

Steel, concrete and composite bridges —

Part 2: Specification for loads

ICS 93.040

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

This British Standard was published under the authority of the Standards Policy and Strategy Committee on 29 September 2006

© BSI 2006

First edition June 1978

The following BSI references relate to the work on this British Standard:
Committee reference B/525/10
Draft for comment DC04/30071551

ISBN 0 580 46715 5

Amendments issued since publication

Amd. No.	Date	Comments

Contents

	Page
Committees responsible	Inside front cover
Foreword	iv
1 Scope	1
1.1 General	1
1.2 Documents comprising this British Standard	1
1.3 Loads and factors specified in this Part of BS 5400	1
1.4 Wind and temperature	1
2 Normative references	1
3 Principles, definitions and symbols	1
4 Loads — General	7
4.1 Loads and factors specified	7
4.2 Loads to be considered	7
4.3 Classification of loads	7
4.4 Combination of loads	10
4.5 Application of loads	10
4.6 Overturning	11
4.7 Foundation pressures, sliding on foundations, loads on piles, etc.	11
5 Loads applicable to all bridges	11
5.1 Dead load	11
5.2 Superimposed dead load	12
5.3 Wind loads	13
5.4 Temperature	31
5.5 Effects of shrinkage and creep, residual stresses, etc.	40
5.6 Differential settlement	40
5.7 Exceptional loads	40
5.8 Earth pressure on retaining structures	41
5.9 Erection loads	41
6 Highway bridge live loads	42
6.1 General	42
6.2 Type HA loading	43
6.3 Type HB loading	46
6.4 Application of types HA and HB loading	47
6.5 Standard footway and cycle track loading	49
6.6 Accidental loading	52
6.7 Loads due to vehicle collision with parapets	53
6.8 Vehicle collision loads on bridge supports and superstructures over highways	54
6.9 Centrifugal loads	55
6.10 Longitudinal load	56
6.11 Accidental load due to skidding	56
6.12 Loading for fatigue investigations	57
6.13 Dynamic loading on highway bridges	57
7 Foot/cycle track bridge live loads	57
7.1 Standard foot/cycle track bridge loading	57
7.2 Vehicle collision loads on foot/cycle track bridge supports and superstructures over highways	58
7.3 Vibration serviceability	58

8	Railway bridge live loads	58
8.1	General	58
8.2	Nominal loads	58
8.3	Load combinations	62
8.4	Design loads	63
8.5	Derailment loads	63
8.6	Collision load on supports and superstructures of bridges over railways	64
8.7	Loading for fatigue investigations	64
8.8	Deformation requirements	64
8.9	Footway and cycle track loading on railway bridges	64
	Annex A (normative) Basis of HA and HB highway loading	65
A.1	Historical background to highway loading	65
A.2	Design of highway structures subject to abnormal indivisible loads (AIL)	65
	Annex B (normative) Vibration serviceability requirements for foot and cycle track bridges	66
B.1	General	66
B.2	Simplified method for deriving maximum vertical acceleration	66
B.3	General method for deriving maximum vertical acceleration	68
B.4	Damage from forced vibration	68
	Annex C (normative) Temperature differences T for various surfacing depths	70
	Annex D (normative) Derivation of RU and RL railway loadings	72
D.1	RU loading	72
D.2	RL Loading	77
D.3	Use of Table D.1, Table D.2, Table D.3 and Table D.4 when designing for RU loads	78
	Annex E (normative) Probability factor S_p and seasonal factor	83
E.1	Probability factor S_p	83
	Annex F (normative) Topographical factor S_h'	84
F.1	General	84
F.2	Topography significance	84
F.3	Altitude	84
F.4	Gust speeds	84
F.5	Hourly mean speeds	84
F.6	Topography features	84
	Bibliography	88
	Figure 1 — Highway carriageway and traffic lanes	3
	Figure 2 — Basic wind speed V_b in m/s	14
	Figure 3 — Definition of significant topography	17
	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	23
	Figure 5 — Drag coefficient C_D for superstructures with solid elevation	25
	Figure 6 — Lift coefficient C_L	30
	Figure 7 — Isotherm of minimum shade air temperature (in °C)	32
	Figure 8 — Isotherms of maximum shade air temperature (in °C)	33
	Figure 9 — Temperature difference for different types of construction	37
	Figure 10 — Loading curve HA UDL	43
	Figure 11 — Base lengths for highly cusped influence lines	45
	Figure 12 — Dimensions of HB vehicle	46
	Figure 13 — Type HA and HB highway loading in combination	50

