vehicle weight and speed have to be considered in the calculation of centrifugal loads. The factor f given in **8.2.9** allows for the reduction in vehicle weight with increasing speed above certain limits.

B-B LOCOMOTIVE



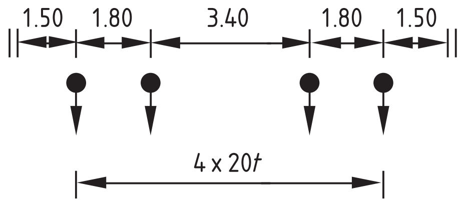
To be considered double headed

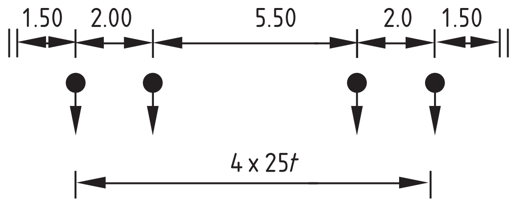
C-C LOCOMOTIVE



To be considered double headed

WAGONS

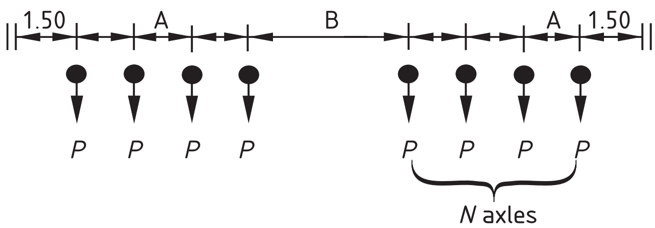




