British Standard

BRITISH STANDARD

BS 5400-2:
1978

Incorporating Amendment No. 1

Steel, concrete and composite bridges —

Part 2: Specification for loads

UDC 624.21.01:624.042

Cooperating organizations

The Steel and Concrete Bridges Standards Committee, under whose direction this British Standard was prepared, consists of representatives from the following Government departments and scientific, technical and professional organizations:

Association of Consulting Engineers
Association of County Councils
British Constructional Steelwork Association
British Precast Concrete Federation Ltd.
British Railways Board
British Steel Industry
Cement and Concrete Association
Concrete Society Limited
Constructional Steel Research and Development Organization
Department of the Environment (Building Research Establishment)
Department of the Environment (Transport and Road Research Laboratory)
Department of Transport
Federation of Civil Engineering Contractors
Greater London Council
Institution of Civil Engineers
Institution of Highway Engineers
Institution of Municipal Engineers
Institution of Structural Engineers
London Transport Executive
Ministry of Defence
Sand and Gravel Association Ltd.
Scottish Development Department
Welding Institute

This British Standard, having been prepared under the direction of the Steel and Concrete Bridges Standards Committee, was published under the authority of the Board of BSI on 30 June 1978

The following BSI references relate to the work on this standard:
Committee reference CSB/30
Draft for comment 73/13234

ISBN 0 580 09939 3

Amendments issued since publication

Amd. No.	Date of issue	Comments
4209	March 1983	Indicated by a sideline in the margin

Contents

Foreword

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

— *Part 1: General statement;*

— *Part 2: Specification 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.*

A British Standard does not purport to include all the necessary provisions of a contract. Users of British Standards are responsible for their 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 vi, pages 1 to 60, an inside back cover and a back cover.

This standard has been updated (see copyright date) and may have had amendments incorporated. This will be indicated in the amendment table on the inside front cover.

1 Scope

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

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

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

2 References

The titles of the standards publications referred to in this Part of BS 5400 are listed on the inside back cover.

3 Principles, definitions and symbols

3.1 Principles[1]

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

3.2 Definitions

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

3.2.1 loads

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

3.2.1.1
load effects

the stress resultants in the structure arising from its response to loads (as defined in **3.2.1**)

3.2.2
dead load

the weight of the materials and parts of the structure that are structural elements, but excluding superimposed materials such as road surfacing, rail track ballast, parapets, mains, ducts, miscellaneous furniture, etc

3.2.3
superimposed dead load

the 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

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

3.2.5
adverse and relieving areas and effects

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

3.2.6
total effects

the algebraic sum of the adverse and relieving effects

NOTE Where elements in a positive area of influence line are being considered the total effects may be negative, in which case the equivalent positive value will be the least negative effect, and where in negative effects are being considered the total effects may be positive, in which case the equivalent negative value will be the least positive effect. In either case the maximum negative or positive total effect should also be considered.

3.2.7
dispersal

the spread of load through surfacing, fill, etc

3.2.8
distribution

the sharing of load between directly loaded members and other members not directly loaded as a consequence of the stiffness of intervening connecting members, as e.g. 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

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

3.2.9.2
traffic lanes

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

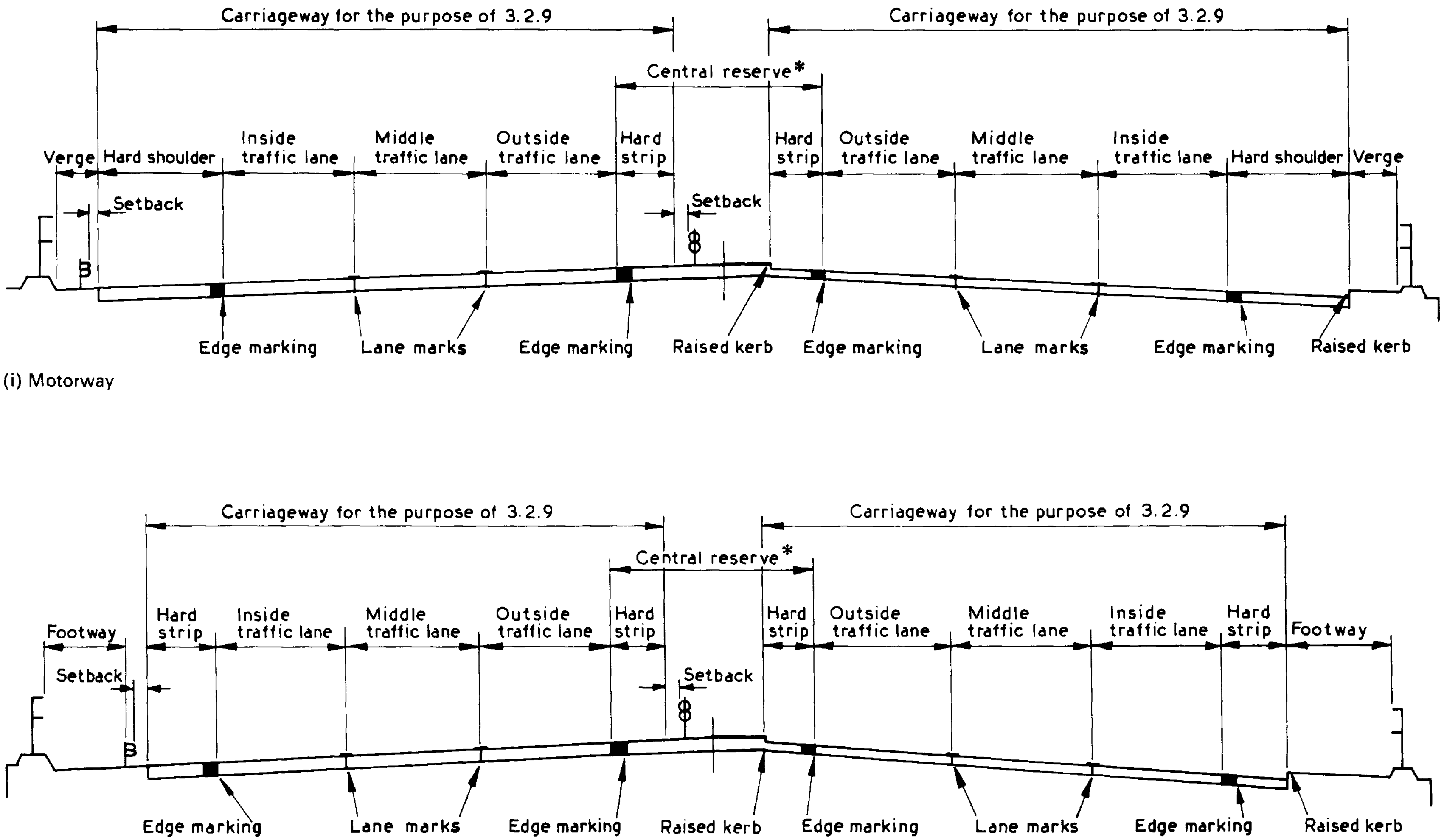
3.2.9.3 notional lanes

the notional parts of the carriageway used solely for the purpose of applying the specified live loads.

3.2.9.3.1
carriageway widths of 4.6 m or more

notional lanes shall be taken to be not less than 2.3 m nor more than 3.8 m wide. The carriageway shall be divided into the least possible integral number of notional lanes having equal widths as follows:

carriageway width m	**number of notional lanes**
4.6 up to and including 7.6	2
above 7.6 up to and including 11.4	3
above 11.4 up to and including 15.2	4
above 15.2 up to and including 19.0	5
above 19.0 up to and including 22.8	6



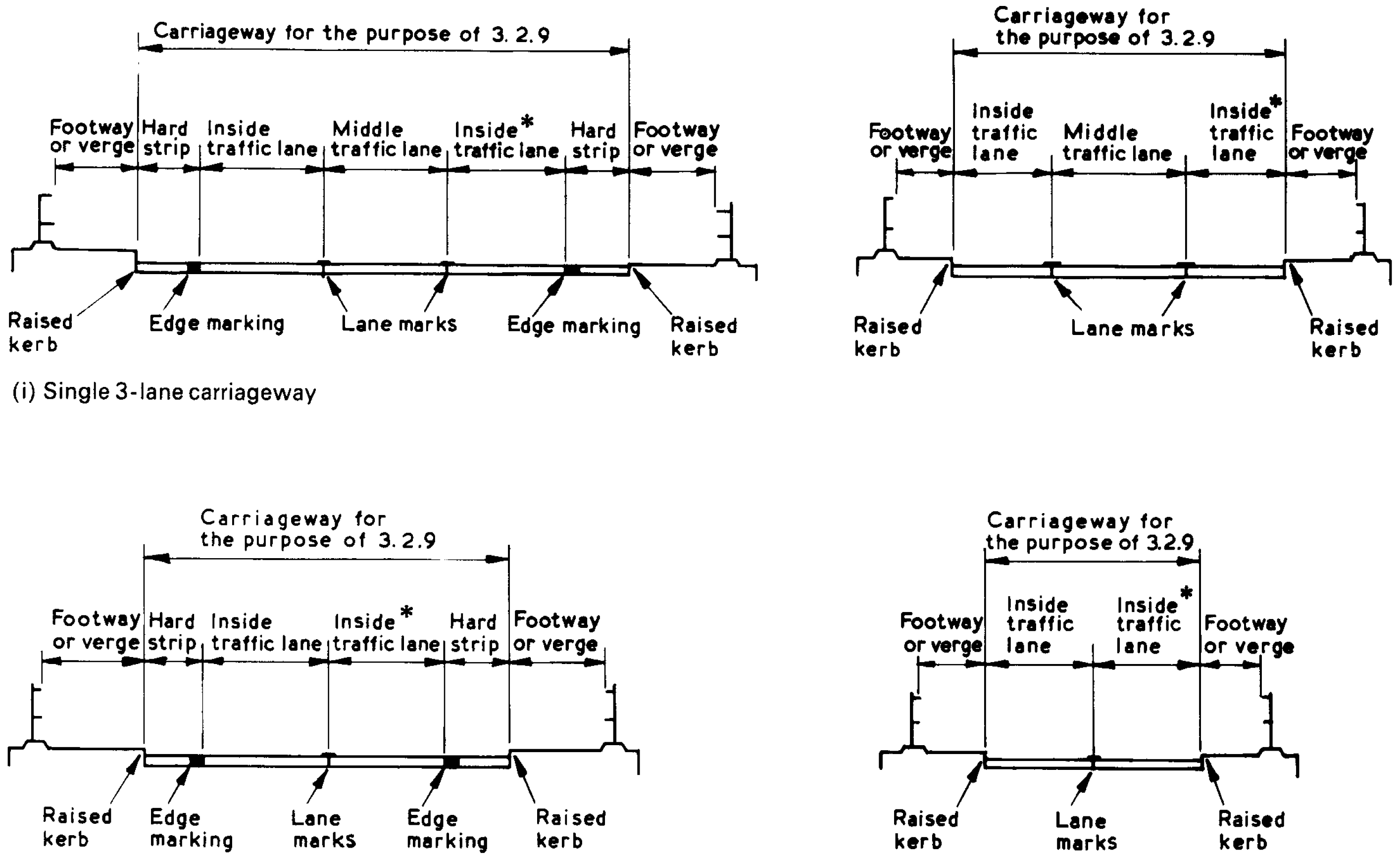
(i) Motorway

(ii) All-purpose road

(a) Superstructures : dual carriageway

*Central reserve will be split on separate superstructures

Figure 1 — Highway carriageway and traffic lanes



(ii) Single 2-lane carriageway

(b) Bridge carrying a single carriageway

* where the carriageway carries unidirectional traffic only, this lane becomes the outside traffic lane.

NOTE 1 The same definitions of inside, middle and outside have been used for notional lanes.

NOTE 2 where a safety fence replaces a raised kerb the limits of the footway or verge and the hard strip shall be as shown in Figure 1 (a)

Figure 1 — Highway carriageway and traffic lanes (concluded)

3.2.9.3.2
carriageway widths of less than 4.6 m
the carriageway shall be taken to have a number of notional lanes.

$$= \frac{\text{width of carriageway (in metres)}}{3.0}$$

Where the number of lanes is not an integer, the loading on the fractional part of a lane shall be taken pro rata the loading for one lane.

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

3.2.10 ***bridge components***

3.2.10.1
superstructure
in a bridge, that part of the structure which is supported by the piers and abutments.

3.2.10.2
substructure
in a bridge, the wing walls and the piers, towers and abutments that support the superstructure.

3.2.10.3
foundation
that part of the substructure in direct contact with, and transmitting load to, the ground.

3.3 Symbols

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

a	maximum vertical acceleration
A_1	solid area in normal projected elevation
A_2	see **5.3.4.6**
A_3	area in plan used to derive vertical wind load
b	width used in deriving wind load
c	spacing of plate girders used in deriving drag 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
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	depth (see Figure 9)
k	a constant used to derive primary live load on foot/cycle track bridges
K	configuration factor
K_1	a wind coefficient related to return period
K_2	hourly wind speed factor
l	main span
l_1	length of the outer spans of a three-span superstructure

L loaded length
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_1 funnelling factor
S_2 gust factor
t thickness of pier
v mean hourly wind speed
v_c maximum wind gust speed
v'_c minimum wind gust speed
v_t speed of highway or rail traffic
W load per metre of lane
Y_s static deflection
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
η shielding factor
ψ dynamic response factor
N number of axles (see Appendix D)
T { time in seconds (see **C.3**)
temperature difference (see Figure 9 and Appendix E)

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 appropriate value of γ_{fL} to derive the design load to be used in the calculation of moments, shears, total loads and other effects for each of the limit states under consideration. Values of γ_{fL} are given in each relevant clause and also in Table 1.

4.1.3 *Additional factor* γ_{f3}. Moments, shears, total loads and other effects of the design loads are also to be multiplied by γ_{f3} in certain circumstances (see **4.3.2** of Part 1 of this standard).

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 values of γ_{fL}, are given in Part 10 of this standard.

4.1.5 *Deflection and camber.* For the purposes of calculating deflection and camber the nominal loads shall be adopted (i.e. γ_{fL} shall be taken as unity).

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

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

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

4.4 Combinations of loads. Three principal and two secondary combinations of loads are specified; values of γ_{fL} for each load for each combination in which it is considered are given in the relevant clauses and also 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.*** Combination 4 does not apply to railway bridges except for vehicle collision loading on bridge supports. For highway bridges, the loads to be considered are the permanent loads and the secondary live loads, together with the appropriate primary live loads associated with them. Secondary live loads shall be considered separately and are not required to be combined. Each shall be taken with its appropriate associated primary live load.

