TECHNICAL MEMORANDUM (BRIDGES) BE 1/77: STANDARD HIGHWAY LOADINGS

1. This Technical Memorandum is a revision of BE 5/73: Standard Highway loadings and is closely based on B116/2 draft loading requirements as consistent with the current elastic stress basis of design for which this Memorandum is intended to be used. It is anticipated that this document will continue to be used during a transition period after B116 has been published and before the changeover to limit state design has been made fully effective.

B116/2 has not been fully implemented. Until further notice therefore, traffic live loading shall continue to be in accordance with BS153: Specification for steel girder bridges: Part 3A loads:1972.

The modifications to BE 5/73 include the following:

2. Structures of span or internal diameter greater than 0.9 metres are now classified as highway structures and subject to standard highway loadings. In addition, footbridges and sign/signal gantries are comprehensively covered and are designed for a 50 year life.

3. Guidance on safety factors for overall stability of substructures and retaining walls is given.

4. Wind clauses have been substantially rearranged to clarify requirements and to give a more logical presentation. Isopleths of mean hourly wind speed are updated. Reductions are permitted for wind sensitive structures for sheltered/urban locations.

5. Temperature requirements are updated and now permit adjustments for the geographical and physical location of the structure. Actual and linearised temperature differences are given. These differences are not to be considered on gantries or in combination with wind on other structures.

6. The dynamic behaviour of footbridges, based on verified levels of pedestrian discomfort is considered.

7. Footbridge/gantry supports must now resist vehicle collision impact of 50 kN. Load combination for impact on all highway structure supports to include for 0.5 wind. Highway and footbridge superstructures must resist a 50 kN impact.

8. Buried structures are to be designed for wheel loads with 2 to 1 dispersal or Boussinesq theory. Modified temperature requirements are given for variable fill depths and structure spans.

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14th February 1977

TECHNICAL MEMORANDUM (BRIDGES) NO. BE 1/77

DEPARTMENT OF TRANSPORT

HIGHWAYS DIRECTORATE

TECHNICAL MEMORANDUM (BRIDGES)

STANDARD HIGHWAY LOADINGS

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STANDARD HIGHWAY LOADINGS

FOREWORD

The Department is at present assisting the British Standards Institution in drafting a document for the design and construction of steel, concrete and composite bridges specifications for the loads, materials and workmanship.

Until the Code is published and adopted by the Department, it is intended that the loading in this Technical Memorandum should be used with elastic theory for the design of highway structures. The current practice of allowing an increase in the basic permissible elastic stresses for certain loading combinations will continue.

The design of members subjected to impact loading and also certain aspects of prestressed concrete design are based on an ultimate load theory.

This Memorandum must be read in conjunction with the relevant clauses of BS 153: Specification for steel girder bridges Part 3A loads: 1972* listed in Appendix A. It supersedes Technical Memorandum (Bridges) BE 5/73: Standard Highway Loadings.

1. SCOPE

The loading to be considered in the design of all highway structures having a span or internal diameter greater than 0.9 metres for motorway and trunk road projects shall be in accordance with this Memorandum, except for the following:

i. Steel box construction - see "Interim Design and Workmanship Rules for Steel Box Girder Bridges".

ii. Bridge parapets - see Technical Memorandum BE5: The Design of Highway Bridge Parapets (Third Revision).

iii. Lighting columns - see Technical Memorandum BE 4/72: Street Lighting Columns of Steel Construction, BS 3989: Street Lighting Columns (Aluminium) 1966, and BS 1308: Street Lighting Columns (Concrete) 1970 as appropriate.

iv. Noise barriers - see Technical Memorandum H14/76: Noise Barriers – Standards and Materials.

v. Safety fences - see Technical Memorandum H9/73: Safety fences.

This document is recommended for use in the design of structures on other road projects.

*All references to the Standard in this Memorandum refer to BS 153: Part 3A; 1972

2. LOADS

2.1 Loads to be considered

For the purposes of calculating stresses and stability the following loads shall be considered, where applicable. Those loads to be considered in combination 1 are marked (1).

2.1.1 Loads other than live loads on highway structures

Dead load	(1)	See 3.1
Superimposed dead load	(1)	See 3.2
Wind loading		See 3.3
Temperature effects		See 3.4
Restraint at bearings		See 3.5
Shrinkage and creep in concrete	(1)	See 3.6
Erection loads and effects		See 3.7
Differential Settlement	(1)	See 3.8
Exceptional forces eg earthquakes, snow, ice		See 3.9
Earth pressures on retaining structures	(1)	See 7.1

2.1.2 Highway live loading

Live loading	(1)	See 4.1 and 4.2
Longitudinal load	(1)	See 4.3
Centrifugal load	(1)	See 4.4
Loads on parapets		See 4.5
Fatigue loading		See 4.6
Accidental wheel loading		See 4.7
Vehicle collision with highway structures		See 6
Live load surcharge	(1)	See 7.2

2.1.3 Footway and cycle-track live loading

Live load on footway and/or cycle-track bridges	(1)	See 4.8.1
Live load on elements supporting the highway as well as footway and/or cycle track	(1)	See 4.8.2

2.1.4 Sign/signal gantry live loading

Global effects	(1)	See 4.9.1
Local effects	(1)	See 4.9.2

2.1.5 Loading on buried structures See 8

2.2 Stability

2.2.1 Anchorage of superstructure The stability of the superstructure and its parts shall be considered in accordance with Clause 18 of the Standard. The factor of safety specified for stability in Clause 18 shall also be applied to the anchorage of the end of continuous superstructures.

2.2.2 Substructures* and retaining walls Abutments, wing walls and retaining walls shall be assessed in accordance with CP2: Earth Retaining Structures: 1951.

Foundations of piers and gantry supports shall be assessed in accordance with CP2004: Foundations: 1972. In applying CP2004, the dead load (3.1) superimposed dead load (3.2) and earth pressure from filling material (7.1) shall be regarded as permanent loads and all live loads may be regarded as transient loads.

2.2.2.1 Overall stability for combination 1. The safety factors to be used for the overall stability of substructures and retaining walls shall be derived from CP2.

2.2.2.2 Overall stability for combination 2. The safety factors to be used for the stability may be reduced to 1.4 for combination 2 loading except for earth retaining structures where combinations which include both vehicle collision on parapets and earth pressure are considered when a reduced safety factor of 1.1 may be applied (see 4.5.2).

2.3 Dynamic effects

Dynamic oscillations of highway structures shall be considered in accordance with 3.3.8 for wind effects and 5 for live loading effects.

2.4 Application of loads

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

2.4.2 Selection of loads. Loads shall be selected and applied in such a way that the most severe effect is caused in the element or structure under consideration.

2.4.3 Relieving effect of certain loads. Where the application of loads due to traffic live load or superimposed dead load on any portion of the element or structure under consideration has an effect opposite in sign to the total effect or where the most severe effect on the element or structure will be diminished by the presence of the load the load shall be assumed not to act on that portion.

Where a particular element of superimposed dead load is provided for the purpose of contributing to stability (as, eg kentledge but not road surfacing and ballast) and precautions are taken to ensure that it will not be removed, this element may be taken into consideration.

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* For the purpose of this clause, a substructure includes piers and structure supports (including foundations), abutments and wing walls.

2.5 Combination of loads

Loads shall be combined in accordance with the following provisions for the different types of construction noted below. In addition, Clauses 4.7 and 6 of this Memorandum give the loads to be combined with accidental wheel loading and vehicle collision loading respectively.

2.5.1 Plate girder and rolled section beam structures. Clause 1.3 of Technical Memorandum BE 3/76 requires that loads shall be combined in accordance with "Interim Design and Workmanship Rules for Steel Box Girder Bridges".

2.5.2 Other steel structures. As Clause 16 of BS 153: Part 3A: Loads.

2.5.3 Composite construction in structural steel and concrete. As Clause 5 of CP 117: Composite Construction in Structural Steel and Concrete: Part 2: Beams for Bridges.

2.5.4 Reinforced, prestressed and composite concrete construction

2.5.4.1 Permanent and transient loads. The loads applied to a structure shall be regarded as either permanent or transient.

For the purpose of this Memorandum dead load, superimposed dead load, the effects of shrinkage and creep of concrete (where appropriate) together with other loads derived from them shall be regarded as permanent loads. The effects of differential settlement of supports shall also be regarded as a permament load where there is reason to consider that this will take place and no special provision is made to accommodate the relative movement.

All other loads shall be regarded, for the purpose of this Memorandum, as transient loads.

2.5.4.2 Working load combinations. The following combinations of loads at working load shall be considered:

Combination 1 comprises the permanent loads and live load due to traffic, including all loads which derive from traffic using the structure, but with the exception of loads due to vehicular collision with parapets, supports or superstructures and accidental wheel loading on central reserves, footways and cycle tracks.

Combination 2 comprises any or all compatible permanent and transient loads.

2.5.4.3 Ultimate load combination. The effects of factored values of dead load, superimposed dead load and live load due to traffic shall be combined as described in Technical Memorandum (Bridges) BE 2/73: Prestressed Concrete for Highway Structures, Clause 7 to check the adequacy of the ultimate moment of resistance and the shear capacity of prestressed concrete superstructures.

2.5.5 Roller and sliding bearings. These bearings are an exception to the general principal of grouping loads into combinations since they are to be dealt with in association with permanent loads only. The reason for this is that the bearings will move and release the restraint when the frictional resistance load is at its lowest value and this is when there are no transient loads on the bridge.

3. LOADS APPLICABLE TO HIGHWAY STRUCTURES

3.1 Dead load

The dead load consists of the parts of the structure which are structural elements, excluding superimposed materials such as road surfacing, parapets, mains, ducts, miscellaneous furniture and the like.