EXCEPTIONAL WAGONS

N is the number of axles in each end

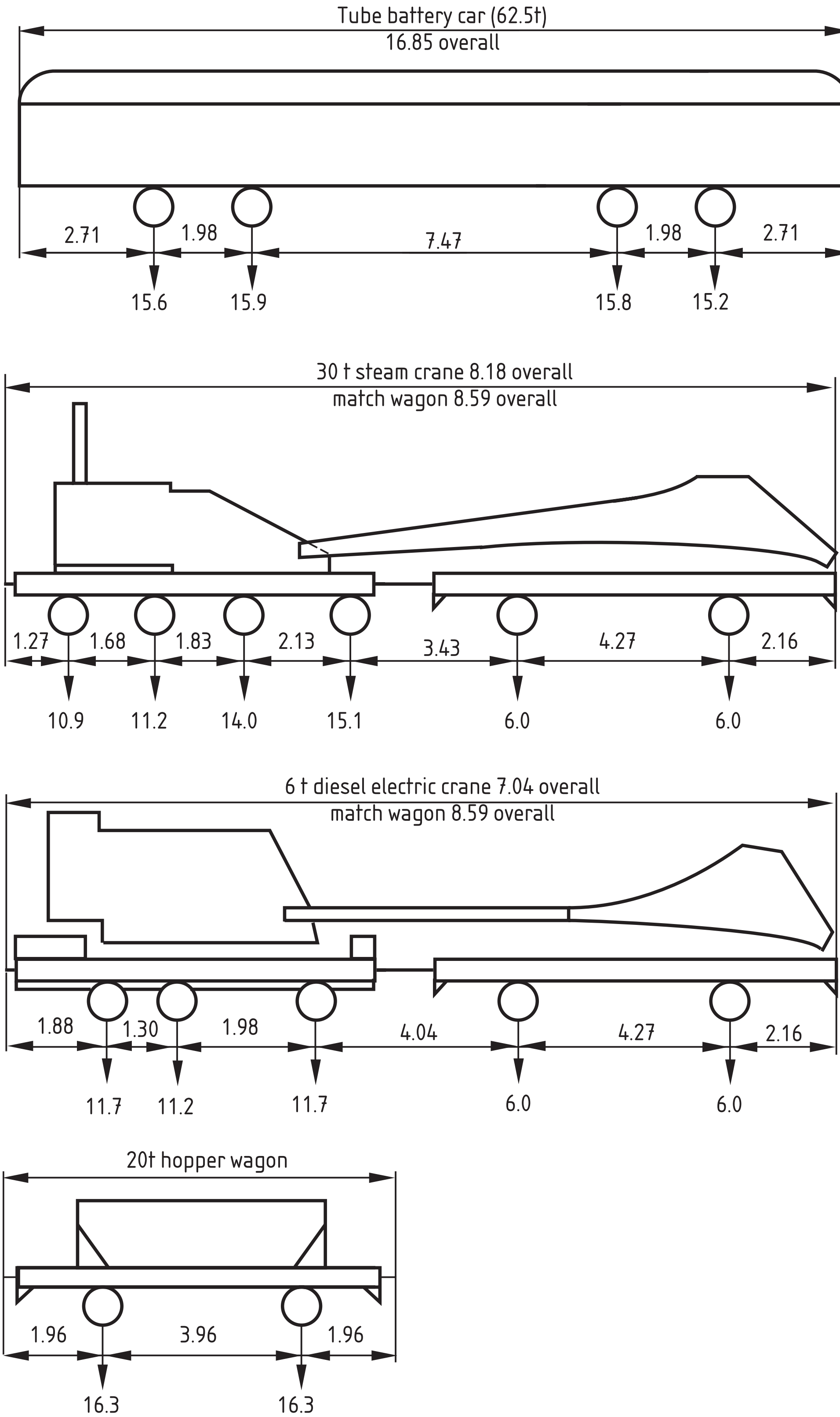
2*N* is the total number of axles



P	***A*** metres						
Weight per axle	**number of axles *N***						
tonnes	**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 metres						
No variation	6.00	6.00	7.00	8.00	8.00	10.00	10.00

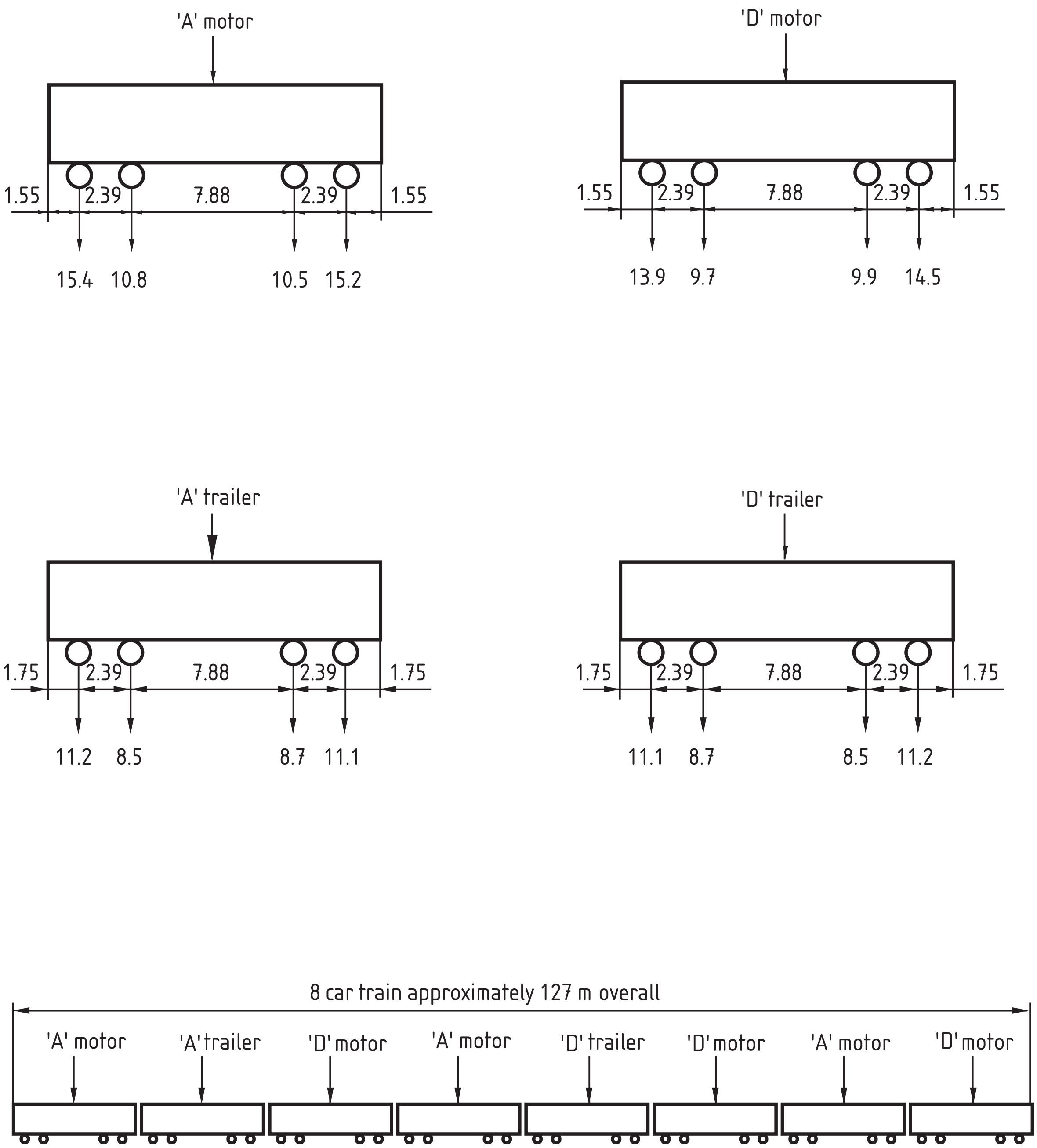
NOTE Dimensions are in metres. Axle weights are in tonnes.