Figure 14 — Accidental wheel loading	52
Figure 15 — Type RU loading and Type SW/0 loading	59
Figure 16 — Type RL loading	59
Figure B.1 — Dynamic response factor Ψ	69
Figure D.1 — Wagons and locomotives covered by RU loading	73
Figure D.2 — Works trains vehicles covered by RL loading	74
Figure D.3 — Passenger vehicles covered by RL loading	75
Figure D.4 — Shear force determination	78
Figure F.1 — Definition of topographic dimensions	86
Figure F.2 — Topographical location factors for hills and ridges	86
Figure F.3 — Topographic location factors for cliffs and escarpments	87
Table 1 — Loads to be taken in each combination with appropriate γ_{FL}	8
Table 2 — Values of direction factor S_d	15
Table 3 — Values of terrain and bridge factor S_b' , hourly speed factor S_c' and fetch correction factor, K_F	18
Table 4 — Gust speed reduction factor T_g for bridges in towns	19
Table 5 — Hourly mean reduction factor T_c for bridges in towns	19
Table 6 — Drag coefficient C_D for a single truss	26
Table 7 — Shielding factor η	26
Table 8 — Drag coefficient C_D for parapets and safety fences	27
Table 9 — Drag coefficient C_D for piers	28
Table 10 — Minimum effective bridge temperature	35
Table 11 — Maximum effective bridge temperature	36
Table 12 — Adjustment to effective bridge temperature for deck surfacing	36
Table 13 — Type HA uniformly distributed load	44
Table 14 — HA lane factors	48
Table 15 — Collision loads on supports of bridges over highways	55
Table 16 — Dynamic factors for type RU loading	59
Table 17 — Dimension L used in calculating the dynamic factor for RU loading	60
Table 18 — Nominal longitudinal loads	63
Table B.1 — Configuration factor C	67
Table B.2 — Configuration factor K	67
Table B.3 — Logarithmic decrement of decay of vibration δ	68
Table C.1a) — Values of T for group 1	70
Table C.1b) — Values of T for group 2	70
Table C.2 — Values of T for group 3	70
Table C.3 — Values of T for group 4	71
Table D.1 — Equivalent uniformly distributed loads for bending moments for simply supported beams (static loading) under RU loading	79
Table D.2 — End shear forces for simply supported beams (static loading) under RU loading	80
Table D.3 — Equivalent uniformly distributed loads for bending moments for simply supported beams, including dynamic effects under RU loading	81
Table D.4 — End shear forces for simply supported beams, including dynamic effects, under RU loading	82
Table E.1 — Values of seasonal factor S_s	83
Table F.1 — Values of L_e and S_h	85

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.

The BSI copyright notice displayed in this document indicates when the document was last issued.