Where an accurate assessment of the dead loads is not possible, as in the initial stages of design, the unit weights given in BS 648: 1964 may be used, except that for reinforced concrete, the unit weight may be taken as $(2400 + 5400\alpha)$ kg/m^3, where α is the proportion of reinforcing steel.

The dead load initially assumed shall be accurately checked with the actual weights to be used in construction and designs revised where necessary.

3.2 Superimposed dead load

The superimposed dead load shall be the weight of all materials which are not structural elements forming permanent loads on the structure. Consideration shall also be given to the effects of the removal in whole or in part of the superimposed dead load and also to the weight of any part of the superimposed dead load which has an effect algebraically opposite in sign to the total effect being evaluated. For superimposed dead load due to earth pressures, refer to 7.1.

3.3 Wind load

3.3.1 General. Wind loading need not be applied to single-span beam/slab and slab type highway bridges with a clear span less than 20 metres and a deck width greater than 10 metres.

The wind pressure on a structure depends on the geographical location, the local topography, the height of the structure above ground, and the horizontal dimensions and cross section of the structure or element under consideration. The maximum pressures are due to gusts which cause local and transient fluctuations about the mean wind pressure. The design gust pressures are deduced from the values of mean hourly wind speed shown in Fig 1 which are likely to be exceeded only once in 120 years, taken as the design life for highway bridges.

For footbridges, sign/signal gantries and other such ancillary highway structures, a design life of 50 years is acceptable and the mean hourly wind speed is reduced as directed in 3.3.2.1.

For the British Isles at sites less than 300 metres above sea level the gust speed derived below shall be used. At greater altitudes these wind speeds will be exceeded and the Meteorological Office shall be consulted.

3.3.2 Derivation of wind gust speed v_c. For unloaded structures the wind gust speed v_c on those parts of the structure or element on which the application of wind loading increases the effect being considered shall be taken as

$v_c = vK_1S_1S_2$ where

v	=	the mean hourly wind speed	See 3.3.2.1
K_1	=	a coefficient depending on conditions	See 3.3.2.2
S_1	=	funnelling factor	See 3.3.2.3
S_2	=	gust factor	See 3.3.2.4 and 3.3.2.5

For live loaded structures, the value of v_c shall be modified as directed in clause 3.3.2.7, 3.3.2.8 or 3.3.2.9 as appropriate.

For the remaining parts of the structure or element, whether loaded or unloaded, on which the application of wind loading gives relief to the effects under consideration, a reduced wind gust speed shall be derived as in 3.3.2.6.

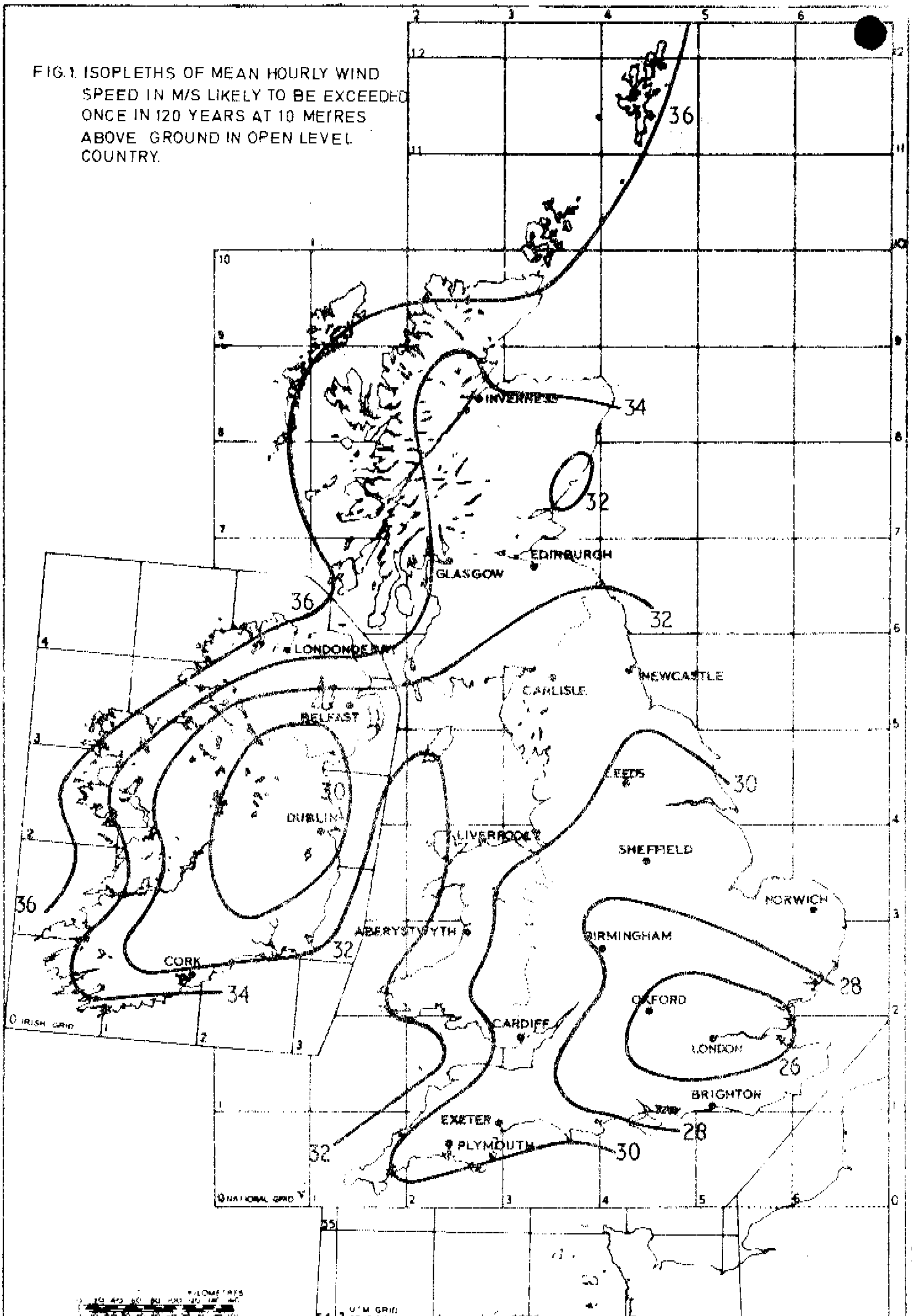


FIG.1. ISOPLETHS OF MEAN HOURLY WIND SPEED IN M/S LIKELY TO BE EXCEEDED ONCE IN 120 YEARS AT 10 METRES ABOVE GROUND IN OPEN LEVEL COUNTRY.

3.3.2.1 Mean hourly wind speed v. Values of v in metre/second for the location of the structure shall be obtained from the map of isopleths given in Fig 1. These values shall be multiplied by a reduction factor of 0.94 for the design of footbridges, sign/signal gantries and other ancillary highway structures with a 50 year design life.

3.3.2.2 Coefficient K_1. This is normally to be taken as 1.0 except where temperature effects (3.4) are considered in conjunction with wind when the value of K_1 shall be taken as 0.3.

During erection the value of K_1 shall be taken as not less than 0.85 (ie giving that speed likely to be exceeded only once in 10 years). In circumstances where a particular erection can be completed within a one or two day period for which reliable wind speed predictions can be made, this predicted speed may be used as the mean hourly wind speed and the value of K_1 shall be taken as 1.0.

3.3.2.3 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 structure is sited in the lee of a range of hills causing local acceleration of wind, a value not less than 1.1 should be taken, subject to advice from the Meteorological Office who shall be consulted wherever terrain conditions are abnormal.

3.3.2.4 Gust factor S_2. Values of S_2 are given in Table 1. These are valid for sites not exceeding 300 metres above sea level.

The height above ground level shall be measured from the foot of cliffs or steep escarpments where the structure is located near the top of such features and from mean water level for structures over tidal waters.

On vertical elements such as piers and other highway structure supports, the height shall be divided into units in accordance with the heights given in Column 1, Table 1 and the gust factor and speed derived for the centroid of each unit.

TABLE 1. VALUES OF GUST FACTOR S_2 AND HOURLY SPEED FACTOR K_2

Height above ground level Metres	Horizontal wind loaded length in metres									Hourly speed factor K_2
	20 or less	40	60	100	200	400	600	1000	2000	
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

The horizontal wind loaded length is the total length of the base of the positive or negative portions, as the case may be, of the influence line diagram for the structure or member under consideration. The most severe effect may derive from the whole or part of the loaded length using the gust speed appropriate to the actual length considered.

3.3.2.5 Modification of gust factor S_2. The values of gust factor given in Table 1 are for an exposed rural situation and take no account of the variation in ground roughness around a structure. The wind gust speeds so derived can therefore by unduly severe on wind sensitive structures located in an environment where there are many windbreaks.

Exceptionally, therefore, for footbridges, sign/signal gantries and other ancillary highway structures located in an urban or rural environment with many windbreaks of general height at least 10 metres above ground level, the values of S_2 given in 3.3.2.4 may be multiplied by a reduction factor derived from Table 2 below. At heights greater than 20 metres above ground level, no reduction shall be made.

TABLE 2. REDUCTION FACTOR FOR GROUND ROUGHNESS

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

3.3.2.6 Hourly speed factor K_2 for minimum gust speed.

3.3.2.6.1 Unloaded structure. Where wind on any part of a structure gives relief to the member under consideration, the effective coexistent value of v_c on the part affording relief shall be taken as

$$v_c = vK_1K_2$$

subject to a maximum value equal to that derived from 3.3.2.1 to 3.3.2.5.