Figure D.1 — Wagons and locomotives covered by RU loading



NOTE Dimensions are in metres. Axle weights are in tonnes.

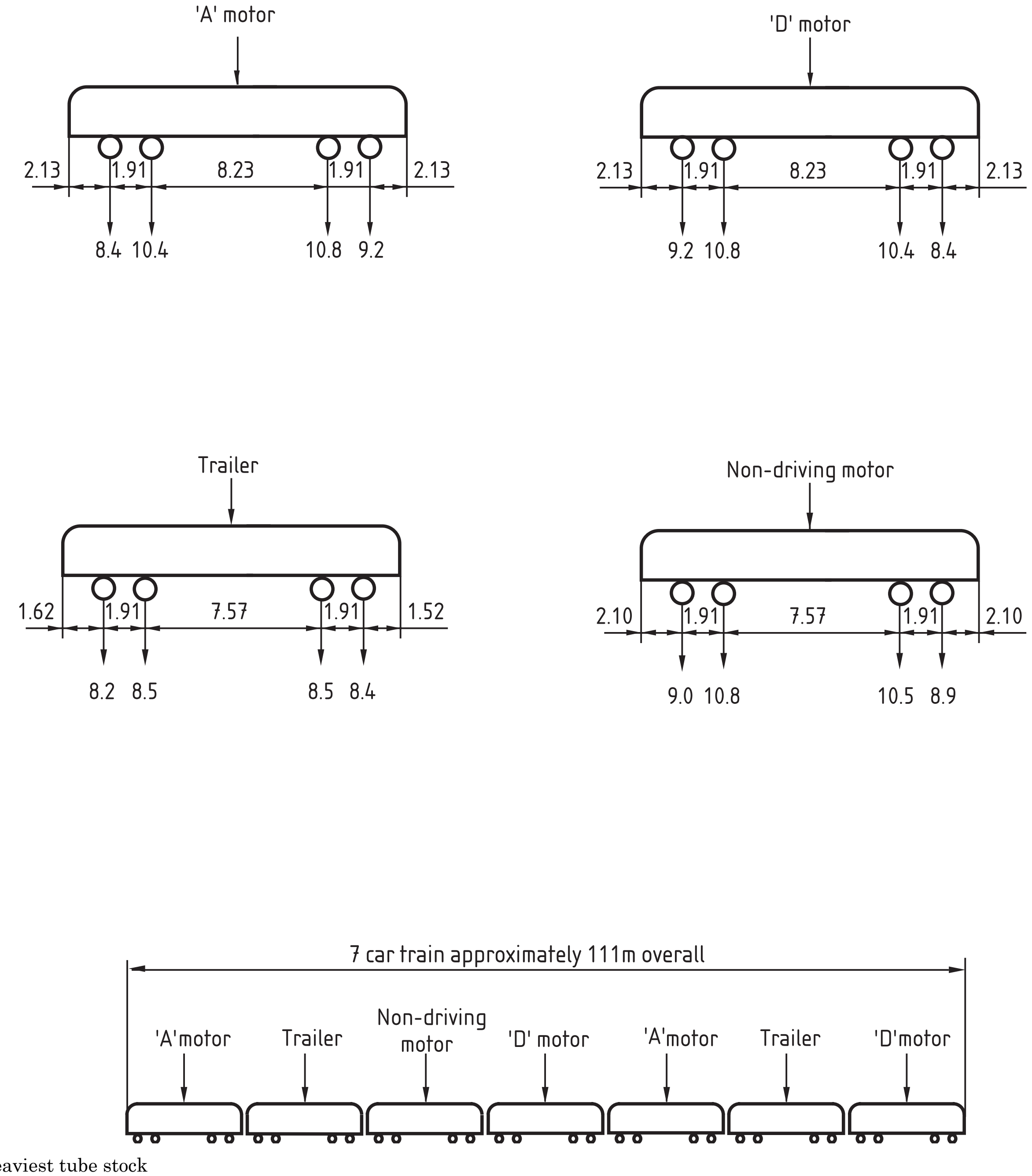
Figure D.2 — Works trains vehicles covered by RL loading