For foot/cycle track bridges, the only secondary live load to be considered is the vehicle collision load with bridge supports (see **6.9**).

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[2)].

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

4.5.1 ***Selection to cause most adverse effect***[3)]. Design loads shall be selected and applied in such a way that the most adverse total effect is caused in the element or structure under consideration.

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

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

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

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

---

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

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

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

ULS: ultimate limit state
SLS: serviceability limit state

Clause number	Load	Limit state	γ_{fL} to be considered in combination 1	2	3	4	5
5.1	**Dead:** steel	ULS[a] SLS	1.05 1.00	1.05 1.00	1.05 1.00	1.05 1.00	1.05 1.00
	concrete	ULS[a] SLS	1.15 1.00	1.15 1.00	1.15 1.00	1.15 1.00	1.15 1.00
5.2	**Superimposed dead**	ULS[b] SLS[b]	1.75 1.20	1.75 1.20	1.75 1.20	1.75 1.20	1.75 1.20
5.1.2.2 & **5.2.2.2**	Reduced load factor for dead and superimposed dead load where this has a more severe total effect	ULS	1.00	1.00	1.00	1.00	1.00
5.3	**Wind:** during erection	ULS SLS		1.10 1.00			
	with dead plus superimposed dead load only, and for members primarily resisting wind loads	ULS SLS		1.40 1.00			
	with dead plus superimposed dead plus other appropriate combination 2 loads	ULS SLS		1.10 1.00			
	relieving effect of wind	ULS SLS		1.00 1.00			
5.4	**Temperature: restraint to movement, except frictional**	ULS SLS			1.30 1.00		
	frictional restraint	ULS SLS					1.30 1.00
	effect of temperature difference	ULS SLS			1.00 0.80		
5.6	**Differential settlement**	ULS SLS	to be assessed and agreed between the engineer and the appropriate authority				
5.7	**Exceptional loads**						
5.8	**Earth pressure:** retained fill and/or live load surcharge	ULS SLS	1.50 1.00	1.50 1.00	1.50 1.00	1.50 1.00	1.50 1.00
	relieving effect	ULS	1.00	1.00	1.00	1.00	1.00
5.9	**Erection:** temporary loads	ULS		1.15	1.15		
6.2	**Highway bridges live loading:** HA alone	ULS SLS	1.50 1.20	1.25 1.00	1.25 1.00		
6.3	HA with HB or HB alone	ULS SLS	1.30 1.10	1.10 1.00	1.10 1.00		
6.5	**Centrifugal load and associated primary live load**	ULS SLS (each secondary live shall be considered separately together with the other combination 4 loads as appropriate)				1.50 1.00	
6.6	**Longitudinal load:** HA and associated primary live load	ULS SLS				1.25 1.00	
	HB and associated primary live load	ULS SLS				1.10 1.00	
6.7	**Accidental skidding load and associated primary live load**	ULS SLS				1.25 1.00	
6.8	**Vehicle collision load with bridge parapets and associated primary live load**	ULS SLS				1.25 1.00	
6.9	**Vehicle collision load with bridge supports**[c]	ULS SLS				1.25 1.00	
7	**Foot/cycle track bridges:** live load and parapet load	ULS SLS	1.50 1.00	1.25 1.00	1.25 1.00	1.25 1.00	
8	**Railway bridges:** type RU and RL primary and secondary live loading	ULS SLS	1.40 1.10	1.20 1.00	1.20 1.00		

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.

[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 appropriate authority (see **5.2.2.1**)

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

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 × 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 shall be based on the principles set out in CP 2004.

4.7.1 ***Design loads to be considered with CP 2004.*** CP 2004[4)] 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 CP 2004.

5 Loads applicable to all bridges

5.1 Dead load

5.1.1 ***Nominal dead load.*** 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 toad 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 of 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.

5.2 Superimposed dead load

5.2.1 ***Nominal superimposed dead load.*** 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.

[4)] In course of revision.

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
1.75	1.20

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

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

5.2.2.2 *Alternative load factor*. Where the structure or element under consideration is such that the application of γ_{fL} as 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.

5.3 Wind load[5)]

5.3.1 *General.* The wind pressure on a bridge depends on the geographical 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. Design gust pressures are derived from the isotachs of mean hourly wind speed shown in Figure 2. These wind speeds are appropriate to a height above ground level of 10 m in open level country and a 120-year return period[6)].

For the British Isles at sites less than 300 m above sea level the wind gust speed shall be derived in accordance with **5.3.2**. At greater altitudes these wind speeds will be exceeded and a special local study will be required.

5.3.2 *Wind gust speed*

5.3.2.1 *Maximum wind gust speed v_c on bridges without live load*. The maximum wind gust speed-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:

$$v_c = vK_1S_1S_2$$

where

v is the mean hourly wind speed (see **5.3.2.1.1**)

K_1 is a wind coefficient related to the return period (see **5.3.2.1.2**)

S_1 is the funnelling factor (see **5.3.2.1.3**)

S_2 is the gust factor (see **5.3.2.1.4** and **5.3.2.1.5**)

For the remaining parts of the bridge or element on which the application of wind loading gives relief to the effects under consideration, a reduced wind gust speed shall be derived as specified in **5.3.2.2**.

5.3.2.1.1 *Mean hourly wind speed v*. Values of v in m/s for the location of the bridge shall be obtained from the map of isotachs shown in Figure 2.

5.3.2.1.2 *Coefficient K_1*. The coefficient shall be taken as 1.0 for highway, railway and foot/cycle track bridges for a return period of 120 years.

---

5) The wind loads given in this Part of BS 5400 have been derived from general wind tunnel tests and can therefore be conservative. If wind loads have a considerable effect on any structure or part of a structure it may be advantageous to derive data from wind tunnel tests.

6) Wind loading will not be significant in its effect on a large proportion of bridges, as e.g. concrete slab or slab and beam structures 20 m or less in span, 10 m or more in width and at normal heights above ground.

In general, a suitable check for 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.

For foot/cycle track bridges, subject to the agreement of the appropriate authority, a return period of 50 years may be adopted and K_1 shall be taken as 0.94.

During erection, the value of K_1 may be taken as 0.85, corresponding to a return period of 10 years. Where a particular erection will be completed in 2 days or less, and for which reliable wind speed forecasts are available, this predicted wind speed may be used as the mean hourly wind speed v, in which case the value of K_1 shall be taken as 1.0.

5.3.2.1.3 *Funnelling factor S_1.* In general the funnelling factor shall 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.

5.3.2.1.4 *Gust factor S_2.* Values of S_2 are given in Table 2. These are valid for sites up to 300 m above sea level.

Table 2 — Values of gust factor S_2 and hourly speed factor K_2

Height above ground level	Horizontal wind loaded length m									Hourly speed factor K_2
	20 or less	40	60	100	200	400	600	1 000	2 000	
m										
5	1.47	1.43	1.40	1.35	1.27	1.19	1.15	1.10	1.06	0.89
10	1.56	1.53	1.49	1.45	1.37	1.29	1.25	1.21	1.16	1.00
15	1.62	1.59	1.56	1.51	1.43	1.35	1.31	1.27	1.23	1.07
20	1.66	1.63	1.60	1.56	1.48	1.40	1.36	1.32	1.28	1.13
30	1.73	1.70	1.67	1.63	1.56	1.48	1.44	1.40	1.35	1.21
40	1.77	1.74	1.72	1.68	1.61	1.54	1.50	1.46	1.41	1.27
50	1.81	1.78	1.76	1.72	1.66	1.59	1.55	1.51	1.46	1.32
60	1.84	1.81	1.79	1.76	1.69	1.62	1.58	1.54	1.50	1.36
80	1.88	1.86	1.84	1.81	1.74	1.68	1.64	1.60	1.56	1.42
100	1.92	1.90	1.88	1.84	1.78	1.72	1.68	1.65	1.60	1.48
150	1.99	1.97	1.95	1.92	1.86	1.80	1.77	1.74	1.70	1.59
200	2.04	2.02	2.01	1.98	1.92	1.87	1.84	1.80	1.77	1.66

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 wind gust speed 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 a gust speed as specified in **5.3.2.2** for bridges without live load and in **5.3.2.4** for bridges with live load.

NOTE 2 Where the bridge is located at or near the top of a cliff or a steep escarpment, the height above ground level shall be measured from the foot of such features. For bridges over tidal waters, the height above ground shall be measured from the mean water level.

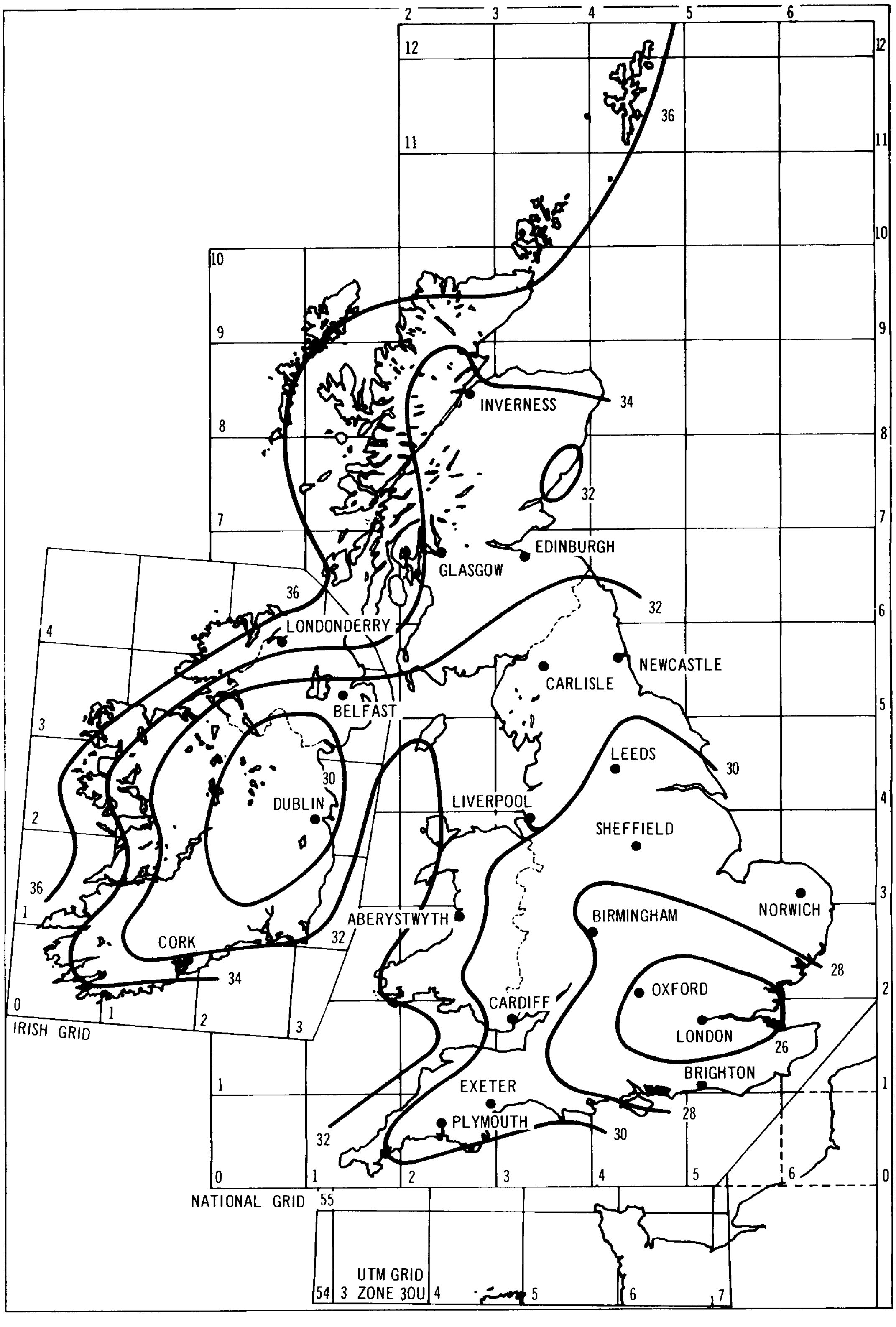
NOTE 3 The height of vertical elements such as piers and towers shall be divided into units in accordance with the heights given in column 1 of Table 2, and the gust factor and maximum wind gust speed shall be derived for the centroid of each unit.

5.3.2.1.5 *Reduction factor for foot/cycle track bridges.* The values of gust factor S_2 given in Table 2 are for an exposed rural situation and take no account of the variation in ground roughness around a bridge. The wind gust speeds so derived can therefore be unduly severe on wind sensitive structures located in an environment where there are many windbreaks.

For foot/cycle track bridges located in an urban or rural environment with many windbreaks of general height at least 10 m above ground level, the values of S_2 and K_2 specified in **5.3.2.1.4** may be multiplied by a reduction factor derived from Table 3. For bridges more than 20 m above ground level, no reduction shall be made.

Table 3 — Reduction factor for ground roughness

Height above ground level	Reduction factor
m	
5	0.75
10	0.80
15	0.85
20	0.90



NOTE The isotachs are derived from Meteorological Office data.

Figure 2 — Isotachs of mean hourly wind speed (in m/s)

5.3.2.2 *Minimum wind gust speed v'_c on relieving areas of bridges without live load.* Where wind on any part of a bridge or element gives relief to the member under consideration, the effective coexistent value of minimum wind gust speed v'_c on the parts affording relief shall be taken as:

$v'_c = vK_1 K_2$

where v' and K_1 are as derived in **5.3.2.1.1** and **5.3.2.1.2**, respectively, and K_2 is the hourly speed factor as given in Table 2, modified where appropriate, in accordance with **5.3.2.1.5**.

5.3.2.3 *Maximum wind gust speed v_c on bridges with live load.* The maximum wind gust speed 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:

for highway and foot/cycle track bridges, as specified in **5.3.2.1** to **5.3.2.1.5** inclusive, but not exceeding 35 m/s;

for railway bridges, as specified in **5.3.2.1** to **5.3.2.1.5** inclusive.