3.3.2.6.2 Live loaded structure. The effective coexistent value of v_c on the parts affording relief shall be the lesser of

$$35 \times \frac{K_2}{S_2 \text{ for the loaded length under consideration}} \text{ and } vK_1K_2S_1 \text{ m/s}$$

3.3.2.7 Live loaded highway bridges. The wind gust speed v_c shall be taken as follows:

For loaded lengths up to 60 metres: the lesser of 33 and $vK_1S_1S_2$ m/s

For loaded lengths in excess of 60 metres: the lesser of $vK_1S_1S_2$ and

$$33 \times \frac{S_2 \text{ for the loaded length under consideration}}{S_2 \text{ for 20m loaded length}} \text{ m/s}$$

3.3.2.8 Live loaded footbridges. The wind gust speed shall be reduced to 0.7 times the value derived from 3.3.2.1 to 3.3.2.6.

3.3.2.9 Live loaded sign/signal gantries. The wind gust speed shall be reduced to 0.5 times the value derived from 3.3.2.1 to 3.3.2.6.

3.3.3 Derivation of transverse wind load. The transverse wind load P_t in Newtons shall be taken as horizontal unless conditions* change the direction of the wind and shall be derived from the following equation:

* The local conditions which might be taken into account when deciding that the incidence of the wind is other than horizontal include

a. the general gradient of the local topography, and
b. the proximity of high buildings, cuttings, cliff faces etc.

$P_t = q\, C_D\, A_1$ where

q = dynamic pressure head = $0.613\, v_c^2$ in N/m^2 where v_c is in m/s

C_D = drag coefficient See 3.3.3.2 to 3.3.3.8

A_1 = the net area in m^2 in normal projected elevation of the structure or element. See 3.3.3.9.

3.3.3.1 Transverse wind load P_t. The transverse wind load shall be considered as follows:

3.3.3.1.1 Erection stages for all structures. The effects of wind shall be considered at all stages of construction.

3.3.3.1.2 Non-truss highway bridges

i. Unloaded bridge with open parapets. The transverse wind load P_t is derived separately for the following:

a. deck
b. windward parapet
c. leeward parapet

Where there are more than two parapets, irrespective of the width of the bridge only those two parapets having the greatest unshielded effect shall be considered.

ii. Unloaded bridge with solid parapets. P_t is derived for the deck, which includes the windward and leeward parapets. Where there are safety fences or additional parapets, P_t should be derived separately for those portions of such elements above the top of the solid parapets.

iii. Live loaded bridge with open or solid parapets. P_t is derived from the values of C_D and A_1 given in 3.3.3.3 and 3.3.3.9.2 respectively which include for the effects of parapets. Where safety fences or leeward parapets are higher than the surface representing live load, those portions above the vertical surface shall also be considered.

iv. Superstructures separated by an air gap. Where two generally similar superstructures are separated transversely by a gap not exceeding 1 metre the load on the windward superstructure 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.

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

3.3.3.1.3 Truss girder highway and footbridges

i. Unloaded bridge. The transverse wind load P_t shall be derived separately for:

a. windward and leeward truss girders
b. deck
c. windward and leeward parapets

No wind loading need be considered on the projected areas of:

a. the windward parapet screened by the windward truss or vice versa
b. the deck screened by the windward truss or vice versa
c. the leeward truss screened by the deck
d. the leeward parapet screened by the leeward truss or vice versa

ii. Live loaded bridge. P_t shall be derived for the unloaded bridge and for the vertical surface representing live loading.

In addition to those areas screened from the effects of wind as in i. above, wind loading can also be disregarded on projected areas of:

a. the live load screened by the windward truss
b. the leeward truss screened by the live load

3.3.3.1.4 Non-truss footbridges

i. Unloaded bridge. Where the ratio b/d as defined in Fig 4 is greater than or equal to 1.0, the transverse wind load P_t is derived for the deck including windward parapet. The leeward parapet is disregarded.

Where the ratio b/d is less than 1.0, P_t is derived as in 3.3.3.1.2.

ii. Live loaded bridge. Where the ratio b/d is greater than or equal to 1.0, P_t is derived for the deck including the vertical surface representing pedestrian loading and those parts of the windward parapet above this vertical surface. The leeward parapet is disregarded.

Where the ratio b/d is less than 1.0, P_t is derived as in 3.3.3.1.2.

3.3.3.1.5 Parapets and safety fences. The transverse wind load P_t shall be derived for the equivalent solid area of the element.

3.3.3.1.6 Piers and Structure supports. The transverse wind load P_t shall be derived for each pier or support without any allowance for shielding.

3.3.3.1.7 Sign/Signal gantries. For unclad portions of open web girders, P_t shall be derived as in 3.3.3.1.3 above.

For clad portions of girders and for solid web girders, the leeward girder shall be disregarded.

3.3.3.2 Drag coefficient for erection stages for beam and box, plate and truss girder structures

The following clauses refer to discrete beams or girders before deck construction or other infilling.

3.3.3.2.1 Single beams or box girders with vertical sides. C_D shall be derived from Fig 5 as appropriate in accordance with the ratio b/d as indicated in Fig 4.

3.3.3.2.2 Two or more beams or box girders. Where the ratio of the clear distance between 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 above 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 above for the single beams or boxes where n is the number of beams or box girders.

3.3.3.2.3 Single plate girders. C_D shall be taken as 2.2.

3.3.3.2.4 Two or more plate girders. C_D for the combined structure shall be taken as $2(1 + b/20d)$, but not more than 4,

where b = distance centre to centre of adjacent girders

d = depth of the windward girder

3.3.3.2.5 Truss girders. The discrete stages of erection shall be considered in accordance with 3.3.3.4.1.

3.3.3.3 Drag coefficient for non-truss highway bridges (see Fig 2).

3.3.3.3.1 Unloaded bridge. The drag coefficient C_D shall be derived from Fig 5 in accordance with the ratio b/d as indicated in Fig 4. Where designs do not accord with Fig 4 and for thos types of bridge illustrated in Fig 3, drag coefficients shall be ascertained from wind tunnel tests.

3.3.3.3.2 Live loaded bridge. The drag coefficient C_D shall be derived from Fig 5 for the structure and traffic. The value of d and the ratio b/d shall be taken from Fig 4.

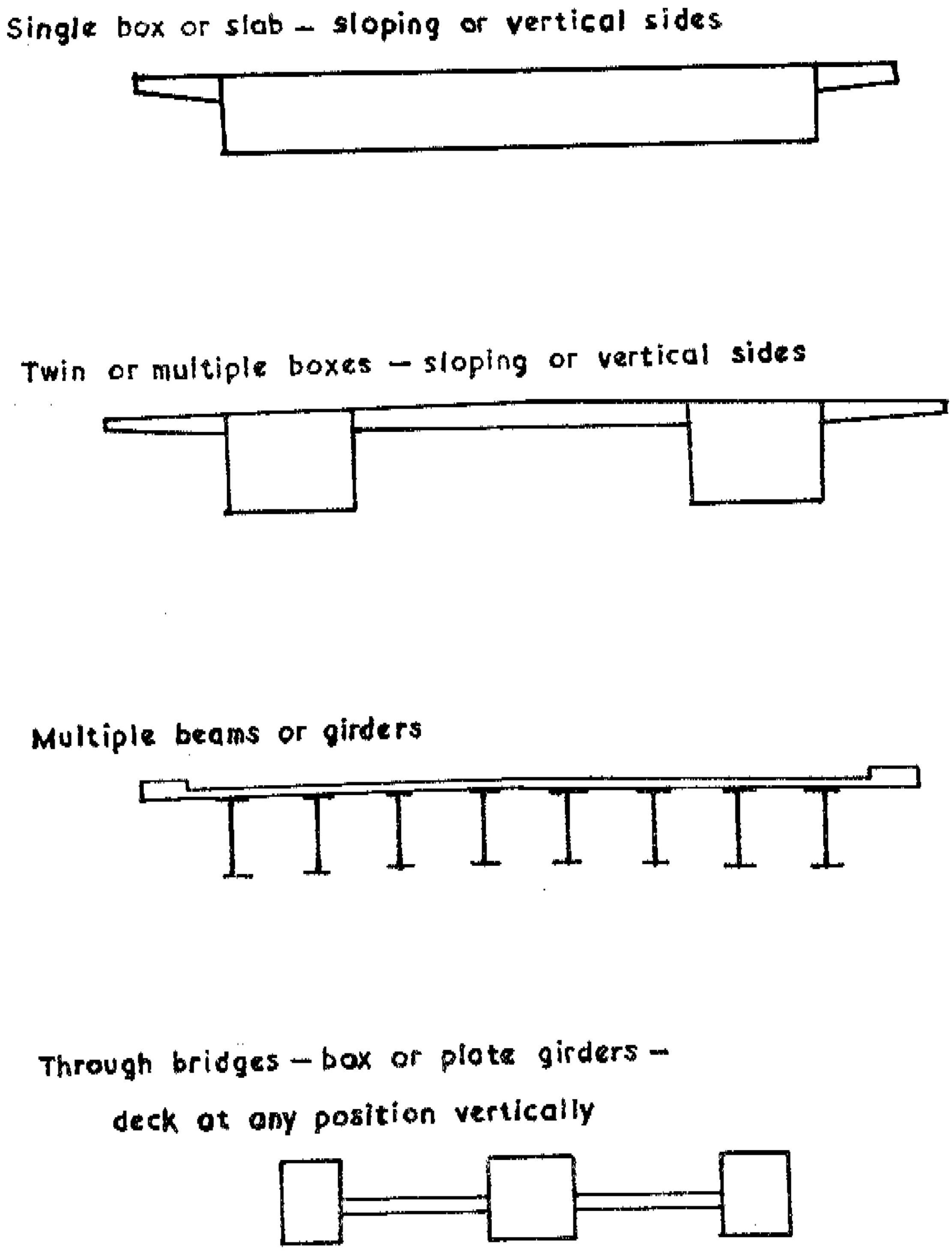


Fig. 2. Typical Highway Bridge Cross Sections for which
Drag Coefficients are derived from Clause 3.3.3.3