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

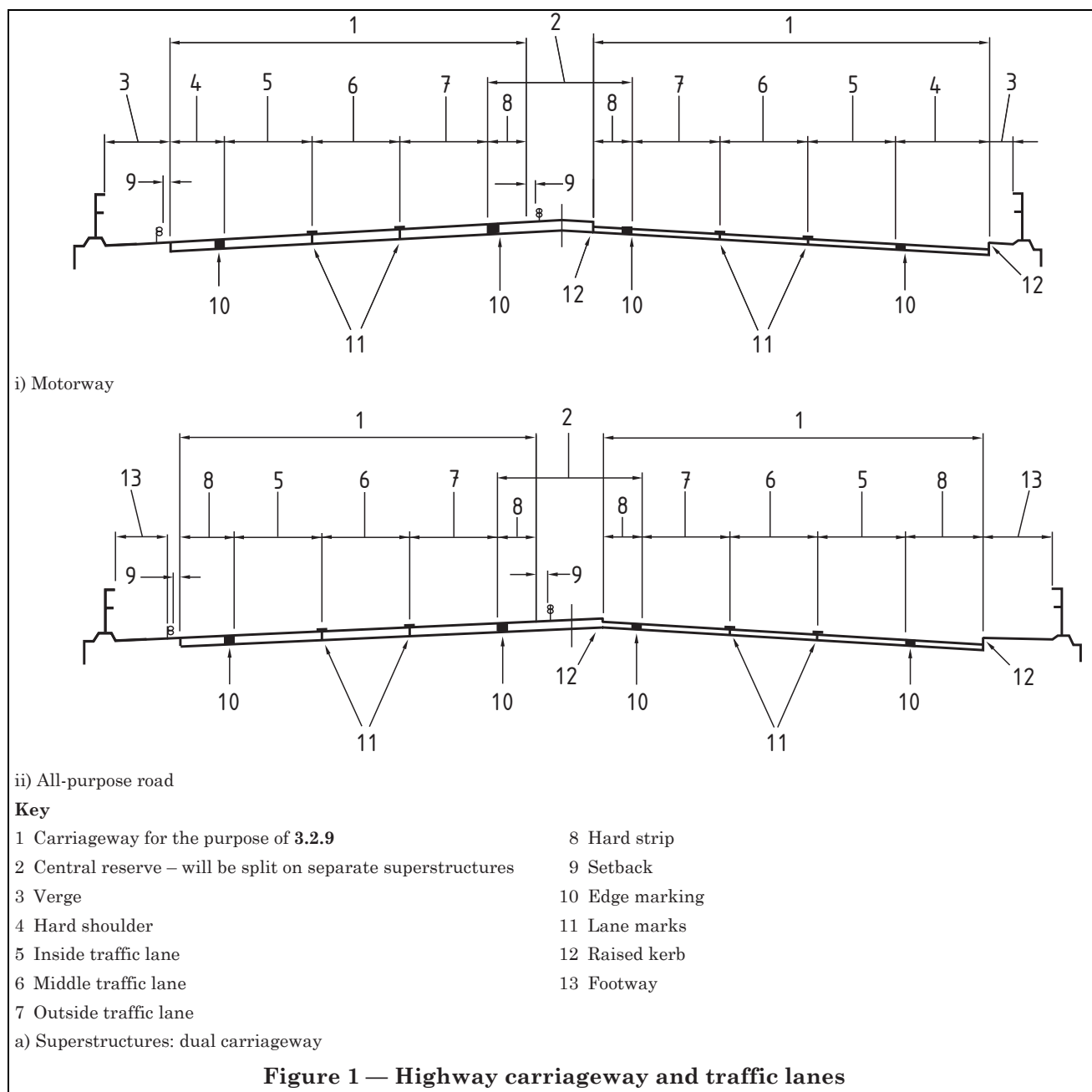
notional parts of the carriageway used solely for the purposes of applying the specified live loads, measured in a direction at right angles to the line of the raised kerbs, lane marking or edge marking