a) Heaviest surface stock

NOTE Dimensions are in metres. Axle weights are in tonnes.

Figure D.3 — Passenger vehicles covered by RL loading



b) Heaviest tube stock

NOTE Dimensions are in metres. Axle weights are in tonnes.

Figure D.3 — Passenger vehicles covered by RL loading (*continued*)

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 annex 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 Figure D.2 and Figure D.3.

a) *Work 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 30 t cranes, 6 t diesel cranes, 20 t 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 10 km/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.

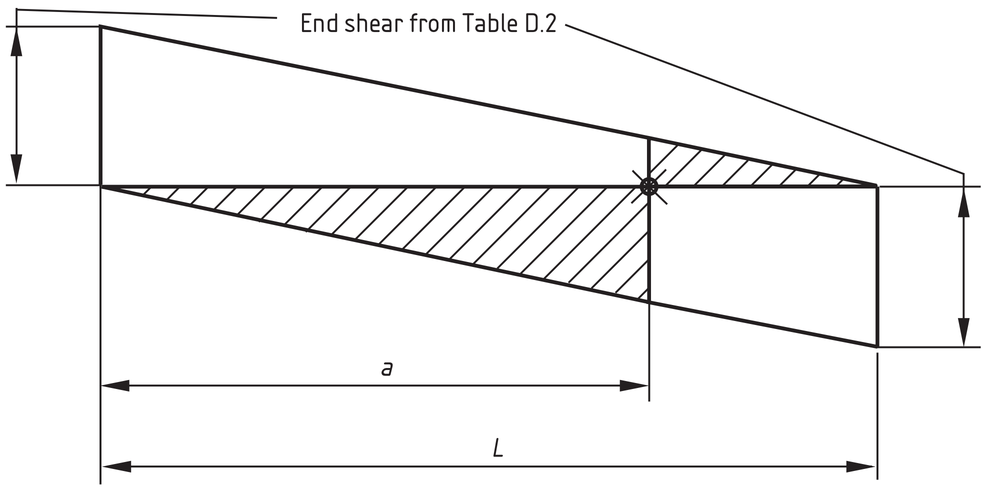
D.3 Use of Table D.1, Table D.2, Table D.3 and Table D.4 when designing for RU loads

D.3.1 *Simply supported main girders and rail bearers*

Bending moments in simply supported girders may 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 may be taken from the tables giving end shear forces.