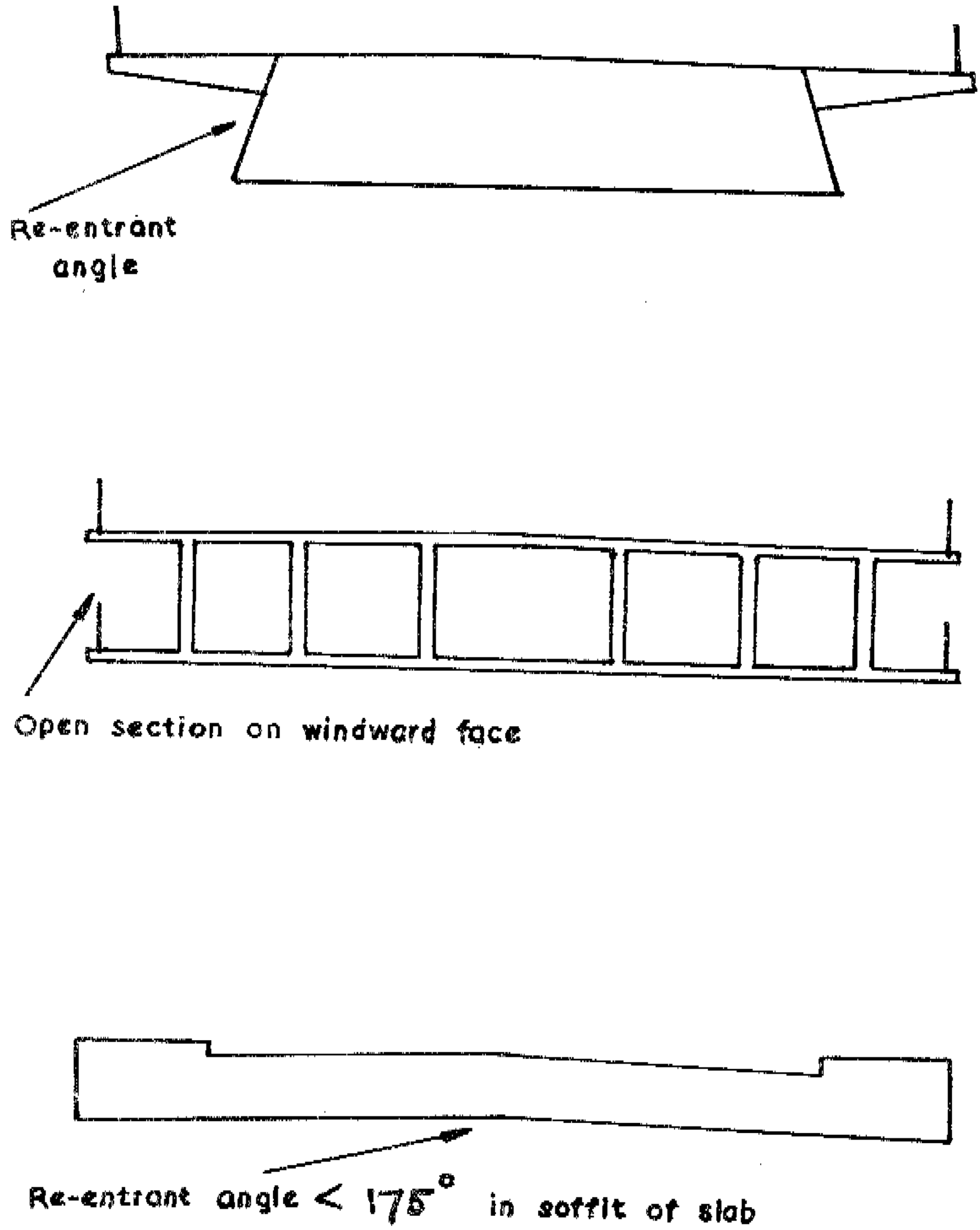
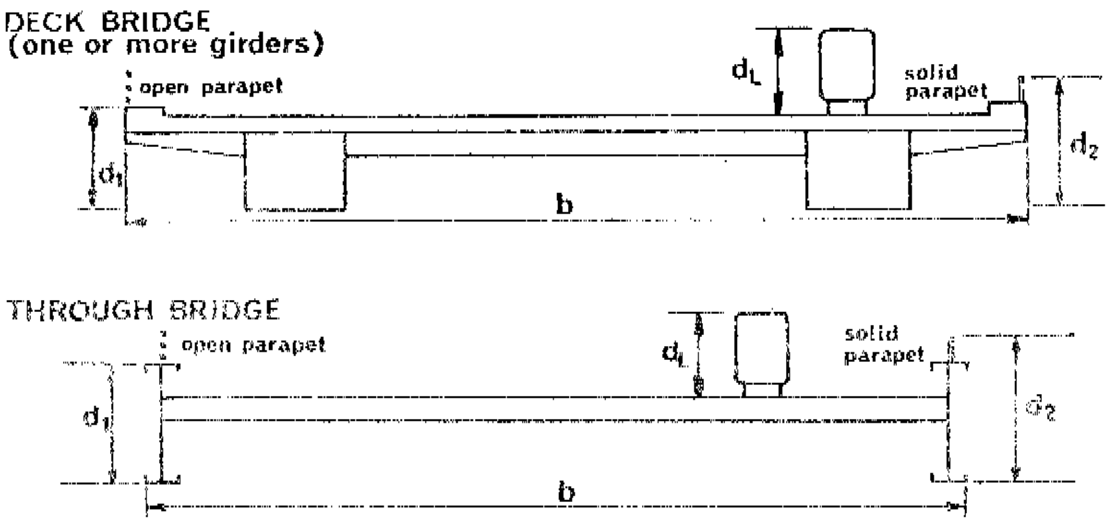


Fig. 3. Bridge Cross Sections for which Drag Coefficients must be derived from Wind Tunnel Tests.

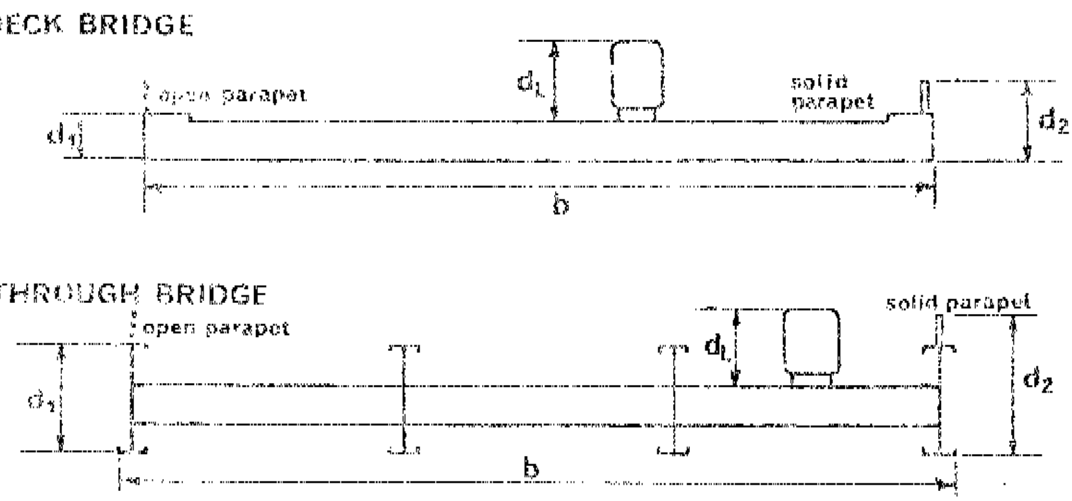
(i) DEPTH OF SUPERSTRUCTURE (d_1 or d_2) GREATER THAN LIVE LOAD HEIGHT (d_L)



d = depth d_1 for bridges with open parapets with or without live load.

d = depth d_2 for bridges with solid parapets with or without live load.

(ii) DEPTH OF SUPERSTRUCTURE (d_1 or d_2) LESS THAN LIVE LOAD HEIGHT (d_L)



d = depth d_1 for bridges with open parapets without live load.

d = depth d_2 for bridges with solid parapets without live load.

d = depth d_L for bridges with open or solid parapets with live load.

FIG. 4 DIMENSIONS OF b AND d TO BE USED IN DERIVING THE DRAG COEFFICIENT FOR NON TRUSS HIGHWAY BRIDGES.

NOTE: In all cases, b and d are to be taken as if the cross-sections were normal to the vertical (ie not superelevated) but see Note 4 to Fig 5 for the treatment of superelevation.