5.3.2.4 *Minimum wind gust speed v'_c on relieving areas of 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'_c on the parts affording relief shall be taken as:

for highway and foot/cycle track bridges, the lesser of

$35 \times \dfrac{K_2}{S_2}$ m/s and vK_1K_2 m/s;

for railway bridges, vK_1K_2 m/s

where v, K_1, K_2 and S_2 are as derived in **5.3.2.1.1** to **5.3.2.1.5**.

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:

$P_t = qA_1C_D$

where

q is the dynamic pressure head ($= 0.613 v^2_c$ in N/m^2, with v_c in m/s)
A_1 is the solid area (in m^2) (see **5.3.3.1**)
C_D is the drag coefficient (see **5.3.3.2** to **5.3.3.6**).

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

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

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

a) *Superstructures without five 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 Table 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.

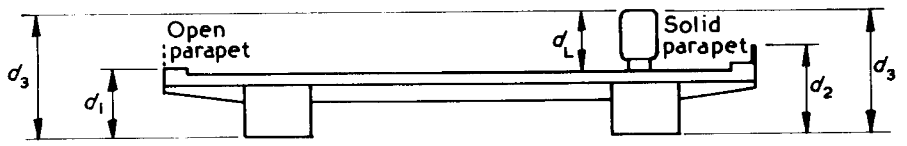
2) For superstructures with solid parapets; the superstructure, using depth d_2 from Table 4 which includes the effects of the windward and leeward parapets. Where there are safety fences or additional parapets, P_t shall be derived separately for the solid areas of the elements above the top of the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

b) *Superstructures with live load.* P_t shall be derived for the area A_1 as given in Table 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.

Table 4 — Depth d to be used in deriving area A_1



Parapet	Unloaded bridge	Live loaded bridge
Open	$d = d_1$	$d = d_3$
Solid	$d = d_2$	$d = d_2$ or d_3 whichever is greater
d_L = 2.5 m above the highway carriageway, or 3.7 m above the rail level, or 1.25 m above footway or cycle track		

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 Table 5 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 Table 5 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 Table 4.

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

1) the windward and leeward truss girders;

2) the deck;

3) the windward and leeward parapets;

4) the live load depth;

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

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

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

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

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

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

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

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

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

5.3.3.2 *Drag coefficient C_D for erection stages for beams and girders.* In **5.3.3.2.1** to **5.3.3.2.5** requirements are specified for discrete beams or girders before deck construction or other infilling (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 girders.* C_D for each beam or box shall be derived from Figure 5 without any allowance for shielding. Where the combined beams or boxes are required to be considered, C_D shall be derived as follows.

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

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

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

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

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

5.3.3.3 *Drag coefficient C_D for all superstructures with solid elevation (see Figure 3).* 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 Table 5. Where designs are not in accordance with Table 5, and for those types of superstructure illustrated in Figure 4, drag coefficients shall be ascertained from wind tunnel tests.

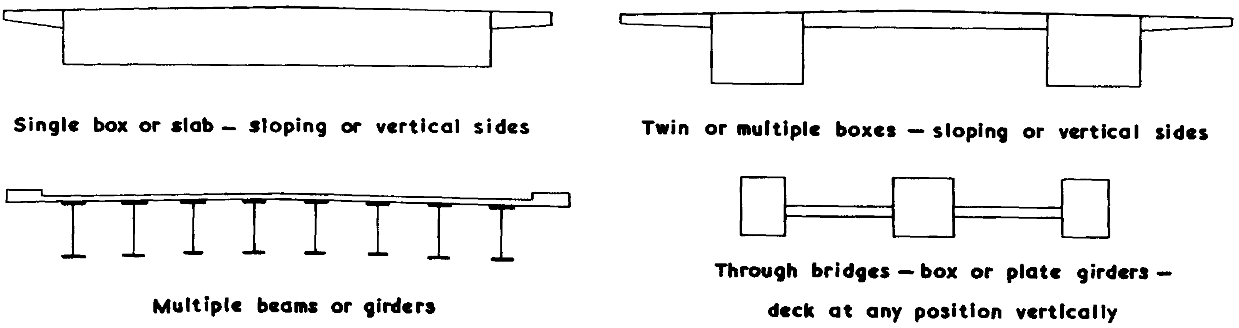


Figure 3 — Typical superstructures to which Figure 5 applies

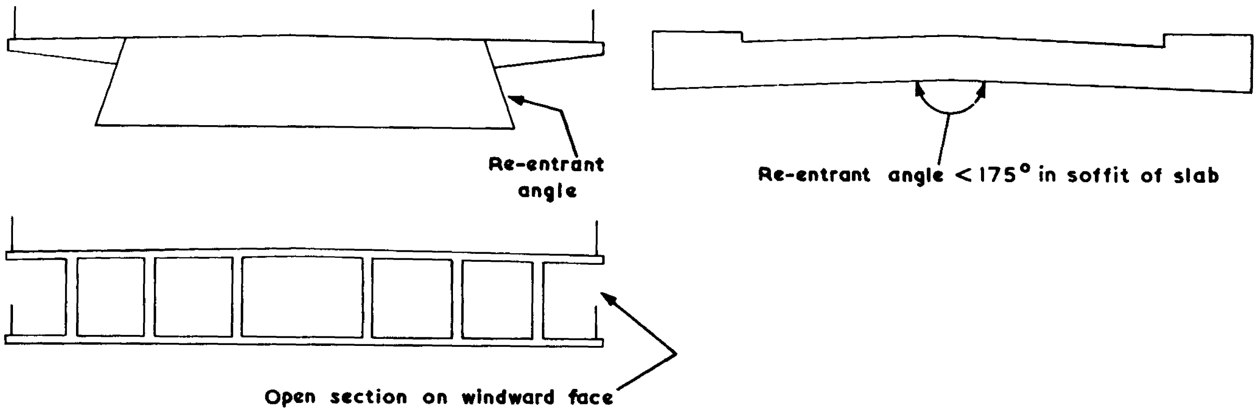
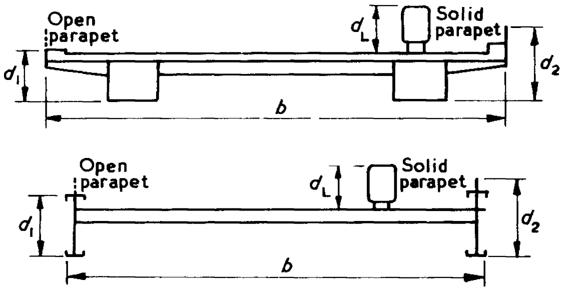
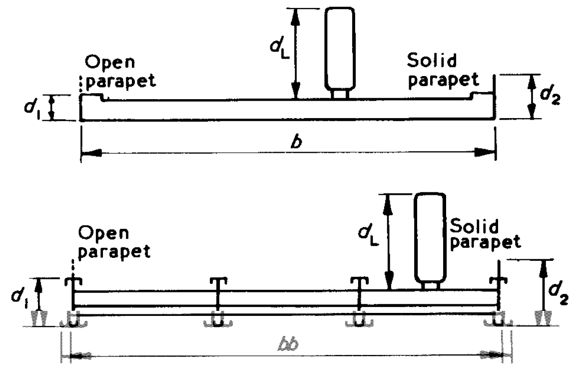


Figure 4 — Typical superstructures that require wind tunnel tests

Table 5 — Depth d to be used in deriving C_D

<table>
<tr><th></th><th>Parapet</th><th>Superstructures without live load</th><th>Superstructures with live load</th></tr>
<tr><td rowspan="2">(a) Superstructures where the depth of the superstructure (d_1 or d_2) exceeds d_L
</td><td>Open</td><td>$d = d_1$</td><td>$d = d_1$</td></tr>
<tr><td>Solid</td><td>$d = d_2$</td><td>$d = d_2$</td></tr>
<tr><td rowspan="2">(b) Superstructures where the depth of the superstructure (d_1 or d_2) is less than d_L.
</td><td>Open</td><td>$d = d_1$</td><td>$d = d_L$</td></tr>
<tr><td>Solid</td><td>$d = d_2$</td><td>$d = d_L$</td></tr>
</table>

5.3.3.4 *Drag coefficient C_D for all truss girder superstructures*

a) *Superstructures without five 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	
		$dv_c < 6$ m²/s or dv'_c	$dv_c \geq 6$ m²/s or dv'_c
0.1	1.9	1.2	0.7
0.2	1.8	1.2	0.8
0.3	1.7	1.2	0.8
0.4	1.7	1.1	0.8
0.5	1.6	1.1	0.8

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

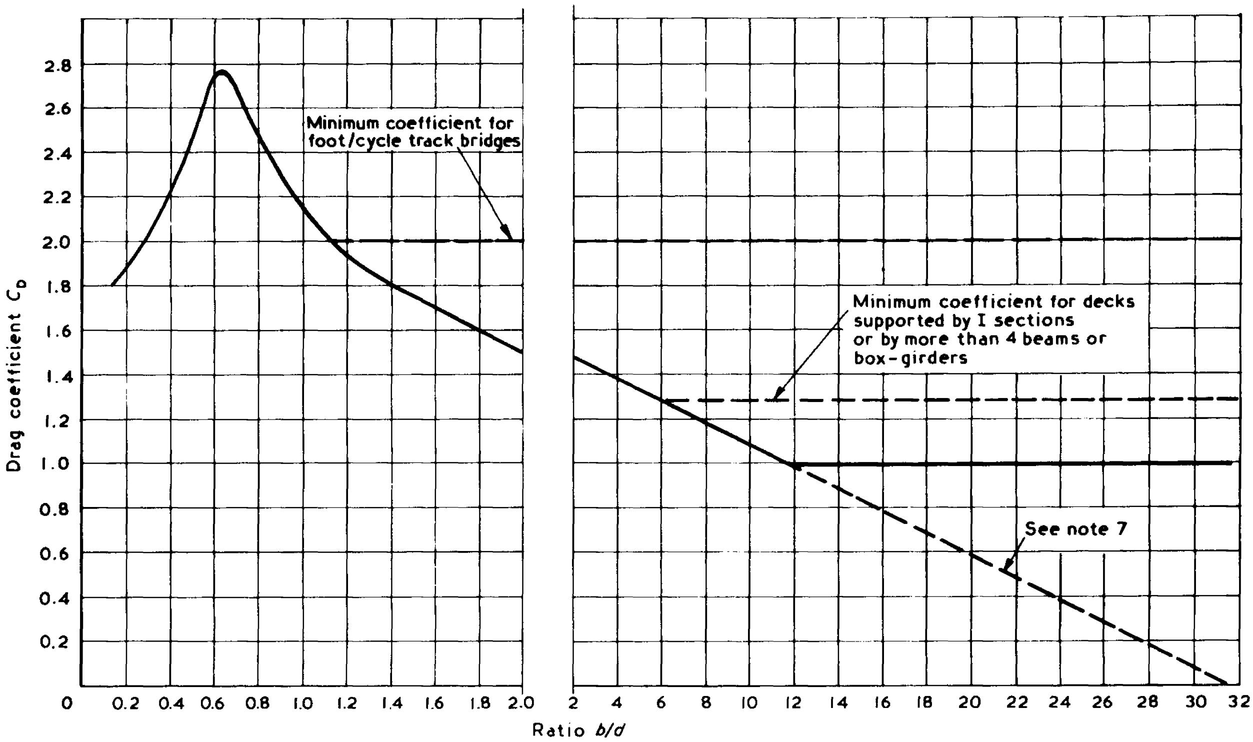
Spacing ratio	Value of η for solidity ratio of:				
	0.1	0.2	0.3	0.4	0.5
Less then 1	1.0	0.90	0.80	0.60	0.45
2	1.0	0.90	0.80	0.65	0.50
3	1.0	0.95	0.80	0.70	0.55
4	1.0	0.95	0.85	0.70	0.60
5	1.0	0.95	0.85	0.75	0.65
6	1.0	0.95	0.90	0.80	0.70

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.



NOTES to Figure 5

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

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

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

For each part of the face, the depth shall be taken as the total vertical depth of the face (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.

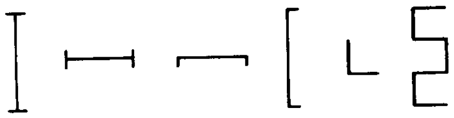
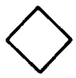
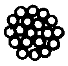
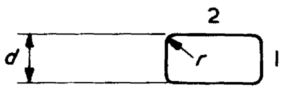
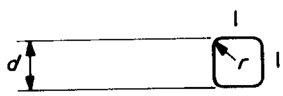
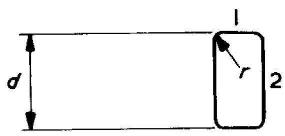
NOTE 6 Where the superstructure is superelevated and also subject to inclined wind, the drag coefficient C_D shall be specially investigated.

NOTE 7 Where two generally similar superstructures are separated transversely by a gap not exceeding 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 of the leeward superstructure shall be the difference between that of the combined superstructure and that of the windward superstructure. For the purposes of determining this distribution, if b/d is greater than 12 the broken line in Figure 5 shall be used to derive C_D. The load on the leeward structure is generally opposite in sign to that on the windward superstructure.

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

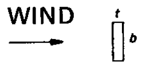
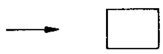
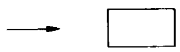
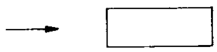
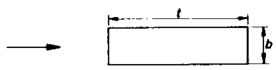
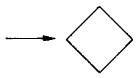
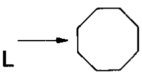
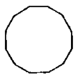
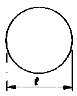
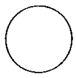
Figure 5 — Drag coefficient C_D for superstructures with solid elevation

Table 8 — Drag coefficient C_D for parapets and safety fences

	Circular section $dv_c < 6$	1.2
(where v_c is in m/s and d is in m)	$dv_c \geqslant 6$	0.7
NOTE On relieving areas use v'_c instead of v_c		
	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]
	Rectangular 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

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

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

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

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

NOTE 4 For a pier tapering with height, C_D shall be derived for each of the unit heights into which the support has been subdivided (see **5.3.2.1.4**). 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.

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

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

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

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

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

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

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

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

5.3.4.1 *All superstructures with solid elevation*

$$P_{Ls} = 0.25qA_1 C_D$$

where

q is as defined in **5.3.3**, the appropriate value of v_c 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_c 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 **5.3.2.4** a), ηC_D being adopted where appropriate

5.3.4.3 *Live load on all superstructures*

$$P_{LL} = 0.5qA_1 C_D$$

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 Table 4 and the appropriate horizontal wind loaded length as defined in the note to Table 2

$C_D = 1.45$.