Shear forces at points other than the end may be determined by using the static shear force from Table D.2 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 D.4) and the result shall be multiplied by the dynamic factor for shear in which L is taken to be the span of the girder.



The shear force at point X is the end shear for span length a multiplied by a/L

Figure D.4 — Shear force determination

D.3.2 *Cross girders loaded through simply supported rail bearers*

The cross girders may be designed to carry two concentrated point loads for each track. Each of these loads being taken as one-quarter of the equivalent uniformly distributed load for bending moments shown in Table D.1 for a span equal to twice the cross girder spacing, multiplied by the appropriate dynamic factor.

Table D.1 — Equivalent uniformly distributed loads for bending moments for simply supported beams (static loading) under RU loading

Span m	Load kN	Span m	Load kN	Span m	Load 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 D.2 — End shear forces for simply supported beams (static loading) under RU loading

Span m	Load kN	Span m	Load kN	Span m	Load 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 D.3 — Equivalent uniformly distributed loads for bending moments for simply supported beams, including dynamic effects under RU loading

Span m	Load kN	Span m	Load kN	Span m	Load 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 D.4 — End shear forces for simply supported beams, including dynamic effects, under RU loading

Span m	Load kN	Span m	Load kN	Span m	Load 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		

Annex E (normative)
Probability factor S_p and seasonal factor

E.1 Probability factor S_p

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

$$S_p = \sqrt{\left(\frac{\{5 - \ln(-\ln[1 - Q])\}}{\{5 - \ln(-\ln 0.98)\}}\right)}$$

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 $Q = 0.008\ 3$ 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 hourly mean wind speeds while maintaining a risk of exceedance of $Q = 0.02$ in the stated period. The basic hourly mean 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 E.1.

Table E.1 — 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>January</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>February</td><td>0.83</td><td rowspan="2">0.86</td><td rowspan="4">0.87</td></tr>
<tr><td>March</td><td>0.82</td><td rowspan="2">0.83</td><td rowspan="4">0.83</td></tr>
<tr><td>April</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>June</td><td>0.66</td><td rowspan="2">0.67</td><td rowspan="4">0.83</td></tr>
<tr><td>July</td><td>0.62</td><td rowspan="2">0.71</td><td rowspan="4">0.86</td></tr>
<tr><td>August</td><td>0.71</td><td rowspan="2">0.82</td><td rowspan="4">0.90</td></tr>
<tr><td>September</td><td>0.82</td><td rowspan="2">0.85</td><td rowspan="4">0.96</td></tr>
<tr><td>October</td><td>0.82</td><td rowspan="2">0.89</td><td rowspan="4">1.00</td></tr>
<tr><td>November</td><td>0.88</td><td rowspan="2">0.95</td><td rowspan="5">1.00</td></tr>
<tr><td>December</td><td>0.94</td><td rowspan="2">1.00</td><td rowspan="3">1.00</td></tr>
<tr><td>January</td><td>0.98</td><td rowspan="2">0.98</td><td rowspan="3"></td></tr>
<tr><td>February</td><td>0.83</td><td rowspan="2">0.86</td><td rowspan="2"></td></tr>
<tr><td>March</td><td>0.82</td><td></td><td></td></tr>
<tr><td colspan="8">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.</td></tr>
</table>

Annex F (normative) Topographical 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 Topgraphy 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 **5.3.2.3.3** and Figure F.1, Figure F.2a) and Figure F.2b), 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 **5.3.2.2.2**.

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 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 F.1. 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 F.1 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_U the upwind slope Z/L in the wind direction
- Ψ_D the downwind slope Z/L in the wind direction
- L_e the effective length of the upwind slope, defined in Table F.1
- 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 Figure F.2a), Figure F.2b), Figure F.3a) and Figure F.3b).