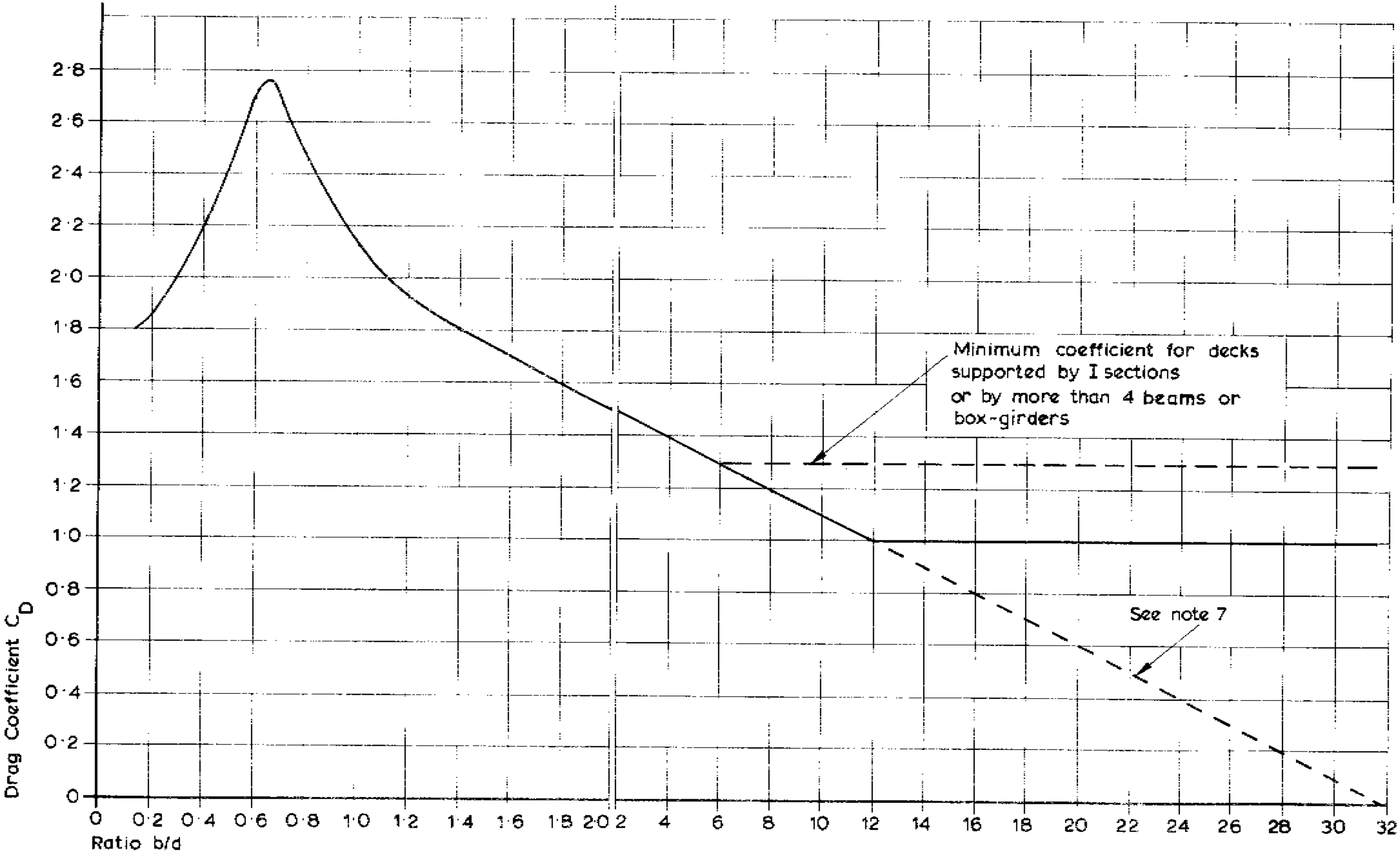


Fig. 5 Drag Coefficient C_D for Bridges other than Truss Girder Bridges.

NOTES FOR FIG 5

1. These values are given for vertical elevation.

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%.

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

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

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

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

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

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

7. Where two generally similar superstructures are separated transversely by a gap not exceeding 1 metre the drag coefficient for the combined structure shall be obtained by taking as b the combined width of the superstructure.

In determining the wind load on the leeward superstructure only (taken as the difference between the loads calculated for the combined and the windward structures), where b/d is greater than 12, the broken line in Fig 5 shall be used to derive C_D.

3.3.3.4 Drag coefficient for truss girder highway and footbridges

3.3.3.4.1 Unloaded bridge. The drag coefficient C_D for each truss and for the deck shall be derived separately as follows.

i. For a windward truss C_D shall be taken from Table 3.

TABLE 3. DRAG COEFFICIENT C_D FOR A SINGLE TRUSS

Solidity Ratio	For flat sided members	For round members where d is diameter of member	
		$dv_c < 6m^2/s$	$dv_c \geq 6m^2/s$
0.1	1.9	1.2	0.7
0.2	1.8	1.2	0,8
0.3	1.7	1.2	0.8
0.4	1.7	1.1	0.8
0.5	1.6	1.1	0.8

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

ii. For the leeward truss of a bridge with 2 trusses, the drag coefficient shall be taken as ηC_D. Values of η are given in Table 4.

TABLE 4. SHIELDING FACTOR η

Spacing Ratio	Value of η for solidity ratio of:				
	0.1	0.2	0.3	0.4	0.5
Less than 1	1.0	.90	.80	.60	.45
2	1.0	.90	.80	.65	.50
3	1.0	.95	.80	.70	.55
4	1.0	.95	.85	.70	.60
5	1.0	.95	.85	.75	.65
6	1.0	.95	.90	.80	.70

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

iii. Where a bridge has more than 2 trusses, the drag coefficient for the truss adjacent to the windward truss shall be derived as in ii. above. The coefficient for all other trusses shall be taken as equal to this value.

iv. For the deck construction the drag coefficient C_D shall be taken as 1.1.

3.3.3.4.2 Live loaded bridge. The drag coefficient C_D on unshielded parts of the live load shall be taken as 1.45.

3.3.3.5 Drag coefficient for non-truss footbridges

3.3.3.5.1 Unloaded bridge. C_D shall be taken as 2.0 for the footbridge except where b/d is less than 1.0 when C_D shall be derived from Fig 5.

For footbridges similar in configuration to those bridges in Fig 3, C_D shall be derived from wind tunnel tests.

Where a footbridge is subject to inclined wind the value of C_D derived above shall be increased by 15%.

3.3.3.5.2 Live loaded bridge. C_D for the live loaded footbridge shall be taken as 2.0 except where b/d is less than 1.0 when C_D for the live loaded bridge shall be derived from Fig 5.

3.3.3.6 Drag coefficients for parapets and safety fences. For the windward parapet or fence the drag coefficient C_D for the element shall be taken from Table 5.

TABLE 5. DRAG COEFFICIENTS FOR PARAPETS AND SAFETY FENCES

Circular sections	1.2
Other sections including solid parapets	2.2

Alternatively, drag coefficients derived from wind tunnel tests may be used.

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 5 for the two elements having the greatest unshielded effect.

3.3.3.7 Drag coefficient for piers and structure supports. The drag coefficient C_D shall be taken from Table 6. For piers or supports with cross sections dissimilar to those given in Table 6, wind tunnel tests shall be carried out.

C_D shall be derived for each pier or support without reduction for shielding.

3.3.3.8 Drag coefficient for sign/signal gantries. For gantries with solid web main girders spanning between the columns, or with solid or open web main girders that are fully clad in frontal elevation, the drag coefficient shall be taken as :

i. 1.6 for bending moment on girder, and

ii. 1.8 for shear on girder and reaction on any supporting column. (A higher value is required for shear and reaction than for bending because of the asymmetry of the pressure distribution across the face of the gantry due to inclined wind).

TABLE.6.

DRAG COEFFICIENTS FOR SUPPORTS OF HIGHWAY STRUCTURES

PLAN SHAPE	$\frac{t}{b}$	C_D FOR SUPPORT $\frac{\text{HEIGHT}}{\text{BREADTH}}$ RATIOS OF 1	2	4	6	10	20	40
WIND b	$\leq \frac{1}{4}$	1·3	1·4	1·5	1·6	1·7	1·9	2·1
	$\frac{1}{2}$	1·3	1·4	1·5	1·6	1·8	2·0	2·2
	$\frac{2}{3}$	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
	$1\frac{1}{2}$	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
t b	≥ 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 $tv_c \geq 6 m^2/sec$		0·5	0·5	0·5	0·5	0·5	0·6	0·6
CIRCLE WITH SMOOTH SURFACE WHERE $tv_c < 6 m^2/sec$ ALSO. CIRCLE WITH ROUGH SURFACE OR WITH PROJECTIONS.		0·7	0·7	0·8	0·8	0·9	1·0	1·2

Notes

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

2. For a support with $t/b \geq \frac{1}{4}$, located in a built-up area where turbulent flow may occur, a lesser drag coefficient may apply and C_D shall be obtained from wind tunnel tests.

3. For a rectangular support with radiused corners, the value of C_D derived from the above table shall be multiplied by (1-1.5r/b) or 0.5, whichever is lesser.

4. For a support with triangular nosings, C_D shall be derived as for the rectangle encompassing the outer edges of the support.

5. For a support tapering with height, C_D shall be derived for each of the unit heights into which the support has been subdivided (see 3.3.2.4). Mean values of t and b for each unit height shall be used to evaluate t/b. The overall support height and the mean breadth of each unit height shall be used to evaluate height/breadth.

3.3.3.9 Derivation of net area A_1

3.3.3.9.1 Erection stage for all structures. The net area A_1 shall be the appropriate equivalent solid area of the structure.

3.3.3.9.2 Non-truss highway bridges. For unloaded and live loaded bridges, the net area A_1 in m^2 is derived using the appropriate value of d in accordance with Fig 6 below.