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 area 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 as derived from Figure 6 for superstructures where the angle of superelevation is less than 1°

Where the angle of superelevation of a superstructure is between 1° and 5°, C_L shall be taken as ± 0.75.

Where the angle of superelevation of a superstructure exceeds 5°, the value of C_L shall be determined by testing.

Where inclined wind may affect the structure, C_L shall be taken as ± 0.75 for wind inclinations up to 5°. The angle of inclination in these circumstances shall be taken as the sum of the angle of inclination of the wind and that of the superelevation of the bridge. The effects of wind inclinations in excess of 5° shall be investigated by testing.

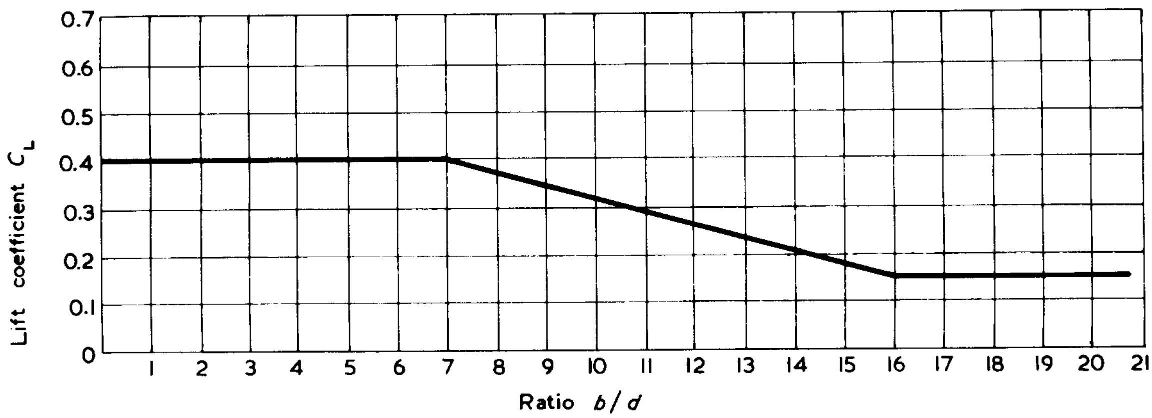


Figure 6 — Lift coefficient C_L

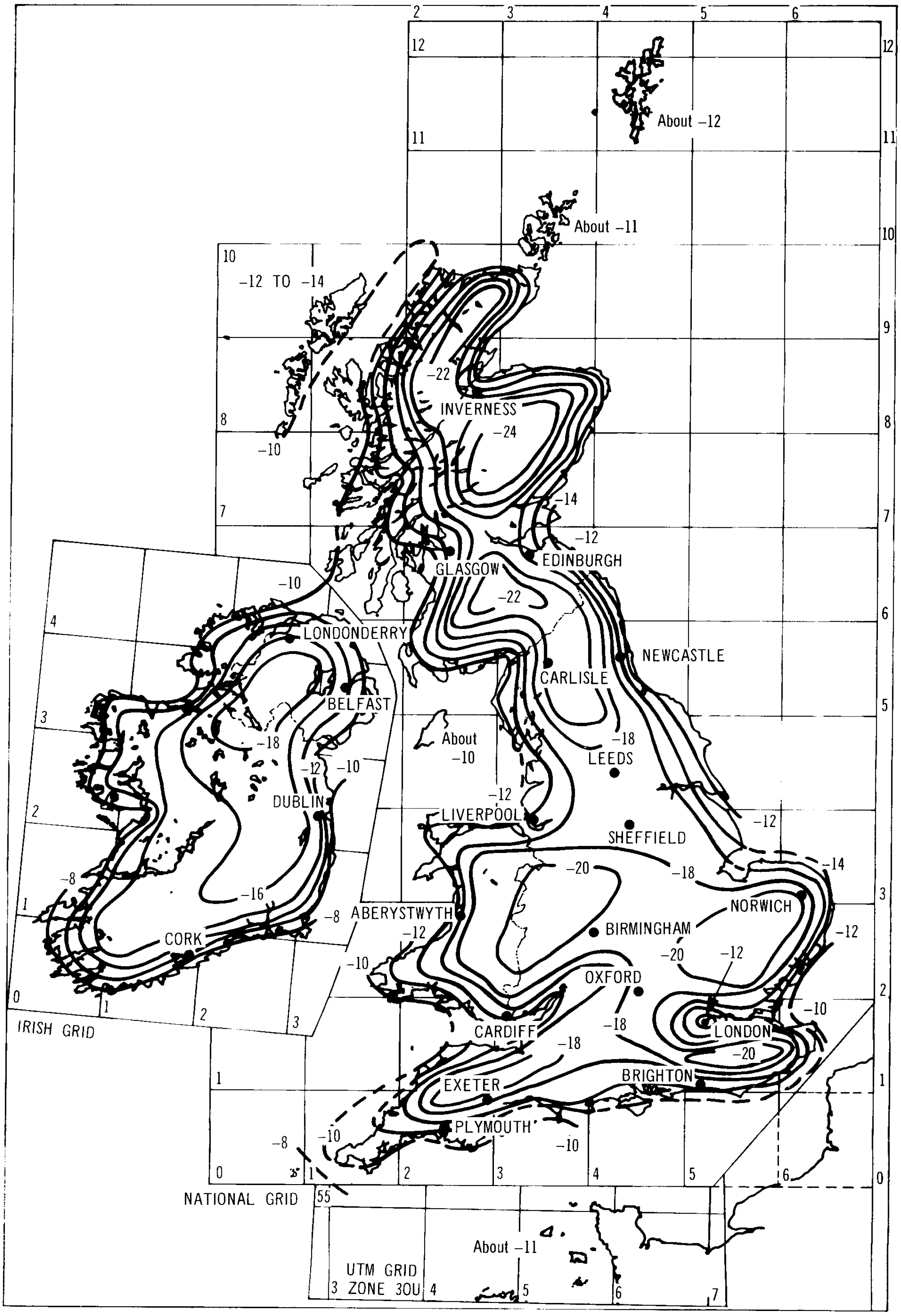
5.3.6 ***Load combination.*** The wind loads P_t, P_L and P_v shall be considered in combination with the other loads in combination 2, as appropriate, taking four separate cases:

a) P_t alone;

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

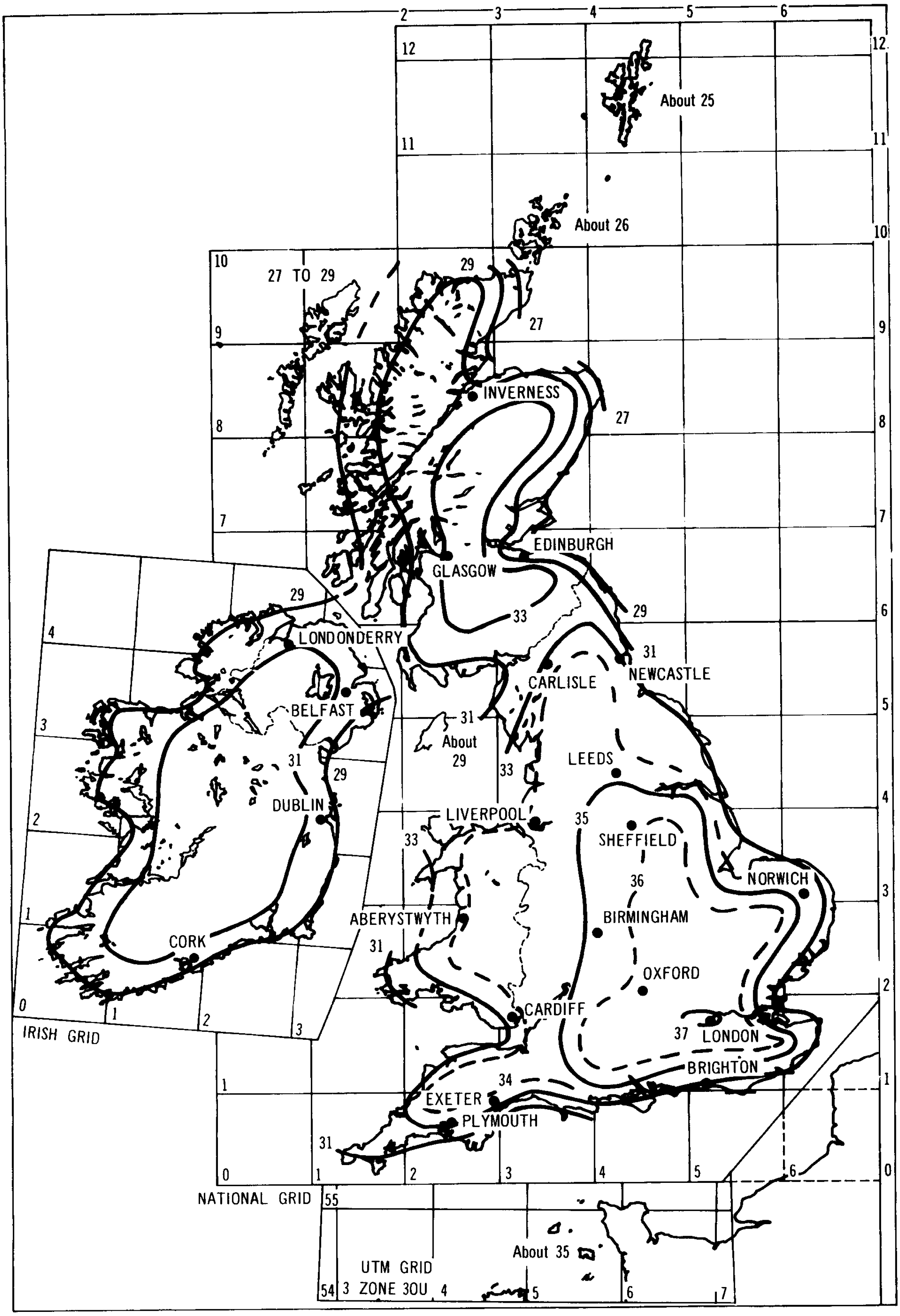
c) P_L alone;

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



NOTE The isotherms are derived from Meteorological Office data.

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



NOTE The isotherms are derived from Meteorological Office data.

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

5.3.7 ***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 states and serviceability limit states shall be taken as 1.0.

5.3.9 ***Aerodynamic effects.*** Consideration shall be given to wind excited oscillations, and where necessary this behaviour shall be determined by tests.

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 overall temperature of the bridge, referred to as the effective bridge temperature. Over a prescribed period, there will be a minimum and a maximum, together with a range of effective bridge temperatures, resulting in loads and/or load effects within the bridge 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 through the depth of the superstructure, referred to as temperature difference and resulting in associated loads and/or load effects within the structure.

Effective bridge temperatures 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.

5.4.2 ***Minimum and maximum shade air temperatures.*** For all bridges, extremes of shade air temperature 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 appropriate authority, a return period of 50 years may be adopted, and the shade air temperatures may be reduced as specified in **5.4.2.1**.

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

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

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

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

5.4.2.2 *Adjustment for height above mean sea level.* The values of shade air temperature shall be adjusted for height above sea level by subtracting 0.5 °C per 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 maximum effective bridge temperatures.*** The minimum and maximum effective bridge temperatures for different types of construction shall be derived from the minimum and maximum shade air temperatures by reference to Table 10 and Table 11 respectively. The different types of construction are as shown in Figure 9.

Table 10 — Minimum effective bridge temperature

Minimum shade air temperature	Minimum effective bridge temperature		
	Type of superstructure		
	Groups 1 & 2	Group 3	Group 4
°C	°C	°C	°C
– 24	– 28	– 19	– 14
– 23	– 27	– 18	– 13
– 22	– 26	– 18	– 13
– 21	– 25	– 17	– 12
– 20	– 23	– 17	– 12
– 19	– 22	– 16	– 11
– 18	– 21	– 15	– 11
– 17	– 20	– 15	– 10
– 16	– 19	– 14	– 10
– 15	– 18	– 13	– 9
– 14	– 17	– 12	– 9
– 13	– 16	– 11	– 8
– 12	– 15	– 10	– 7
– 11	– 14	– 10	– 6
– 10	– 12	– 9	– 6
– 9	– 11	– 8	– 5
– 8	– 10	– 7	– 4
– 7	– 9	– 6	– 3
– 6	– 8	– 5	– 3
– 5	– 7	– 4	– 2

Table 11 — Maximum effective bridge temperature

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

NOTE See Figure 9 for different types of superstructure.

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.

Table 12 — Adjustment to effective bridge temperature for deck surfacing

	Addition to minimum effective bridge temperature			Addition to maximum effective bridge temperature		
Deck Surface	Groups 1 & 2	Group 3	Group 4	Groups 1 & 2	Group 3	Group 4
	°C	°C	°C	°C	°C	°C
Unsurfaced	0	– 3	– 1	+ 4	0	0
Waterproofed	0	– 3	– 1	+ 2	+ 4	+ 2
40 mm surfacing	0	– 2	– 1	0	+ 2	+ 1
100 mm surfacing[a]	—	0	0	—	0	0
200 mm surfacing[a]	—	+ 3	+ 1	—	– 4	– 2

[a] Surfacing depths include waterproofing.

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 reradiation and other effects.

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

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

5.4.6 ***Coefficient of thermal expansion.*** For the purpose of calculating temperature effects, the coefficient 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 7×10^{-6}/°C shall be adopted for the concrete.

5.4.7 ***Nominal values***

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

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

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

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

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

The nominal load shall be determined in accordance with **5.14.2.6** of BS 5400-9.1:1983.

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 Table 2 and Table 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 need be provided for.

5.4.8.2 *Design load for temperature restraint.* For combination 3, $\gamma_{f L}$ shall be taken as follows.

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

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

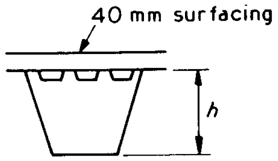
For the ultimate limit state	For the serviceability limit state
1.30	1.00

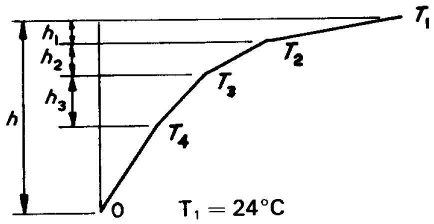
5.4.8.3.1 *Associated vertical design load.* The design dead load (see **5.1.2**) and design superimposed dead load (see **5.2.2.**) shall be considered in conjunction with the design load due to frictional bearing restraint.