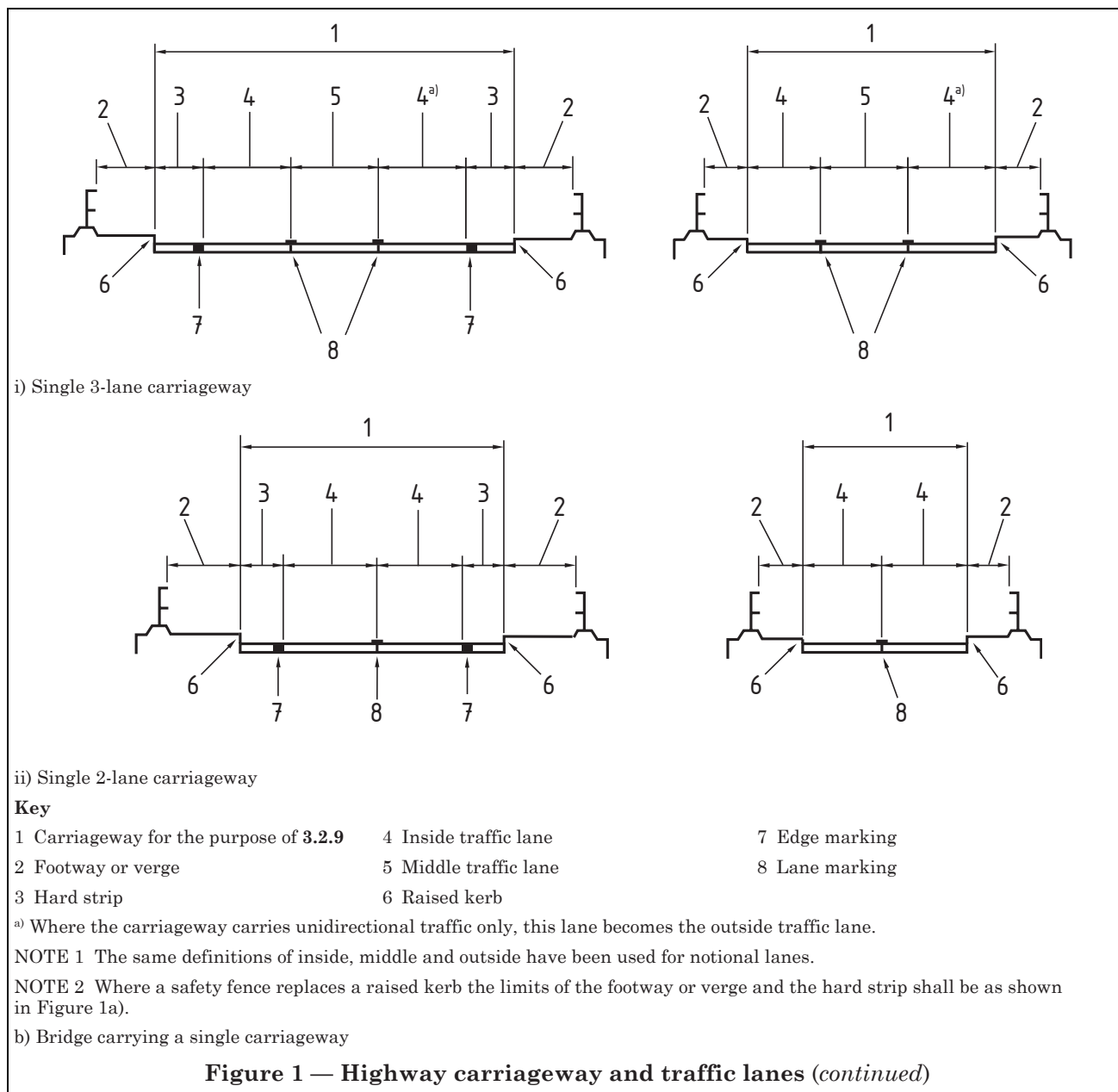
3.2.9.3.1

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





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_0	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_0	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:	restraint to movement, except frictional	ULS			1.30		
			SLS			1.00		
		frictional bearing restraint	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			
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 bearings¹⁾.

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 effect²⁾

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 $\gamma_{FL} = 1.0$ and $\gamma_{F3} = 1.0$) 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/m² 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.