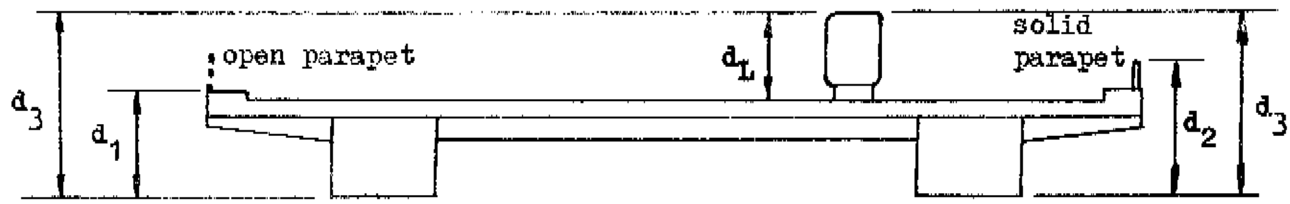


FIG 6. DEPTH d TO BE USED IN ASSESSING THE NET EXPOSED AREA A_1 FOR NON TRUSS HIGHWAY BRIDGES

$d = d_1$ for bridges with open parapets and without live load

$d = d_2$ for bridges with solid parapets and without live load

$d = d_2$ or d_3 whichever is larger when live load is present

$d_L = 2.5$ m from the carriageway for highway bridges

3.3.3.9.3 Truss girder highway and footbridges

i. Unloaded bridge. The value of A_1 for each truss being considered shall be taken as the equivalent solid area of the truss. A_1 for the deck shall be based on the full depth of the deck.

ii. Live loaded bridge. The area A_1 on live load shall be derived using the value of d_L defined in Fig 6. The area of the trusses and deck shall be as for the unloaded bridge except that where parts of the trusses are above the carriageway or footway, those portions of the trusses within the projected depth d_L shall be disregarded.

3.3.3.9.4 Non-truss footbridges

i. Unloaded bridge. The value of A_1 shall comprise the equivalent solid area of the windward exposed face of the deck and parapet.

ii. Live loaded bridge. The value of A_1 shall comprise the equivalent solid area of deck below footway level, the vertical surface of height $d_L = 1.25$ metres above the footway representing pedestrian loading together with the net exposed area of the parts of the windward parapet which are below deck soffit level or more than 1.25 metres above the footway.

3.3.3.9.5 Parapets and safety fences.

i. Open parapet or fence. A_1 = equivalent solid area.

ii. Solid parapet or fence. A_1 = actual solid area.

3.3.3.9.6 Piers and structure supports. The net projected area A_1 for each pier or support shall be taken.

3.3.3.9.7 Sign/signal gantries. For solid web girders and clad portions of solid or open web girders, the net exposed area of the windward face shall be taken. For unclad portions of open web girders, the equivalent solid depth of each girder shall be taken.

On live loaded gantries, no allowance need be made for the exposed area of live loading.

3.3.4 Derivation of longitudinal wind load. The longitudinal wind load P_L in Newtons shall be derived as follows.

3.3.4.1 All bridges other than truss girder bridges

P_L = 0.25 q C_D A_1 where

q = dynamic pressure head = 0.613 v_c^2 in N/m^2 where v_c is in m/s

C_D = drag coefficient (excluding reduction for inclined webs) as defined in 3.3.3.3 and 3.3.3.5 for unloaded bridges, but not less than 1.3.

A_1 = as defined in 3.3.3.9 for unloaded bridges

3.3.4.2 Truss girder bridges

P_L = 0.5 P_t where P_t = the appropriate transverse wind load for the unloaded superstructure.

3.3.4.3 Live load on all bridges

P_L = 0.5 q C_D A_1

q = dynamic pressure head = 0.613 v_c^2 in N/m^2 where v_c is in m/s

C_D = drag coefficient, taken as 1.45

A_1 = area of live load as defined in 3.3.3.9

3.3.4.4 Parapets and safety fences.

3.3.4.4.1 With vertical infill members

P_L = 0.8 P_t

3.3.4.4.2 With 2 or 3 horizontal rails only

P_L = 0.4 P_t

Where P_t = the appropriate transverse wind load on the element.

3.3.4.5 Cantilever brackets extending outside main girders or trusses

P_L = 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 brackets. The drag coefficient C_D shall be taken from Table 5.

3.3.4.6 Piers and structure supports

P_L = q C_D A_2 where

q = dynamic pressure head = 0.613 v_c^2 in N/m^2 where v_c is in m/s

C_D = drag coefficient, taken from Table 6, with values of b and t interchanged.

A_2 = net projected area in the longitudinal wind direction (in m^2).

3.3.4.7 Sign/signal gantries

P_L = q C_D A_2 where

q = dynamic pressure head = 0.613 v_c^2 in N/m^2 where v_c is in m/s

C_D = drag coefficient, taken as 6.0

A_2 = the net exposed area in m^2 in end view of the sign and gantry structure, 3.3.5 Derivation of vertical load. An upward or downward vertical load P_v in Newtons shall be derived from the following equation.

P_v = $q\ C_L\ A_3$ where

q = dynamic pressure head = $0.613\ v_c^2$ in N/m^2 where v_c is in m/s

C_L = Lift coefficient derived from Fig 7

A_3 = net area in plan in m^2.

Where the angle of superelevation of a structure is between 1° and 5°, C_L shall be taken as $\pm$ 0.75. For angles exceeding 5°, C_L shall be derived from tests.

Where inclined wind may affect the structure, C_L shall be taken as $\pm$ 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 the 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.

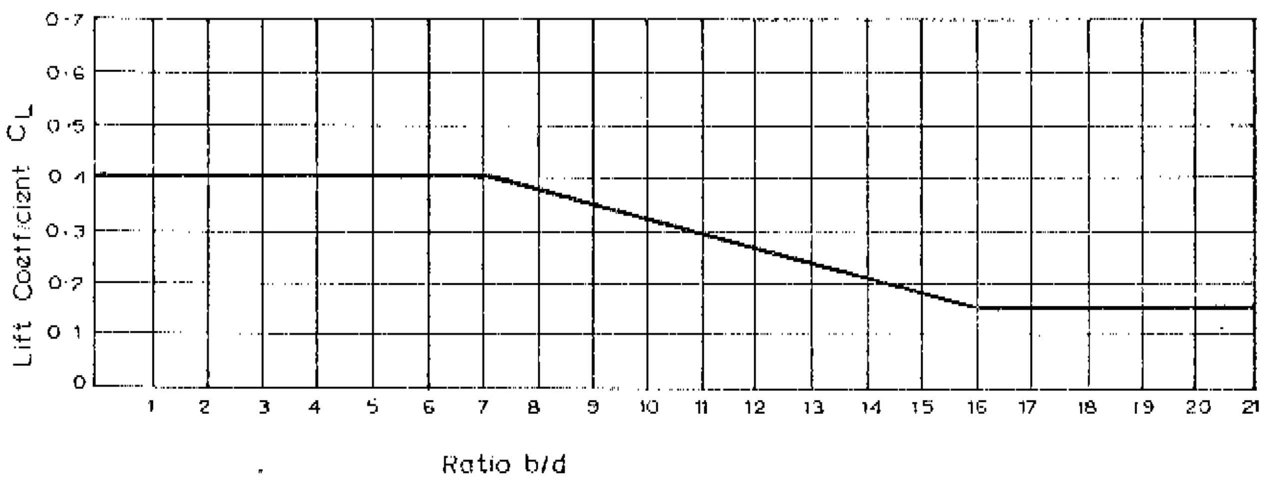


FIG 7. VALUES OF LIFT COEFFICIENT C_L

3.3.6 Overturning effects. Where overturning effects are being investigated the wind load shall also be considered in combination with live load.

On highway bridges where the vertical live load has a relieving effect, it shall be limited to 6kN/m of bridge in one notional lane only.

3.3.7 Application of wind loads. Appropriate wind loads shall be taken to act on the loaded and on the unloaded structure. The wind shall be taken as blowing in the same direction on all parts of the structure or element.

3.3.7.1 Combination of wind loads. The wind loads P_t, P_L and P_v shall be combined as follows:

a. P_t alone

b. P_t in combination with $\pm P_v$

c. P_L alone

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

e. P_t in combination with $P_L \pm 0.5\ P_v$

3.3.7.1.1 Highway and footbridge. Combinations a, b, c and d shall be considered.

3.3.7.1.2 Sign/signal gantries. Combinations a, b, c and e shall be considered.

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

3.4 Temperature effects

The minimum and maximum temperatures and the range of temperatures to be considered as affecting the structure are referred to as the minimum, maximum and range of effective bridge temperatures respectively. These effective values take into account the influence of solar radiation and shall be derived from the shade air temperatures in accordance with the following clauses.

3.4.1 Derivation of minimum and maximum shade air temperatures. Subject to the reservations in 3.4.1.6 these shall be derived from Figs 8 and 9 respectively which show isotherms for annual minimum and maximum shade air temperature for the British Isles for a return period of 120 years, taken as the design life for highway bridges.

3.4.1.1 Footbridges and sign/signal gantries. For footbridges and sign/signal gantries, a return period of 50 years may be adopted and the shade air temperatures adjusted in accordance with 3.4.1.4.

3.4.1.2 Equipment having a life of 50 years or less. Carriageway joints and similar equipment which will be replaced during the life of the structure shall be designed for temperatures related to a 50 year return period. The values of shade air temperature from Figures 8 and 9 may be corrected for this return period in accordance with 3.4.1.4.

3.4.1.3 During erection. For erection, a 20 year return period may be used and the shade air temperatures adjusted in accordance with 3.4.1.4.

In circumstances where a particular erection can 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.

3.4.1.4 Adjustment for 50 and 20 year return periods. For return periods of 50 and 20 years, the values derived from Figs 8 and 9 shall be adjusted by the addition of the appropriate values given in Table 7.

TABLE 7. CORRECTION FOR 50 AND 20 YEAR RETURN PERIODS

Return Period (years)	Correction to be added (°C) to Annual minimum shade air temp	Annual maximum shade air temp
50	+2	-1.5
20	+4	-2.5

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

3.4.1.6 Divergence from minimum shade air temperature. There are locations where the minimum values diverge from the value given in 3.4.1 as, for example, frost pockets and in sheltered low lying areas where the minimum may be substantially lower, or urban areas, except London and coastal sites, where the minimum may be higher than that indicated by Fig 8. In coastal areas, values are likely to be 1°C higher than map values.

These divergences shall be investigated and taken into consideration. Consultation with the local Meteorological Office is likely to be helpful in these conditions.