Group	Type of construction	Temperature difference °C

Positive temperature difference — **Reverse temperature difference**

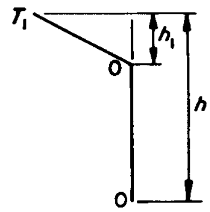
1. Steel deck on steel box girders





$h_1 = 0.1$ m; $h_2 = 0.2$ m; $h_3 = 0.3$ m

$T_1 = 24$°C; $T_2 = 14$°C; $T_3 = 8$°C; $T_4 = 4$°C

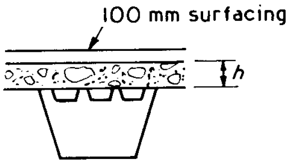


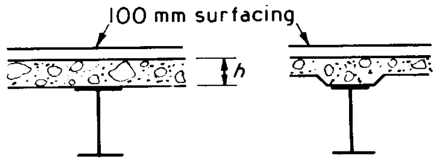
$T_1 = 6$°C; $h_1 = 0.5$ m

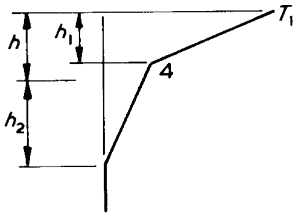
2. Steel deck on steel truss or plate girders

Use differences as for group 1

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



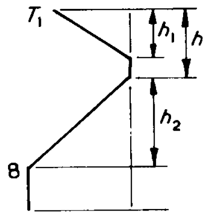




$h_1 = 0.6\,h$

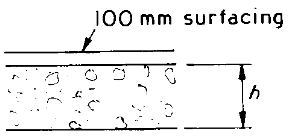
$h_2 = 0.4$ m

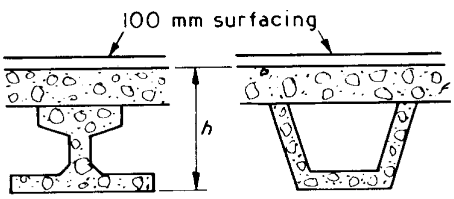
h	T_1
m	°C
0.2	13
0.3	16

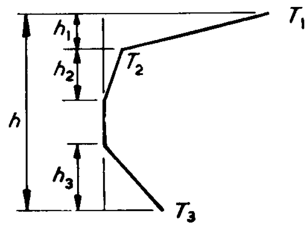


h	T_1
m	°C
0.2	3.5
0.3	5.0

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







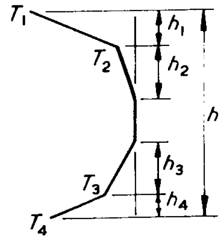
$h_1 = 0.3h \leqslant 0.15$ m

$h_2 = 0.3h \geqslant 0.10$ m
$\leqslant 0.25$ m

$h_3 = 0.3h \leqslant$ (0.1 m+ surfacing depth in metres)

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

h	T_1	T_2	T_3
m	°C		
$\leqslant 0.2$	8.5	3.5	0.5
0.4	12.0	3.0	1.5
0.6	13.0	3.0	2.0
$\geqslant 0.8$	13.5	3.0	2.5



$h_1 = h_4 = 0.2h \leqslant 0.25$ m

$h_2 = h_3 = 0.25h \leqslant 0.2$ m

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

Figure 9 — Temperature difference for different types of construction

5.4.8.4 *Design effects of temperature difference.* For combination 3, γ_{f1} shall be taken as follows.

For the ultimate limit state	For the serviceability limit state
1.00	0.80

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

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

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

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.

5.7 Exceptional loads. Where other loads not specified in this standard are likely to be encountered, e.g. the effects of 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.

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** to **4.4.4**), but there are circumstances, e.g. for opening bridges or where dead load stability is critical, when consideration should be given to it, Any snow load shall be included in combination 5 (see **4.4.5**) as superimposed dead load.

5.7.2 ***Design loads.*** For 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.

5.8 Earth pressure on retaining structures

5.8.1 ***Filling material***

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

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

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

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

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

except as defined in **5.8.1.3**.

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 (intermediate values

25 units: 10 kN/m^2 by interpolation);

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

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

5.8.2.2 *Design load.* For combinations 1 to 4, γ_{fL} shall be as specified in **5.8.1.2**.

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

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

5.9.1 *Temporary loads*

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

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

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 ultimate limit 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 Appendix A for the basis of HA and HB loading.)

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

design HA loading (see **6.4.1**) or

design HA loading combined with design HB loading (see **6.4.2**).

6.1.2 *Notional lanes, hard shoulders, etc.* The width and 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**.

6.1.3 ***Distribution analysis of structure.*** The effects of the design standard loadings shall, where appropriate, be distributed in accordance with a rigorous distribution analysis or from data derived from suitable tests.

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).*** The UDL shall be taken as 30 kN per linear metre of notional lane for loaded lengths up to 30 m, and for loaded lengths in excess of 30 m it shall be derived from the equation

$$W = 151 \left(\frac{1}{L}\right)^{0.475} \text{ but not less than 9.}$$

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

Values for this load per linear metre of notional lane are given in Table 13 and the loading curve is illustrated in Figure 10.

Table 13 — Type HA uniformly distributed load

Loaded length	Load	Loaded length	Load	Loaded length	Load
m	kN/m	m	kN/m	m	kN/m
Up to 30	30.0	73	19.7	160	13.6
32	29.1	76	19.3	170	13.2
34	28.3	79	18.9	180	12.8
36	27.5	82	18.6	190	12.5
38	26.8	85	18.3	200	12.2
40	26.2	90	17.8	210	11.9
42	25.6	95	17.4	220	11.7
44	25.0	100	16.9	230	11.4
46	24.5	105	16.6	240	11.2
49	23.8	110	16.2	255	10.9
52	23.1	115	15.9	270	10.6
55	22.5	120	15.5	285	10.3
58	21.9	125	15.2	300	10.1
61	21.4	130	15.0	320	9.8
64	20.9	135	14.7	340	9.5
67	20.5	140	14.4	360	9.2
70	20.1	145	14.2	380 and	9.0
		150	14.0	above	

NOTE The loaded length for the member under consideration shall be the base length of the adverse area (see **3.2.5**). Where there is more than one adverse area, as for continuous construction, the maximum effect should be determined by consideration of any adverse area or combination of adverse areas using the loading appropriate to the base length or the total combined base lengths.

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

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

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

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

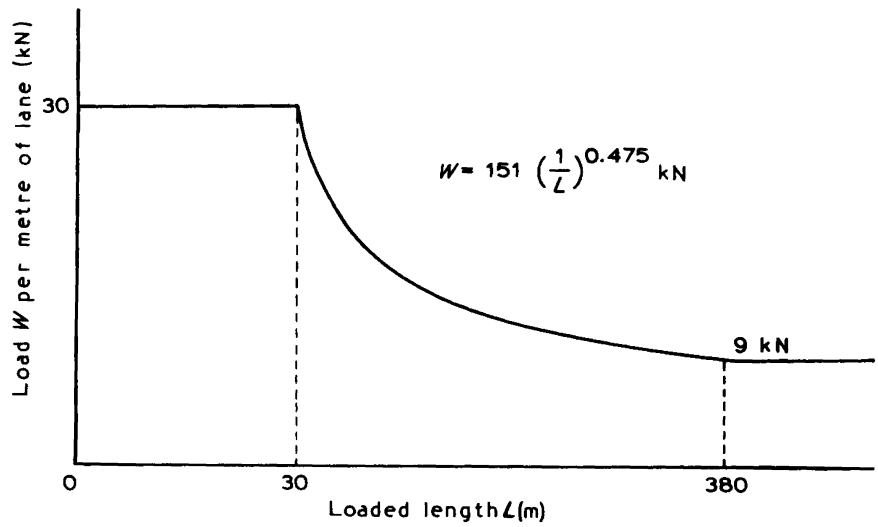


Figure 10 — Loading curve for HA UDL

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 combination 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 25, but this number may be increased up to 45 if so directed by the appropriate authority.

6.3.1 ***Nominal HB loading.*** Figure 11 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.

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 combination 1	1.30	1.10
For combinations 2 and 3	1.10	1.00

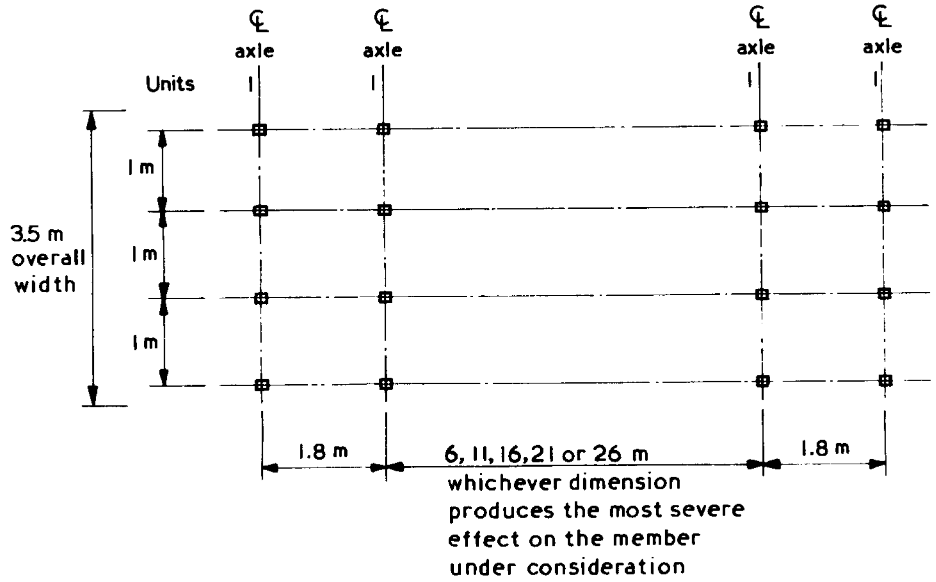


Figure 11 — Dimensions of HB vehicle

6.4 Application of types HA and HB loading

6.4.1 ***Type HA loading.*** Type HA UDL and KEL loads shall be applied to two notional lanes in the appropriate parts of the influence line for the element or member under consideration[7] and one-third type HA UDL and KEL loads shall be similarly applied to all other notional lanes except where otherwise specified by the appropriate authority. The KEL shall be applied at one point only in the loaded length of each notional lane.

Where the most severe effect is caused by locating portions of loaded length on one side of the superstructure over one portion of its length and on the other side of the superstructure in a longitudinally adjacent portion of its length, this shall be taken into consideration, using the loading appropriate to the combined length of the loaded portions.

6.4.1.1 *Multilevel structures.* Where multilevel superstructures are carried on common substructure members (as, e.g., columns of a multilevel interchange) each level shall be loaded and the loaded length shall be taken as the aggregate of the loaded lengths in each superstructure that has the most severe effect on the member under consideration.

6.4.1.2 *Transverse cantilever slabs, central reserves and outer verges.* HA UDL and KEL shall be replaced by the arrangement of HB loading given in **6.4.3**.

6.4.1.3 *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**.

6.4.1.4 *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 or spanning transversely: in a direction parallel to the supporting members or at right angles to the unsupported edges, whichever has the most severe effect. Where the element spans transversely, the KEL shall be considered as acting in a single line made up of portions having the same length as the width of the nominal lanes and having the intensities 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. Where plates or slabs are supported on all four sides see **6.4.3.1**.

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

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

c) On piers, abutments and other members supporting the superstructure: in a direction in line with the bearings.

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

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

6.4.2 ***Types HB and HA loading combined.*** Types HB and HA loading shall be combined and applied as specified in **6.4.2.1** and **6.4.2.2**.

6.4.2.1 *Type HB load.* Type HB loading shall be taken to occupy any transverse position on the carriageway, and in so doing will lie either wholly within one notional lane or will straddle two notional lanes. No other primary live loading shall be considered for 25 m in front of, to 25 m behind, the HB vehicle in the one lane occupied by the HB vehicle when it is wholly in one lane or in the two lanes when the HB vehicle is straddling them.

Only one H B load is required to be considered on any one superstructure or on any substructure supporting two or more superstructures.

6.4.2.2 *Associated type HA loading.* Where the HB vehicle is wholly within one lane, the remainder of the loaded length of this lane shall be loaded with full HA UDL only, of intensity appropriate to the loaded length that includes the total length displaced by the HB vehicle. Full HA loading shall be considered in one other notional lane, together with one-third HA loading in the remaining lanes.

Where the HB vehicle straddles two lanes, the following alternatives for associated HA highway loading shall be considered:

either

a) the remainder of the loaded length of both straddled lanes shall be loaded with full HA UDL only, of intensity appropriate to the loaded length that includes the total length displaced by the HB vehicle; all other lanes shall be loaded with one-third HA loading;

or

b) the remainder of the loaded length of one straddled lane shall be loaded with full HA UDL only and the remainder of the loaded length of the other straddled lane shall be loaded with one-third HA UDL only. The intensity of the full HA UDL and one-third HA UDL shall be that appropriate to a loaded length that includes the total length displaced by the HB vehicle. Full HA loading shall be considered in one other notional lane, together with one-third HA loading in the remaining lanes.