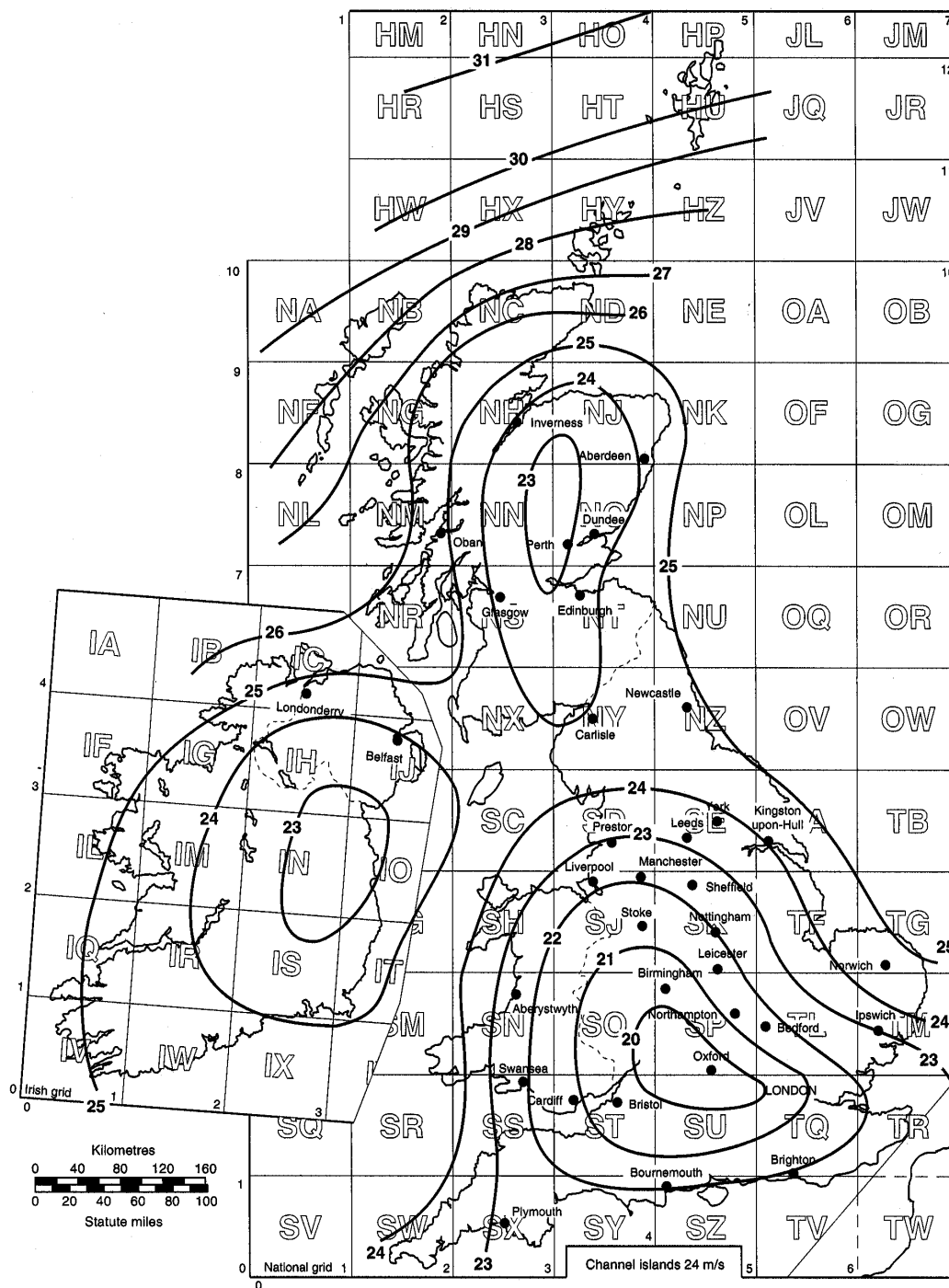


Figure 2 — Basic wind speed V_b in m/s (copyright BRE)

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:

- the ground level of the site when topography is not significant; or
- 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 $\phi = 0^\circ$ to $\phi = 330^\circ$ in 30° intervals. (The wind direction is defined in the conventional manner: an east wind is a wind direction of $\phi = 90^\circ$ 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 V_r for bridges without live load

V_r shall be taken as:

$$V_r = S_m V_s$$

where

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

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

$$S_m = S_c T_c S_h'$$

where

$$S_c = S_c' K_F$$

S_c' is the hourly speed factor (see 5.3.2.4.1)

K_F is the fetch correction factor (see 5.3.2.3.1)

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

S_h' is the topography factor (see 5.3.2.3.3 and Figure F.1).