The influence of any topography feature should be considered to extend $X = 15L_e$ upwind and $X = 2.5L_e$ 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 ($\Psi_D < 0.05$) 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 F.1). 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 5 km 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 F.1 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:

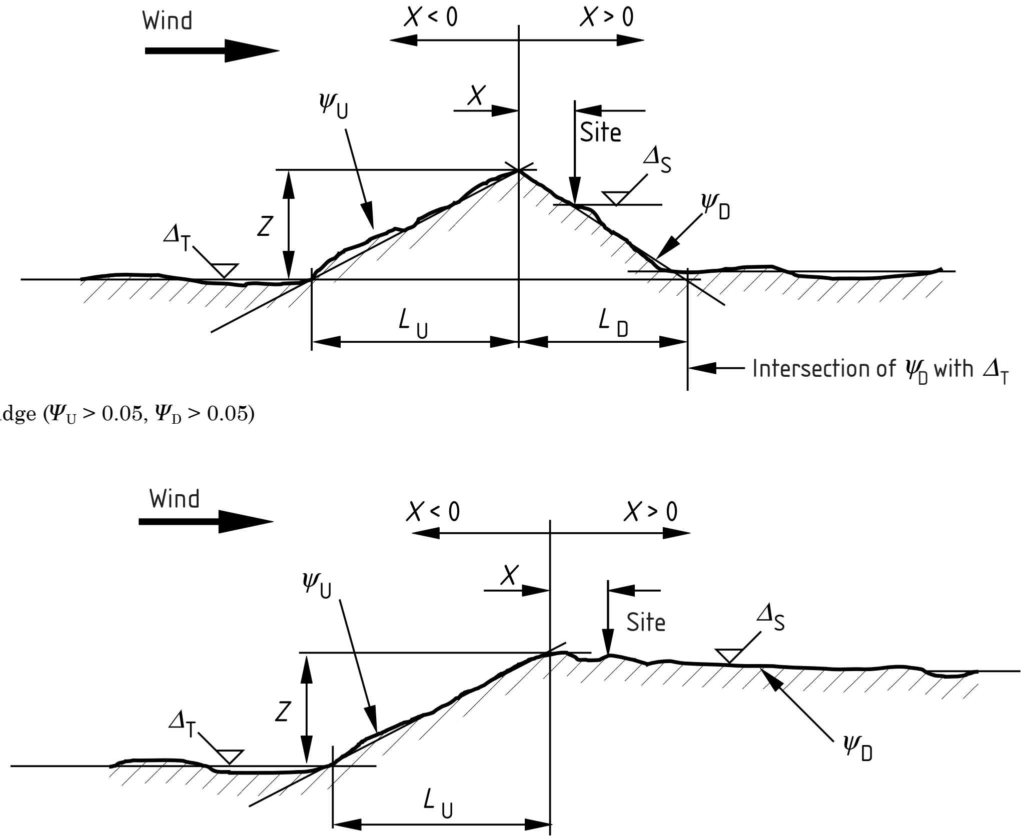
a) Figure F.2a) and Figure F.2b) for hills and ridges; or

b) Figure F.3a) and Figure F.3b) for cliffs and escarpments.

Where the downwind slope of a hill or ridge is greater than $\Psi_D = 0.3$ 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 F.2a) should be used as upper bound values.

Table F.1 — Values of L_e and S_h

Slope ($\Psi = Z/L_D$)	Shallow ($0.05 < \Psi < 0.3$)	Steep ($\Psi > 0.3$)
Effective length L_e	$L_e = L_U$	$L_e = Z/0.3$
Topography factor S_h	$S_h = 2.0\ \Psi s$	$S_h = 0.6s$



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 ($\Psi_U > 0.03$, $\Psi_D < 0.05$)