Figure 12 illustrates type HB loading in combination with type HA loading

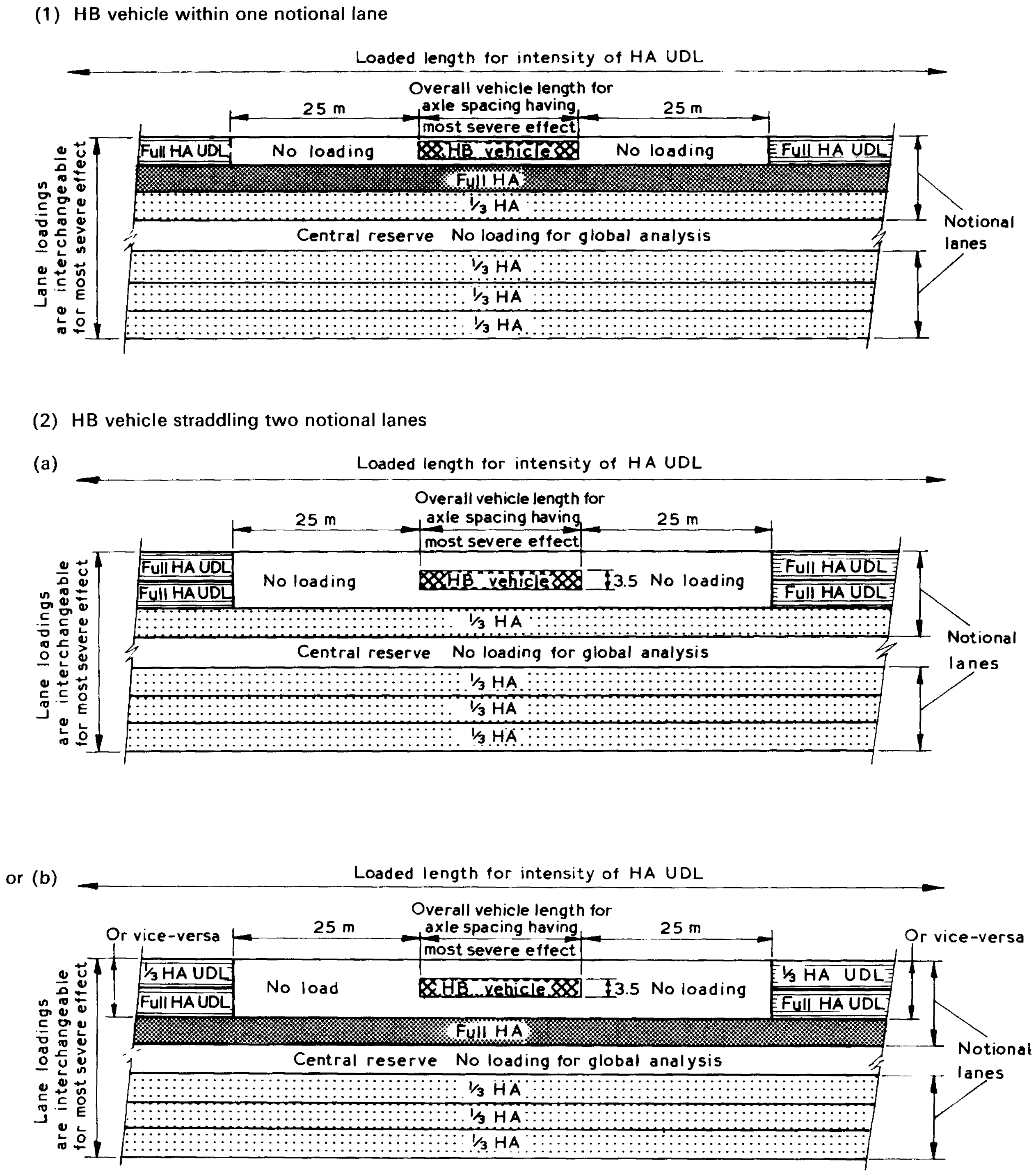
6.4.3 ***Highway loading on transverse cantilever slabs, slabs supported on all four sides, central reserves and outer verges.*** Type HB loading shall be applied to the elements specified in **6.4.3.1** to **6.4.3.3**.

6.4.3.1 *Transverse cantilever slabs and slabs supported on all four sides.* Transverse cantilever slabs shall be so proportioned as to resist the effects of the appropriate number of units of type HB loading placed in one notional lane in combination with 25 units of HB loading placed in one other notional lane. Proper consideration shall be given to transverse joints of transverse cantilever slabs and to the edges of these slabs because of the limitations of distribution[8)].

This does not apply to members supporting transverse cantilever slabs.

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 25 units of HB loading.

[8)] This is the only exception to the rule that not more than one HB vehicle shall be considered to act on a structure. The 25 unit vehicle is to be regarded as a substitute for HA loading for these elements only.



NOTE 1 The overall length and width of the HB vehicle shall be as specified in **6.3.1**.

NOTE 2 Unless otherwise stated, type HA loading includes both uniformly distributed loading (UDL) and knife edge loading (KEL).

Figure 12 — Type HA and HB highway loading in combination

6.4.3.3 *Outer verges.* Outer verges are not required to be loaded with live load in considering the overall design of the structure, but shall be capable of supporting any four wheels of 25 units of HB loading.

6.5 Centrifugal load. On curved bridges, point loads shall be applied in any two notional lanes at 50 m centres, acting in radial direction at the surface of the road and parallel to it.

6.5.1 ***Nominal centrifugal load.*** The nominal centrifugal load F_c shall be taken as

$$F_c = \frac{30000}{r + 150} \text{kN}$$

where r is the radius of curvature of the lane (in m)

Each load F_c shall be either taken as a single load or subdivided into two parts of one-third F_c and two-thirds F_c at 5 m centres longitudinally, whichever gives the lesser effect.

6.5.2 ***Associated nominal primary live load.*** With each centrifugal load there shall also be considered a vertical live load of 300 kN, distributed uniformly over the notional lane for a length of 5 m.

Where the centrifugal load is subdivided, the vertical live load shall be subdivided in the same proportions. The 100 kN portion shall be considered to act as a pointload coincident with the one-third F_c load and the 200 kN portion shall be considered as a distributed load applied uniformly over the notional lane for a length of 1 m and coincident with the two-thirds F_c load.

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

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

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

6.6 Longitudinal load. The longitudinal load resulting from traction or braking of vehicles shall be taken as the more severe of **6.6.1** or **6.6.2**, applied at the road surface and parallel to it in one notional lane only.

6.6.1 ***Nominal load for type HA.*** The nominal load for HA shall be 8 kN/m of loaded length plus 200 kN, subject to a maximum of 700 kN, applied to an area one notional lane wide × the loaded length.

6.6.2 ***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**).

6.6.3 ***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.

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

6.6.5 ***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

6.7 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 parallel to the surface of the highway.

6.7.1 ***Nominal load.*** The nominal load shall be taken as 250 kN.

6.7.2 ***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.

6.7.3 ***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.

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

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

6.8 Loads due to vehicle collision with Parapets[9)]

6.8.1 ***Nominal load.*** in the design of the elements of the structure supporting parapets, the actual loads, moments or shears required to bring about the collapse of the parapet or the connection of the parapet to the element (whichever is the greater) shall be regarded as the nominal loads, moments or shears applied to the element.

6.8.2 ***Associated nominal primary live load.*** Any four wheels of 25 units of HB loading (see **6.3**) shall be considered in whatever position they will have the most adverse effect on the element; where their application has a relieving effect they shall be ignored.

6.8.3 ***Load combination.*** Loads due to vehicular collision with parapets shall be considered in combination 4 only, and need not be taken as coexistent with other secondary live loads.

6.8.4 ***Design load.*** For the load transmitted by the collapse of the parapet or its connection to the element supporting the parapet and for the primary live load, γ_{fL} shall be taken as follows.

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

6.9 Collision loads on supports at bridges over highways. Members supporting bridge superstructures, both in the central reservation and at the road edge, shall be protected by safety fences where traffic is permitted to travel at speeds of 80 km/h or above, and in other cases where damage to the supports might lead to severe consequences. For foot/cycle track bridges see **7.1.4** (and Appendix B).

6.9.1 ***Nominal load.*** The nominal loads are given in Table 14, together with their direction and height of application, and are to be considered as acting horizontally on bridge supports (but see **7.1.4** for foot and cycle bridges).

Supports shall be capable of resisting the load transmitted from the guard rail applied simultaneously with the residual load above the guard rail. Loads normal to the carriageway are to be considered separately from loads parallel to the carriageway.

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

6.9.3 ***Load combination.*** Collision load 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 vehicle collision load on supports, γ_{fL} shall be as follows:

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

Table 14 — Collision loads on supports of bridges over highways

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

6.9.5 ***Bridges over railways, canals or navigable water.*** Collision loading on supports of bridges over railways, canals or navigable water shall be as agreed with the appropriate authority.

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

[9)] This subclause refers to the load transmitted to structural elements supporting the parapets. Rules for the design of highway parapets in the United Kingdom are set out in the appropriate Department of Transport Memorandum.

6.11 Dynamic loading on highway bridges. The effects of vibration due to traffic are not required to be considered.

7 Footway and cycle track live load

7.1 Bridges supporting footways or cycle tracks only

7.1.1 ***Nominal live load.*** The nominal live load on elements supporting footways and cycle tracks only shall be taken as follows:

a) for loaded lengths of 30 m and under, a uniformly distributed live load of 5.0 kN/m^2;

b) for loaded lengths in excess of 30 m, $k \times 5.0$ kN/m^2,

where k is the

$$\frac{\text{nominal HA UDL for appropriate loaded length (in kN/m)}}{30\,\text{kN/m}}$$

Special consideration shall be given to the intensity of the live load to be adopted on loaded lengths in excess of 30 m where exceptional crowds may be expected (as, for example, where a footbridge serves a sports stadium).

7.1.2 ***Nominal load on pedestrian parapets.*** The nominal load shall be taken as 1.4 kN/m length, applied at a height of 1 m above the footway or cycle track and acting horizontally.

7.1.3 ***Design load.*** For the live load on footways and cycle tracks 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

7.1.4 ***Collision load on supports of foot/cycle track bridges.*** The load on the supports of foot/cycle track bridges shall be a single load of 50 kN applied horizontally in any direction up to a height of 3 m above the adjacent carriageway. This is the only secondary live load to be considered for foot/cycle track bridges.

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

7.1.4.2 *Design load.* For combination 4,γ_{fL} shall be taken as follows.

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

7.1.5 ***Vibration serviceability.*** Consideration shall be given to vibration that can be induced in foot/cycle track bridges by resonance with the movement of users. The structure shall be deemed to be satisfactory where its response, as calculated in accordance with Appendix C, complies with the limitations specified therein.

7.2 Elements supporting footways or cycle tracks and a highway or railway

7.2.1 ***Nominal live load.*** On footways and cycle tracks carried by elements that also support highway or railway loading, the nominal live load shall be taken as 0.8 of the value specified in **7.1.1**. a) or b), as appropriate, except where crowd loading is expected, in which case special consideration shall be given to the intensity of live loading to be adopted.

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

Where a main structural member supports two or more highway traffic lanes or railway tracks, the footpath and 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 **7.1.1** a) and b), as appropriate.

On cycle tracks: 0.2 of the value given in **7.1.1** a) and b), as appropriate.

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

7.2.2 ***Nominal wheel load.*** Where the footway or cycle track is not protected from highway traffic by an effective barrier, any four wheels of 25 units of HB loading (see **6.3**) acting alone in any position shall be considered.

7.2.3 ***Associated nominal primary live load.*** Associated nominal primary live load on the carriageway or rail track shall be derived and applied in accordance with **6.4** or clause **8**, as appropriate.

7.2.4 ***Load due to vehicle collision with parapets.*** Where the footway or cycle track is not protected from the highway traffic by an effective barrier, the nominal loads on elements of the structure supporting parapets shall be as specified in **6.8**.

7.2.5 ***Design load.*** γ_{fL}, to be applied to the nominal loads shall be as follows:

a) for live loading on footways and cycle tracks, as specified in **7.1.4**;

b) for highway live loading, as specified in **6.2.7** and **6.3.4**;

c) for railway live loading, as specified in **8.4**;

d) for loading derived from vehicle collision with parapets, as specified in **6.8**.

8 Railway bridge live load

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

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

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

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

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

8.2 Nominal loads

8.2.1 ***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 13.

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 14.

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, provided that maintenance of track and rolling stock is kept to a reasonable standard.

8.2.3.1 *Type RU loading.* The dynamic factor for RU loading applies to all types of track and shall be as given in Table 15.

Table 15 — Dynamic factor for type RU loading

Dimension L	Dynamic factor for evaluating	
	bending moment	shear
m		
up to 3.6	2.00	1.67
from 3.6 to 67	$0.73 + \frac{2.16}{\sqrt{(L-0.2)}}$	$0.82 + \frac{1.44}{\sqrt{(L-0.2)}}$
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.

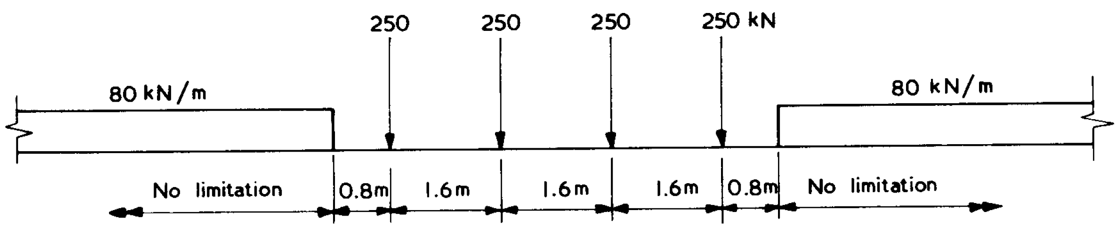
The values given in Table 16 may be used, where appropriate.

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.

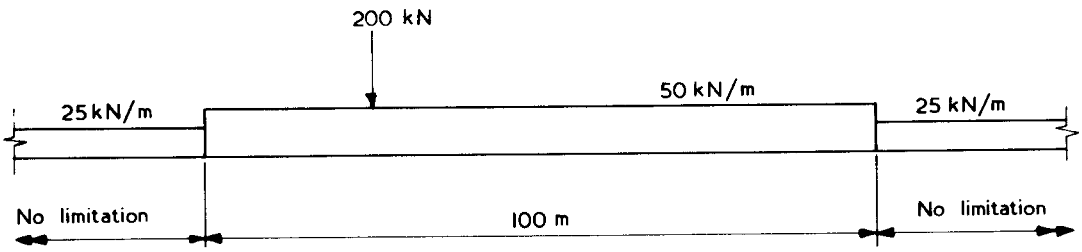
8.2.4 ***Dispersal of concentrated loads.*** Concentrated loads applied to the rail will be distributed both longitudinally by the continuous rail to more than one sleeper, and transversely over a certain area of deck by the sleeper and ballast.

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 by the two sleepers either side.



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

Figure 13 — Type RU loading



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

Figure 14 — Type RL loading

Table 16 — Dimension *L* used in calculating the dynamic factor for RU loading

	Dimension *L*
Main girders:	
simply supported	span
continuous	For 2, 3, 4, 5 and more spans 1.2, 1.3, 1.4, 1.5 × mean span, but at least the greatest span
portal frames and arches	$^1/_2$ 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 element in a continuous deck system	The lesser of the span of the main girders and twice the main spacing

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.*** Type RU or RL loading shall be applied to each and every track as specified in **4.4**. Any number of lengths of the distributed load may be applied, but for RL loading 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.