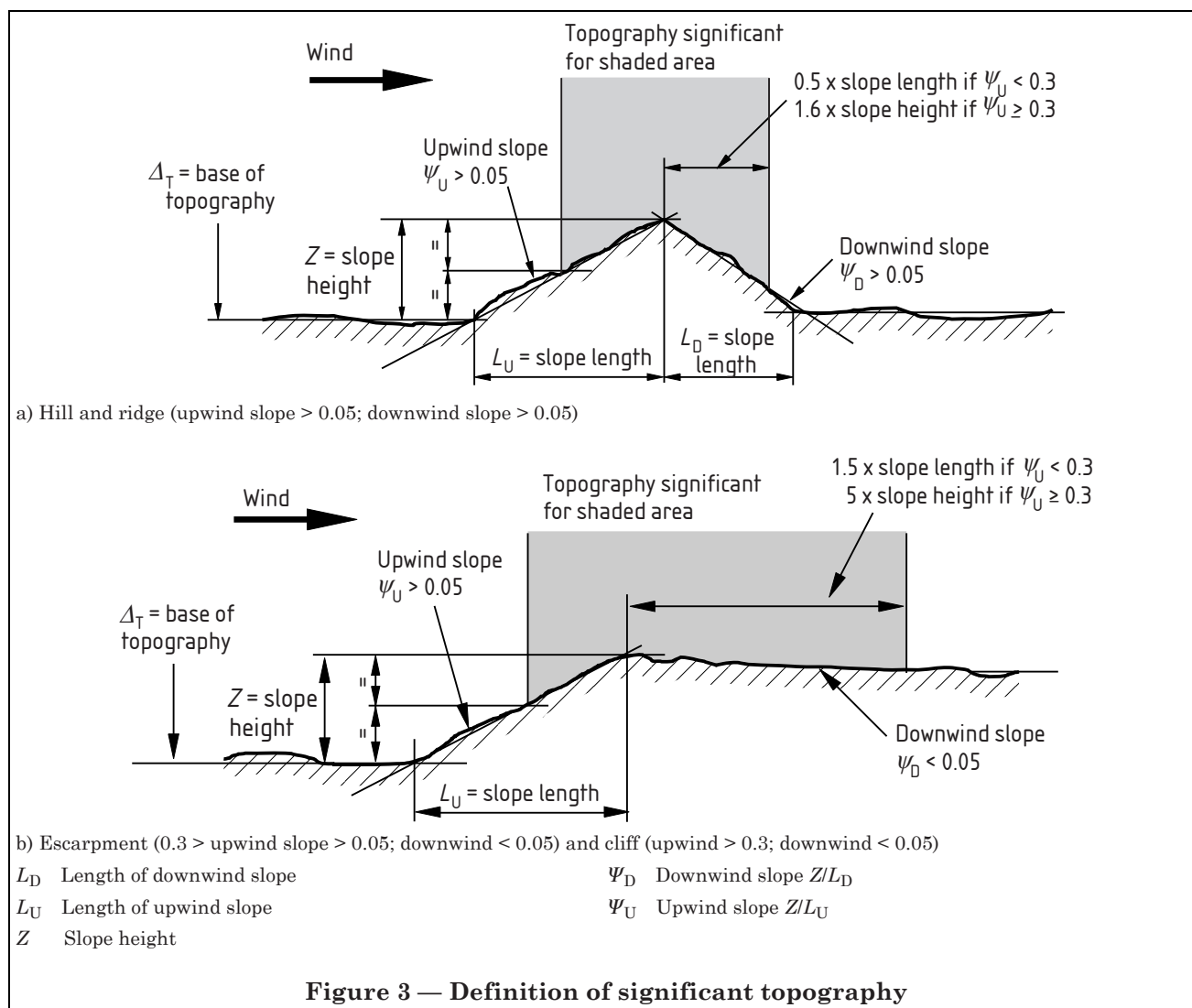


Table 3 — Values of terrain and bridge factor S_b' , hourly speed factor S_c' and fetch correction factor, K_F

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	1.0 throughout		
150			
200			
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:

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

- 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:

0.613 V_d^2 N/m² for those parts of the bridge on which the application of wind loading increases the effect being considered; or

0.613 V_r^2 N/m² for those parts where wind loading gives relief to the effect being considered.

A_1 is the solid area (in m²) (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 fences

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

5.3.3.1.6 Piers

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

5.3.3.2 Drag coefficient C_D for erection stages for beams and girders

In 5.3.3.2.1, 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 $1.5 \times C_D$ derived as specified in 5.3.3.2.1 for the single beam or box.

Where this ratio is greater than 7, C_D for 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.