FIG. 8 ANNUAL MINIMUM SHADE AIR TEMPRATURE (°C) LIKELY TO OCCUR ONCE IN 120 YEARS AT MEAN SEA LEVEL

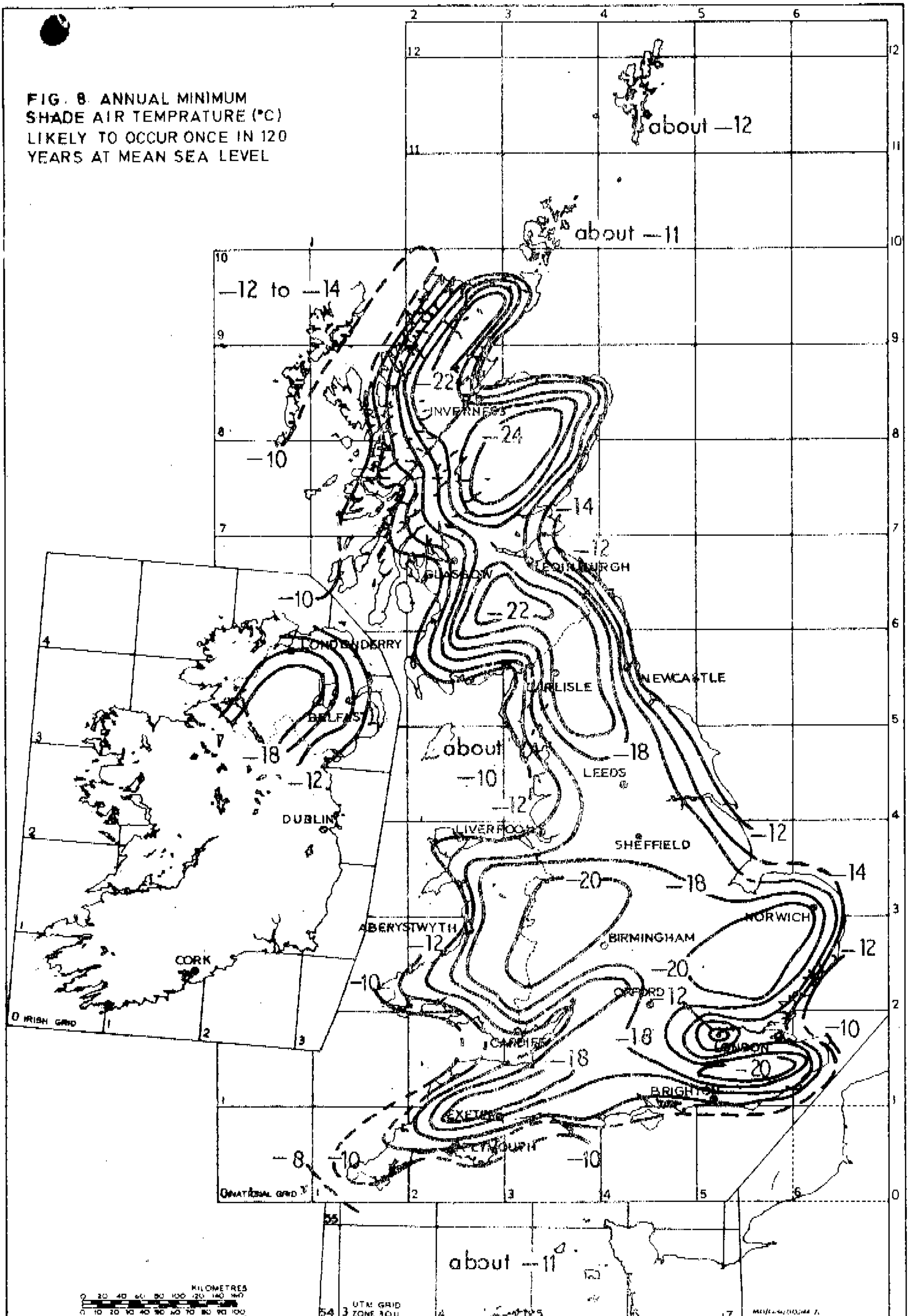
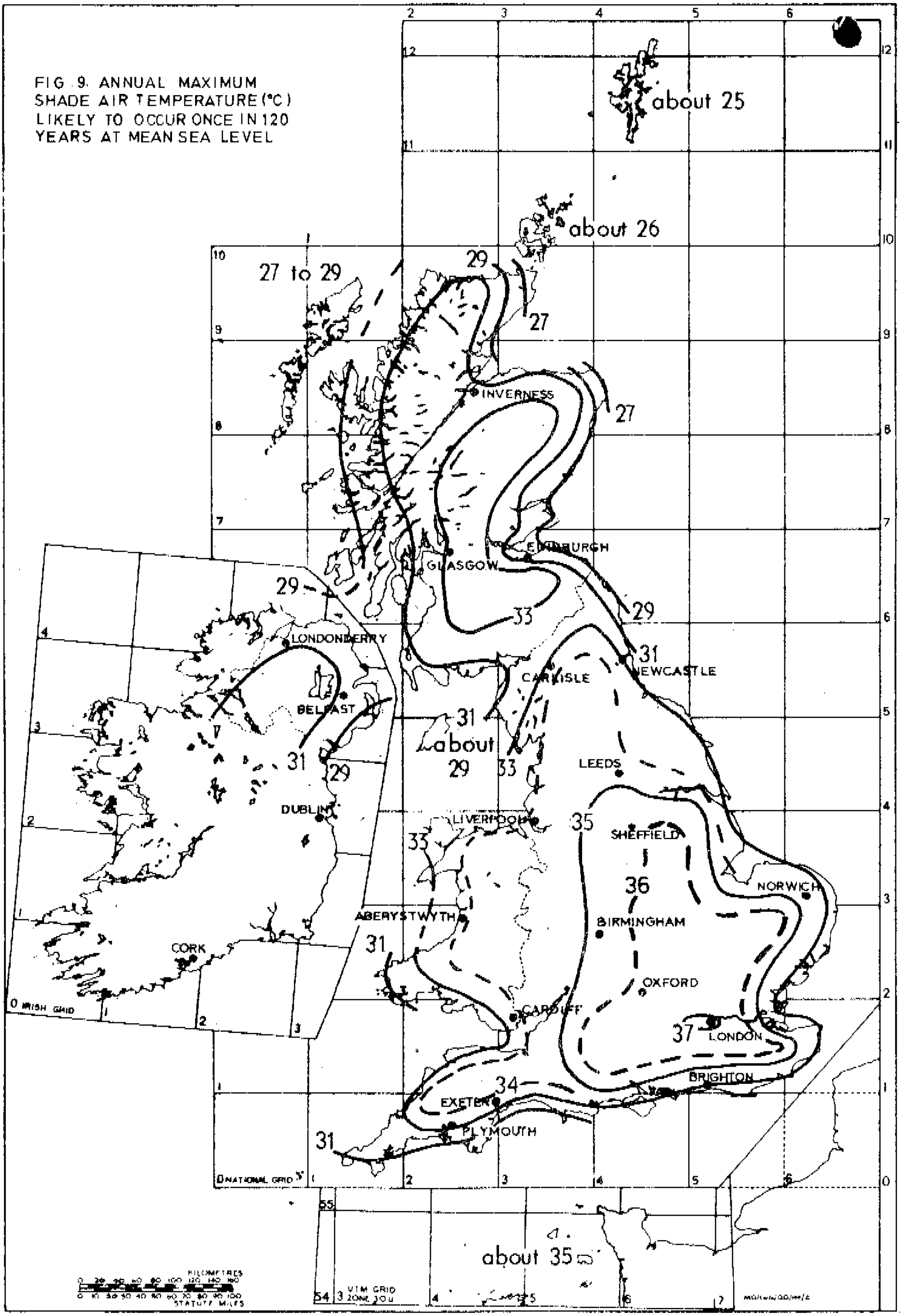


FIG. 9. ANNUAL MAXIMUM SHADE AIR TEMPERATURE (°C) LIKELY TO OCCUR ONCE IN 120 YEARS AT MEAN SEA LEVEL



3.4.2 Minimum and maximum effective bridge temperatures. The minimum and maximum effective bridge temperatures for different types of construction, shall be derived from the minimum and maximum shade air temperatures by reference to Tables 8 and 9 respectively. The different types of construction are shown in Fig 10.

The minimum and maximum effective bridge temperatures and the types of construction are also used, where appropriate for sign/signal gantry structures.

3.4.2.1 Combination with wind effects. Where loads due to temperature changes are to be considered in combination with maximum wind loads, the minimum and maximum effective bridge temperatures shall be taken as 0°C and 20°C respectively for all localities and all types of construction. (See 3.3.2.1 for reduced wind effects in combination with maximum effective temperature range).

3.4.3 Effects of temperature difference. Load effects resulting from differences of temperature within the structure and parts of the structure shall be derived from the data given in Fig 10, which represents linearised differences based on the curves of Fig B1 in Appendix B. Fig B1 may, if desired be used in preference to Fig 10.

Temperature differences are sensitive to the amount of surfacing and the differences shown relate only to the given depths. Further information for other depths of surfacing can be found in the reference in the footnote*.

These differences may occur more than once in any year and at any time during the year and are independent of effective bridge temperatures. Their effects must therefore be considered in combination with the effects arising from the minimum and maximum effective bridge temperatures or such values between these extremes as cause a more severe effect.

Methods of computing temperature differences can be found in the reference in the footnote**. For conditions not covered by Fig 10 these may be adopted as a basis for estimating the temperature differences.

Where any structure, or part of a structure is fully sealed, there may be a build up of pressure which should be taken into consideration.

3.4.3.1 Sign/signal gantries. Effects of temperature difference need not be considered in the design of sign/signal gantries.