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

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

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

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

The vertical effects of this load on secondary elements such as rail bearers shall be considered.

For elements supporting more than one track a single load, as specified, shall be deemed sufficient.

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

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

where

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

r is the radius of curvature (in m)

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

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

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

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

L is the loaded length of the element being considered.

8.2.10 *Longitudinal loads.* Provision shall be made for the nominal loads due to traction and application of brakes as given in Table 17. These loads shall be considered as acting at rail level in a direction parallel to the tracks. No addition for dynamic effects shall be made to the longitudinal loads calculated as specified in this subclause.

For bridges supporting ballasted track, up to one-third of the longitudinal loads may be assumed to be transmitted by the track to resistances outside the bridge structure, provided that no expansion switches or similar rail discontinuities are located on, or within, 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 element carries two tracks, both tracks shall be considered as being occupied simultaneously. Where the tracks carry traffic in opposite directions, the load due to braking shall be applied to one track and the load due to traction to the other. Structures and elements carrying two tracks in the same direction shall be subjected to braking or traction on both tracks, whichever gives the greater effect. Consideration, however, may have to be given to braking and traction, acting in opposite directions, producing rotational effects.

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

8.3 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.

8.4 Design loads. For primary and secondary railway live loads, γ_{fl} shall be taken as follows.

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

Table 17 — Nominal longitudinal loads

Standard loading type	Load arising from	Loaded length	Longitudinal load
		m	kN
RU	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

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

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

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

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

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

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

a) For the serviceability limit state, either

1) a pair of parallel vertical line loads of 20 kN/m each, 1.4 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, with each of the 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.

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

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

a) For the serviceability limit state, either

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

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

b) For the ultimate limit state, four individual concentrated 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 vertical line load of 30 kN/m, applied along the parapet or outermost edge of the bridge, limited to a length of 20 m anywhere on the span.

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

8.6 Collision load on supports of bridges over railways.[10] The collision load on supports of bridges over railways shall be as agreed with the appropriate authorities.

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

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

[10] Requirements for the supports of bridges over highways and waterways are specified in **6.9**.

Appendix A Basis of HA and HB highway loading

Type HA loading is the normal design loading for Great Britain, where it represents the effects of normal permitted vehicles[11] other than those used for the carriage of abnormal indivisible loads.

For loaded lengths up to 30 m, the loading approximately represents closely spaced vehicles of 24 t laden weight in each of two traffic lanes. For longer loaded lengths the spacing is progressively increased and medium weight vehicles of 10 t and 5 t are interspersed. It should be noted that although normal commercial vehicles of considerably greater weight are permitted in Great Britain their effects are restricted, so as not to exceed those of HA loading, by limiting the weight of axles and providing for increased overall length.

In considering the impact effect of vehicles on highway bridges an allowance of 25 % on one axle or pair of adjacent wheels was made in deriving HA loading. This is considered an adequate allowance in conditions such as prevail in Great Britain.

This loading has been examined in comparison with traffic as described for both elastic and collapse methods of analysis, and has been found to give a satisfactory correspondence in behaviour.

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

Appendix B Recommendations for the protection of piers by safety fences

The space available for deflection determines the arrangement of the safety fences. If the clearance between the pier and the guard rail is 0.6 m or more, the guard rail should be mounted on posts to form a free-standing fence. If the clearance is less than 0.6 m, the guard rail should be mounted on the traffic face of the member by means of energy absorbing brackets. Whatever the arrangement the protection afforded should be such that when a car of 1.5 t strikes the safety fence at 110 km/h, and at an angle of 20°, the wheels of the car will only just reach the member.

Appendix C Vibration serviceability requirements for foot and cycle track bridges

C.1 General

For superstructures where f_o, the fundamental natural frequency of vibration for the unloaded bridge, exceeds 5 Hz, 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 ýf_o m/s^2. The maximum vertical acceleration shall be calculated in accordance with **C.2** or **C.3**, as appropriate.

C.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

$$a = 4\,\pi 2\, f^2_o\, y_s\, K\psi$$

where

- f_o is the fundamental natural frequency (in Hz) (see **C.2.3**)
- y_s is the static deflection (in m) (see **C.2.4**)
- K is the configuration factor (see **C.2.5**)
- ψ is the dynamic response factor (see **C.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.

C.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.

[11] As defined in Motor Vehicles (Construction and Use) Regulations (Statutory Instrument no. 321, 1969) and subsequent amendments, available from HMSO.

C.2.2 ***Second moment of area.*** In calculating the values of f_0 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.

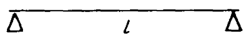
C.2.3 ***Fundamental natural frequency*** f_0. The fundamental natural frequency f_0 is evaluated for the bridge including superimposed dead load but excluding pedestrian live loading.

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

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

C.2.5 ***Configuration factor K.*** Values of *K* shall be taken from Table 18.

Table 18 — Configuration factor *K*

Bridge configuration		*K*
	—	1.0
	—	0.7
	**Ratio** l_1/l	
	1.0	0.6
	0.8	0.8
	0.6 or less	0.9

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

C.2.6 ***Dynamic response factor*** ψ. Values of ψ are given in Figure 15. 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 19 should be used.

Table 19 — Logarithmic decrement of decay of vibration δ

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

C.3 General method for deriving maximum vertical acceleration

For superstructures other than those specified in **C.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.

$F = 180 \sin 2\pi f_0 T$ (in N), where T is the time (in s),

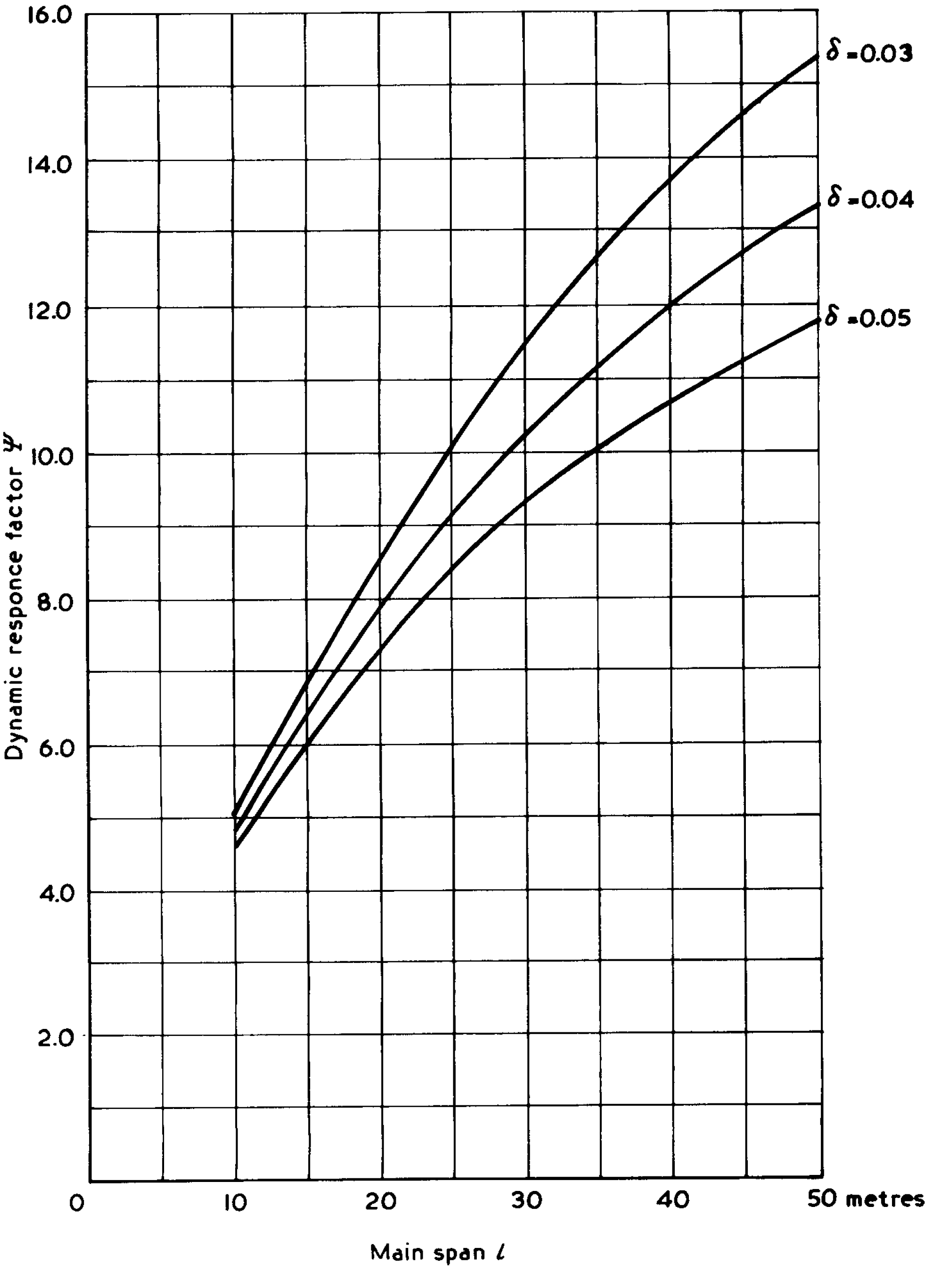
$V_t = 0.9 f_0$ (in m/s).

For values of f_0 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.

C.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/is shown in Table 18.

NOTE 2 Values of δ for different types of construction are given in Table 19.

Figure 15 — Dynamic response factor *Ψ*

Appendix D Derivation of RU and RL railway loadings

D.1 RU loading

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

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

Figure 16 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 loadings.

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 of up to 120 km/h, passenger locomotives at speeds of up to 250 km/h and light, high speed trains at speeds of up to 300 km/h.

The formulae for the dynamic effects are not to be used to calculate dynamic effects for a particular train on a particular bridge. Appropriate methods for this can be found by reference to a recommendation published by the International Union of Railways (UIC), Paris[12].

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

D.2 RL loading

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

RL loadings covers the following conditions, which are illustrated in Figure 17 and Figure 18.

a) *Works trains.* This constitutes locomotives, cranes and wagons used for maintenance purposes. Locomotives are usually of the battery car type although very occasionally diesel shunters may be used. Rolling stock hauled includes a 30 t steam crane, 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.

[12] Leaflet 776—1 R, published by UIC, 14 rue Jean-Ray F, 75015 Paris, France.

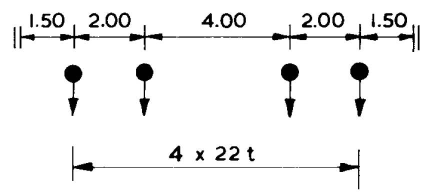
c) Cross girders and rail bearers at rail joints

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.

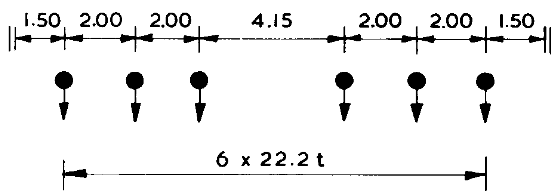
3) With ballasted track, the rail joint effect is considerably reduced, depending on the standard and uniformity of compaction of ballast beneath the sleepers. The maximum increase in poorly maintained track is about 12 % for each 10 km/h of speed.

B – B LOCOMOTIVE



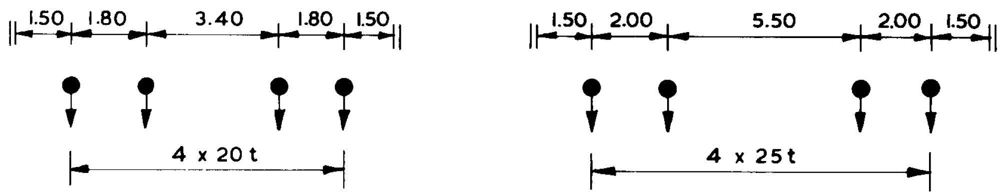
To be considered
Double headed

C – C LOCOMOTIVE



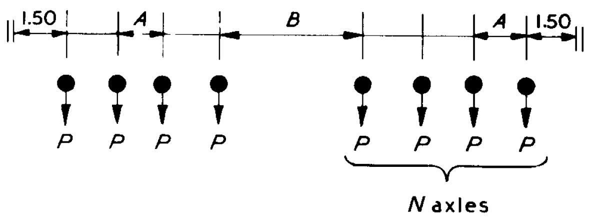
To be considered
Double headed

WAGONS



EXCEPTIONAL WAGONS

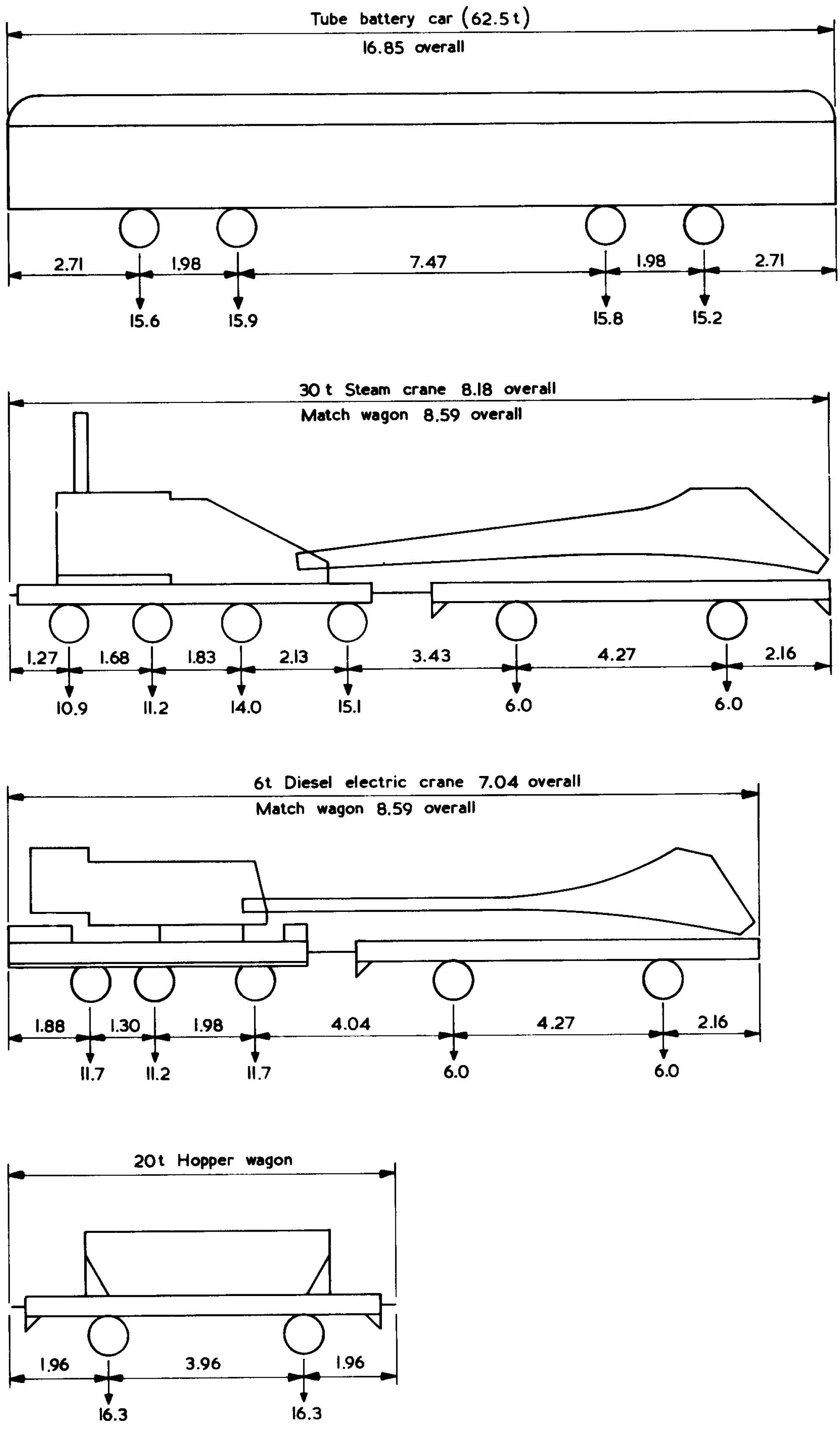
N is the number of axles in each end
2*N* is the total number of axles
t = metric tonnes