Key

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

Figure F.1 — Definition of topographic dimensions

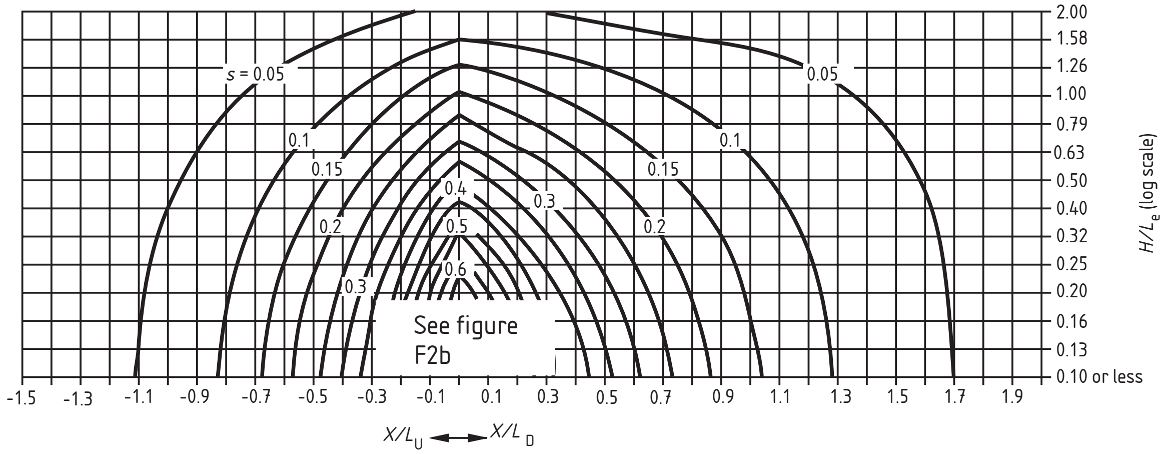


Figure F.2a) — Topographic location factors for hills and ridges

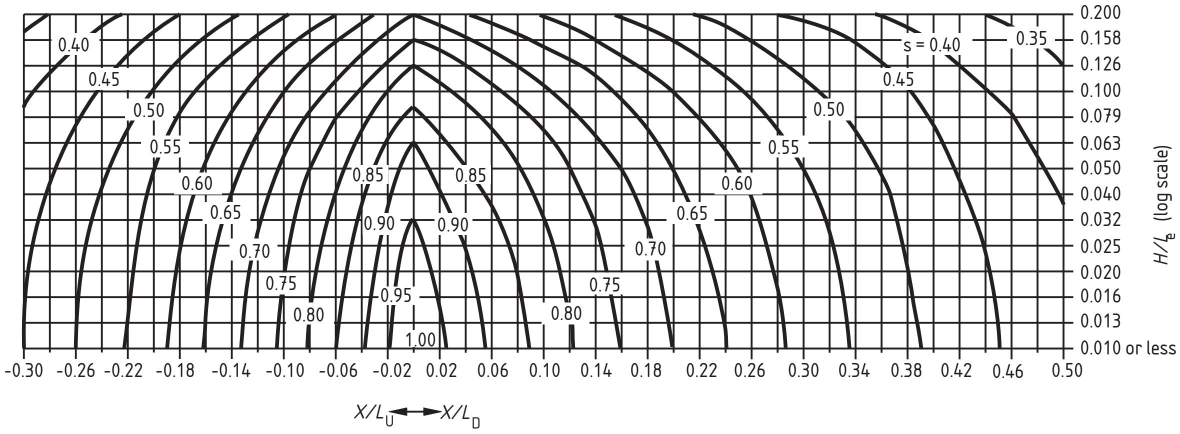


Figure F.2b) — Topographic location factors for hills and ridges

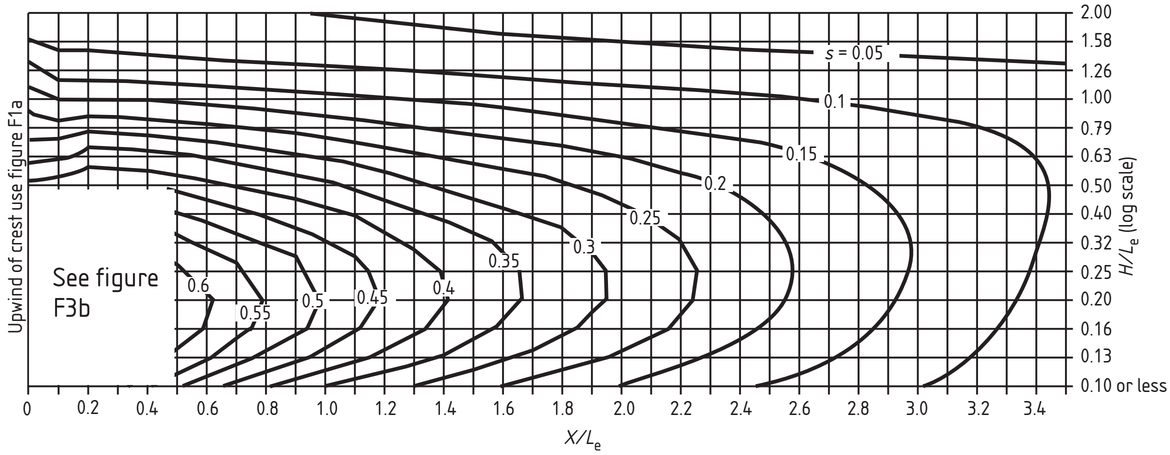


Figure F.3a) — Topographic location factors for cliffs and escarpments

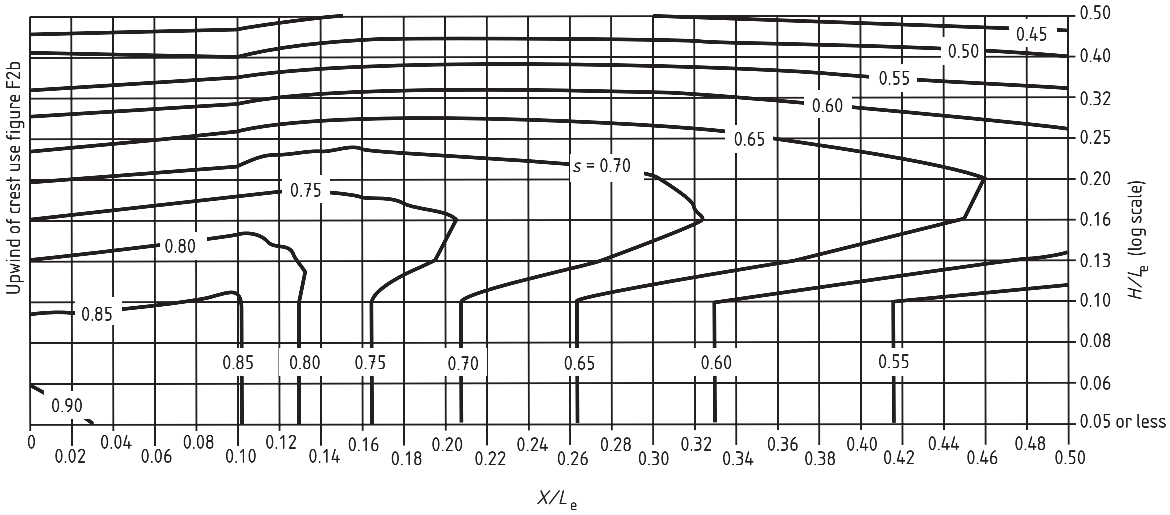


Figure F.3b) — Topographic location factors for cliffs and escarpments

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Standards publications

BS 6100-2 (multisection standard), *Glossary of building and civil engineering terms — Part 2: Civil engineering.*

BS 6399-2:1997, *Loading for buildings — Part 2: Code of practice for wind loads.*

Other publications

[1] The following referenced documents from the Highways Agency Design Manual for Roads and Bridges may provide additional information for the design of highway bridges. The latest edition of the referred document (including any amendments) applies.

— BD 12 (DMRB 2.2), Design of corrugated steel buried structures with spans not exceeding 8 m including circular arches

— BD 30 (DMRB 2.1), Backfilled retained walls and bridge abutments

— BD 31 (DMRB 2.2), Buried concrete box type structures

— BD 42 (DMRB 2.1), Design of embedded retaining walls and bridge abutments

— BD 60 (DMRB 1.3), Design of highway bridges for vehicle collision loads

— TD 27 (DMRB 6.1), Cross-sections and headrooms.

[2] 776-3R (1989 Edition) published by UIC, 16 rue Jean Rey, F, 75015, Paris, France.

[3] Leaflet 776-1R, published by UIC, 16 rue Jean Rey F, 75015 Paris, France.

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