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

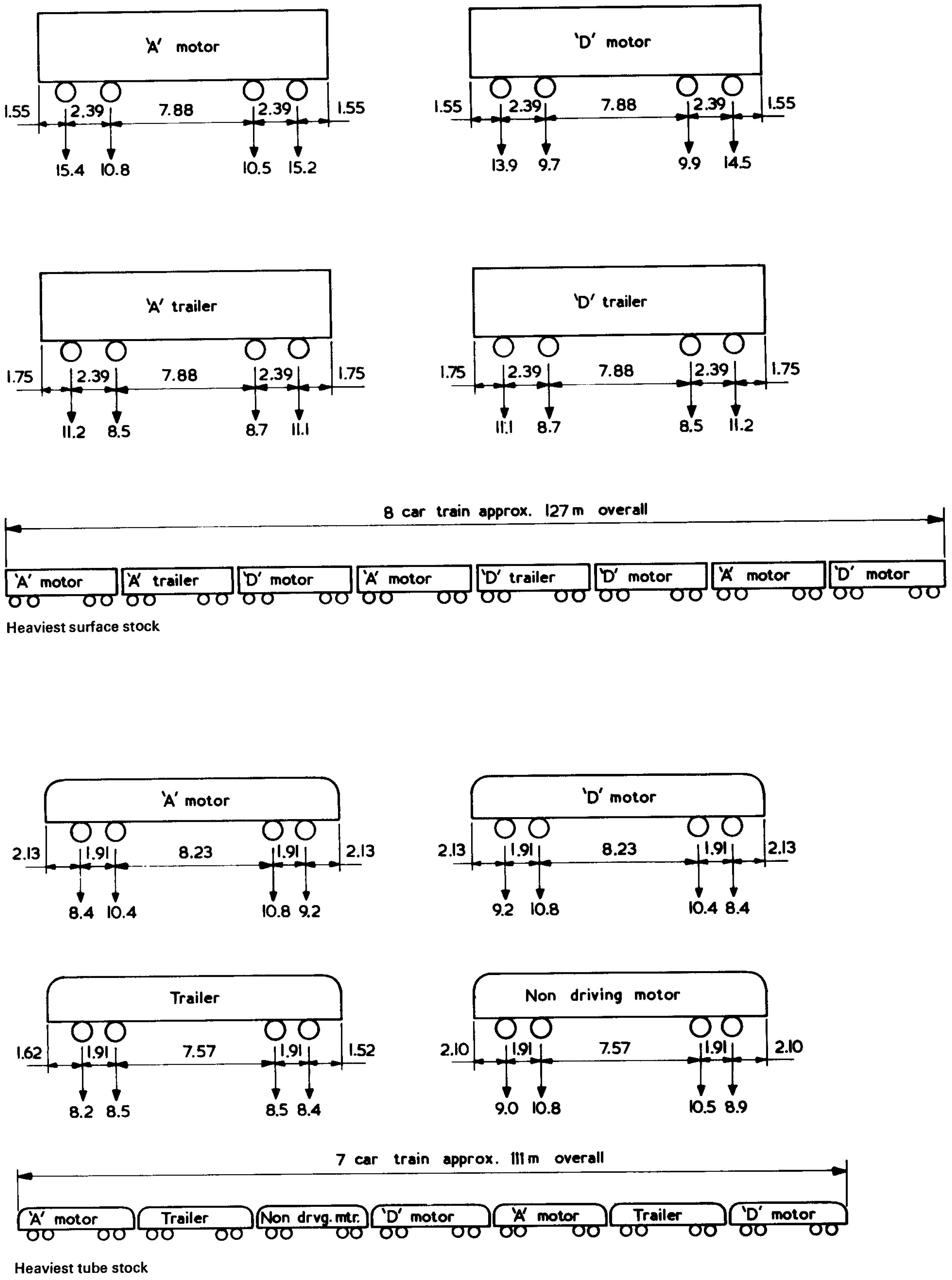
NOTE Dimensions in metres.

Figure 16 — Wagons and locomotives covered by RU loading



Dimensions are in metres. Axle loads are in tonnes.

Figure 17 — Works trains vehicles covered by RL loading



Dimensions are in metres. Axle loads are in tonnes

Figure 18 — Passenger vehicles covered by RL loading

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 20 to Table 23 when designing for RU loading

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

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

Shear forces at points other than the end shall be determined by using the static shear force from Table 21 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 19) and the result shall be multiplied by the dynamic factor for shear in which L is taken to be the span of the girder.

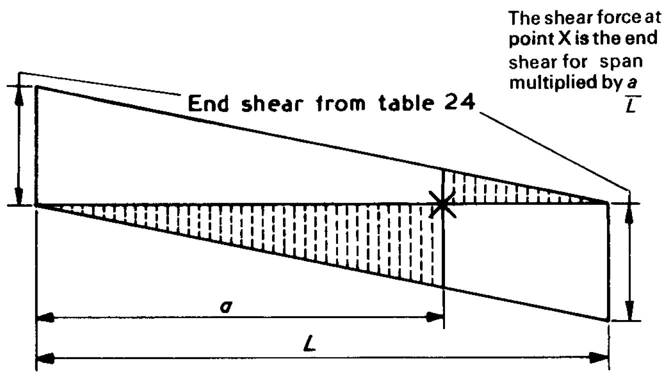


Figure 19 — Shear force determination

D.3.2 ***Cross girders loaded through simply supported rail bearers.*** The cross girders shall be designed to carry two concentrated point loads for each track. Each of these loads is to be taken as one-quarter of the equivalent uniformly distributed load for bending moments shown in Table 20 for a span equal to twice the cross girder spacing, multiplied by the appropriate dynamic factor.

Table 20 — Equivalent uniformly distributed loads for bending moments for simply supported beams (static loading) under RU loading

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

Table 21 — End shear forces for simply supported beams (static loading) under RU loading

Span	Force	Span	Force	Span	Force
m	kN	m	kN	m	kN
1.0	252	8.0	729	50.0	2 529
1,2	255	8.2	740	52.0	2 610
1.4	260	8,4	752	54.0	2 691
1,6	266	8.6	763	56.0	2 772
1.8	278	8.8	774	58.0	2 852
2.0	300	9.0	785	60.0	2 933
2.2	318	9.2	795	65.0	3 134
2.4	333	9.4	806	70.0	3 336
2.6	347	9.6	817	75.0	3 537
2.8	359	9.8	827	80.0	3 738
3.0	371	10.0	837	85.0	3 939
3.2	383	11.0	888	90.0	4 139
3.4	397	12.0	937	95.0	4 340
3.6	417	13.0	984	100,0	4 541
3.8	434	14.0	1 030	105.0	4 741
4.0	450	15.0	1 076	110.0	4 942
4.2	465	16.0	1 120	115.0	5 142
4.4	479	17.0	1 165	120.0	5 342
4.6	492	18.0	1 208	125.0	5 543
4.8	505	19.0	1 252	130.0	5 743
5.0	520	20.0	1 295	135.0	5 944
5.2	538	22.0	1 380	140.0	6 144
5.4	556	24.0	1 464	145.0	6 344
5.6	571	26.0	1 548	150.0	6 544
5.8	586	28.0	1 631	155.0	6 745
6.0	601	30.0	1 714	160.0	6 945
6.2	615	32.0	1 796	165.0	7 145
6.4	629	34.0	1 878	170.0	7 345
6.6	642	36.0	1 960	175.0	7 545
6.8	656	38.0	2 042	180.0	7 746
7.0	668	40.0	2 123	185.0	7 946
7.2	681	42.0	2 205	190.0	8 146
7.4	693	44.0	2 286	195.0	8 346
7.6	705	46.0	2 367	200.0	8 546
7.8	717	48.0	2 448		

Table 22 — Equivalent uniformly distributed loads for bending moments for simply supported beams, including dynamic effects, under RU loading

Span	Load	Span	Load	Span	Load
m	kN	m	kN	m	kN
1.0	1 000	8.0	1 951	50.0	5 136
1.2	1 000	8.2	1 975	52.0	5 273
1.4	1 000	8.4	1 999	54.0	5 411
1.6	1 000	8.6	2 022	56.0	5 547
1.8	1 002	8.8	2 044	58.0	5 684
2.0	1 007	9.0	2 066	60.0	5 820
2.2	1 015	9.2	2 088	65.0	6 160
2,4	1 024	9.9	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 23 — End shear forces for simply supported beams, including dynamic effects, under RU loading

Span	Force	Span	Force	Span	Force
m	kN	m	kN	m	kN
1.0	421	8.0	997	50.0	2 604
1.2	427	8.2	1 007	52.0	2 676
1.4	435	8.4	1 018	54.0	2 748
1.6	445	8.6	1 028	56.0	2 821
1.8	464	8.8	1 037	58,0	2 893
2.0	501	9,0	1 047	60,0	2 965
2.2	532	9.2	1 057	65.0	3 144
2.4	557	9.4	1 066	70.0	3 336
2.6	579	9.6	1 076	75.0	3 537
2.8	601	9.8	1 085	80.0	3 738
3.0	621	10.0	1 094	85.0	3 939
3.2	640	11.0	1 138	90.0	4 139
3.4	663	12.0	1 181	95.0	4 340
3.6	695	13.0	1 223	100.0	4 541
3.8	714	14.0	1 264	105.0	4 741
4.0	729	15.0	1 304	110.0	4 942
4.2	743	16.0	1 343	115.0	5 142
4.4	756	17.0	1 383	120.0	5 342
4.6	768	18.0	1 421	125.0	5 543
4.8	780	19.0	1 460	130.0	5 743
5.0	794	20.0	1 498	135.0	5 944
5.2	815	22.0	1 574	140.0	6 144
5.4	832	24.0	1 649	145.0	6 344
5.6	849	26.0	1 724	150.0	6 544
5.8	864	28.0	1 799	155.0	6 745
6.0	878	30.0	1 873	160.0	6 945
6.2	892	32.0	1 947	165.0	7 145
6.4	905	34.0	2 021	170.0	7 345
6.6	917	36.0	2 094	175.0	7 545
6.8	930	38.0	2 167	180.0	7 746
7.0	942	40.0	2 240	185.0	7 946
7.2	953	42.0	2 313	190.0	8 146
7.4	965	44.0	2 386	195.0	8 346
7.6	976	46.0	2 459	200.0	8 546
7.8	987	48.0	2 531		

Appendix E 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 in Table 24 to Table 26 may be used. These values are based on the temperature difference curves given in Transport and Road Research Laboratories (TRRL) Report LR 765 "Temperature differences in bridges", which may be used in preference. Methods of computing temperature difference are to be found in TRRL Report LR 561 "The calculation of the distribution of temperature in bridges".

Table 24 — Values of *T* for groups 1 and 2

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

Table 25 — Values of *T* for group 3

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

Table 26 — Values of T for group 4

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

Appendix F deleted. Superseded by Part 9 of this standard.

blank

Publications referred to

BS 153, *Steel girder bridges*.
CP 2004, *Foundations*[13].

[13] In course of revision.

BSI — British Standards Institution

BSI is the independent national body responsible for preparing British Standards. It presents the UK view on standards in Europe and at the international level. It is incorporated by Royal Charter.

Revisions

British Standards are updated by amendment or revision. Users of British Standards should make sure that they possess the latest amendments or editions.

It is the constant aim of BSI to improve the quality of our products and services. We would be grateful if anyone finding an inaccuracy or ambiguity while using this British Standard would inform the Secretary of the technical committee responsible, the identity of which can be found on the inside front cover. Tel: 020 8996 9000. Fax: 020 8996 7400.

BSI offers members an individual updating service called PLUS which ensures that subscribers automatically receive the latest editions of standards.

Buying standards

Orders for all BSI, international and foreign standards publications should be addressed to Customer Services. Tel: 020 8996 9001. Fax: 020 8996 7001.

In response to orders for international standards, it is BSI policy to supply the BSI implementation of those that have been published as British Standards, unless otherwise requested.

Information on standards

BSI provides a wide range of information on national, European and international standards through its Library and its Technical Help to Exporters Service. Various BSI electronic information services are also available which give details on all its products and services. Contact the Information Centre. Tel: 020 8996 7111. Fax: 020 8996 7048.

Subscribing members of BSI are kept up to date with standards developments and receive substantial discounts on the purchase price of standards. For details of these and other benefits contact Membership Administration. Tel: 020 8996 7002. Fax: 020 8996 7001.

Copyright

BSI
389 Chiswick High Road
London
W4 4AL