3.4.3.2 Combination with wind effects. Effects of temperature difference need not be considered in combination with wind effects.

3.4.4 Coefficient of thermal expansion. The coefficient of thermal expansion per 1.0°C shall be taken as 12×10^{-6} for structural steel. For reinforced concrete, values of the coefficient are given in Clause 4.2 of BE 1/73.

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* TRRL Report LR765 'Temperature differentials in bridges' available from Transport & Road Research Laboratory, Crowthorne, Berkshire.

** TRRL Report LR561 'The calculation of the distribution of temperature in bridges' available from Transport & Road Research Laboratory, Crowthorne, Berks.

Minimum shade air temperature	Minimum effective bridge temperature		
	Group 1 and 2	Group 3	Group 4
-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 9. MAXIMUM EFFECTIVE BRIDGE TEMPERATURE °C

Maximum shade air temperature	Maximum effective bridge temperature		
	Group 1 and 2	Group 3	Group 4
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

1. The group types are given in Fig 10.

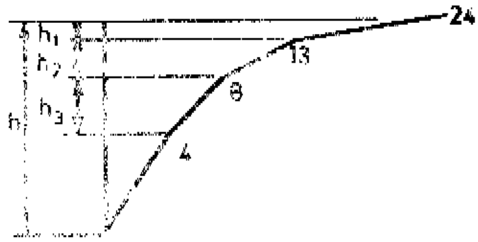
2. The above effective bridge temperatures are dependent on the depth of surfacing on the bridge deck; depths assumed are: Groups 1 and 2, 40mm; Groups 3 and 4, 50mm. Further information for other depths can be found in the reference in the footnote*.

*TRRL Report LR765 'Temperature differentials in bridges' available from Transport and Road Research Laboratory, Crowthorne, Berkshire.

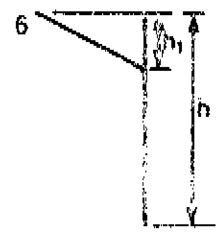
TEMPERATURE DIFFERENCES FOR DIFFERENT TYPES OF CONSTRUCTION

GROUP	TYPE OF CONSTRUCTION	TEMPERATURE DIFFERENCE °C

1 Steel deck on steel box girders



h_1 = 0.1 metres
h_2 = 0.2 metres
h_3 = 0.3 metres



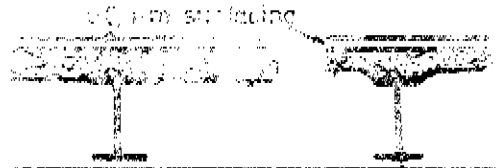
h_1 = 0.5 metres

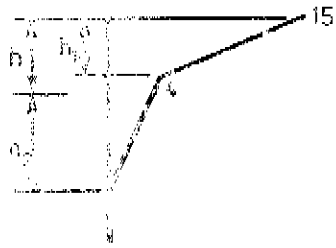
2. Steel deck on steel truss or plate girders — Use differences as for Group 1

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

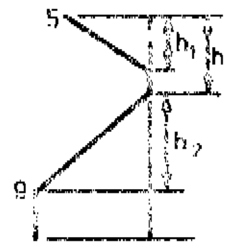
50 mm surfacing

50 mm surfacing



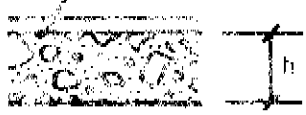


h_1 = 0.75h
h_2 = 0.4 metres



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

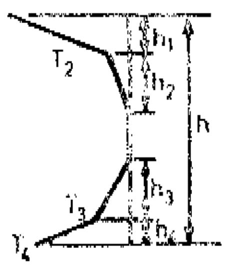
50mm surfacing



50mm surfacing

h_1 = 0.6 h $\not>$ 0.2m
h_2 = 0.4 $(h - h_1)$ $\not>$ 0.3m
h_4 = 0.4 $(h - h_1)$ $\not>$ 0.1m
h_3 = $h - (h_1 + h_2 + h_4)$ $\not>$ 0.2m

h metres	T_1 °centigrade	T_2	T_3	T_4
0.2	9.5	0.5	0.2	0.5
0.4	13.5	0.75	0.5	2.5
0.6	15.0	1.0	1.0	4.0
0.8	16.0	1.75	1.5	5.0
≥ 0.9	16.0	2.0	1.5	5.0



h_1 = 0.4 h $\not>$ 0.2m
h_2 = 0.3 h $\not>$ 0.3m
h_4 = 0.4 $(h - h_1)$ $\not>$ 0.1m
h_3 = $h - (h_1 + h_2 + h_4)$ $\not>$ 0.2m

h metres	T_1 °centigrade	T_2	T_3	T_4
0.2	3.5	0.5	0.25	1.5
0.4	6.5	0.75	0.5	3.5
0.6	8.5	1.0	1.5	5.0
0.8	10.0	1.5	2.5	6.5
≥ 0.9	10.5	2.0	3.5	7.0

3.5 Restraint at bearings

The loads due to friction or other restraint of thermal movement at bearings shall be taken into account as follows:

3.5.1 Roller and sliding bearings. For rollers with bearing plates at the circumference and for sliding bearings, loads shall be derived only from Dead load (3.1) and Superimposed Dead Load (3.2) using the coefficients of friction given in Table 10 (the coefficients are based partly on TRRL Report LR 382: Notes on Bridge Bearings). Where an appropriate coefficient of friction is not given in Table 10 this shall be established by testing.

TABLE 10 - COEFFICIENTS OF FRICTION

Bearing type	Coefficient of friction
Roller bearings*:	
Through-hardened special steel with finely ground finish 425-450 HB.	0.006
1 or 2 rollers with as-turned finish in: Mild steel to BS 4360 grade 43 110-150 HB. High tensile steel to BS 4360 grade 50 160-190 HB. Grey cast iron to BS 1452 grade 23 190-220 HB.	0.03
As above but with more than 2 rollers	0.05
Sliding bearings**	
Grey cast iron on grey cast iron	0.35
Steel on steel or steel on cast iron	0.80

* The rollers shall be turned so that opposite faces shall be parallel within a tolerance of ± 0.05 mm and the seatings shall be ground in the direction of travel of the rollers so that the variation in flatness at any section parallel to the axis of the roller shall not exceed ± 0.05 mm.

** The plates of sliding bearings shall be ground in the direction of travel so that the variation in flatness of either plate across any section shall not exceed ± 0.05 mm.

3.5.2 Elastomeric bearings. Where expansion and contraction is accomodated by shear in the elastomer, the load developed shall be derived in accordance with Technical Memorandum (Bridges) BE 1/76: Design Requirements for Elastomeric Bridge Bearings.

3.5.3 Setting of bearings. All bearings shall be set so that their displacement can accomodate expansion and contraction due to the temperature difference between the effective bridge temperature at which the bearing is set and the minimum and maximum effective bridge temperatures respectively.

3.5.4 Flexure of piers. Where expansion and contraction is accomodated by flexure of piers, the load due to such movements shall be that required to displace the pier at bearing level to accomodate expansion and contraction of the superstructure due to the temperature difference between the effective bridge temperature at which the pier is joined to the superstructure and the minimum and maximum effective bridge temperatures respectively.

3.6 Effects of shrinkage and creep in concrete

Where an element is restrained against change of length due to shrinkage this shall be taken into consideration. Due allowance may be made for the relieving effect of creep in concrete.

3.7 Erection loads and effects

The weight of all permanent and temporary materials, together with all other loads and effects which can operate on any part of the structure shall be taken into account. These weights and loads together with the positions at which they act on the structure shall be accurately assessed and where there is uncertainty about the accuracy of these assessments they shall be increased by a value sufficiently large to ensure that the design loading is not under-estimated.

Where the weight of any part of the permanent or temporary material taken into account has an effect algebraically opposite in sign to the total effect being evaluated this shall be considered.

Wind and temperature effects are to be taken in combination with erection loads.

3.8 Differential settlement

Where differential settlement may affect the structure in whole or in part, the effects of this shall be taken into account.

In assessing the amount of differential movement to be provided for, the Engineer must bear in mind the extent to which its effects will be observed and remedied before damage ensues.

3.9 Exceptional loads.

Where other loads not enumerated in this specification may be encountered, for example, the effects of earthquakes, stream flow or ice packs, these shall be taken into account.

Snow loading should be considered in accordance with local conditions; for those which prevail in Great Britain this loading may generally be ignored but there are circumstances, eg where dead load stability is critical or the case of opening bridges, when this effect should be taken into account.

4. LIVE LOADING

4.1 Standard highway live loading

The effects of the standard live loadings shall, where appropriate, be distributed in accordance with a rigorous distribution analysis or on the basis of data derived from suitable tests.

4.1.1 Type HA and type HB loadings. Standard highway loadings consist of type HA, which is the normal design loading for Great Britain and type HB which is the abnormal unit loading, applied in accordance with 4.2. Details of these loadings as given in Appendix A of the Standard are to be followed subject to the modifications in this Memorandum.

The relevant clauses of the Standard are listed in Appendix A of this Memorandum.

4.1.2 Width and number of traffic lanes to be used in conjunction with standard highway live loadings.

4.1.2.1 Definition of a carriageway. For the purposes of this clause the carriageway shall be the paved road surfacing between raised kerbs as shown in Technical Memorandum H9/71 excluding the hardstrip* adjacent to the offside lane of:-

a. a dual three lane rural motorway,

b. a slip road of a rural motorway, having normal** crossfall,

c. a slip road of a rural all-purpose road having normal** crossfall.

4.1.2.2 Bridges having a carriageway width of 4.60 metres or more. The carriageway width shall be divided into a number of traffic lanes in accordance with Clause 4.1.2.1 of the Standard. The additional requirements in Table 11 shall be complied with where relevant:

TABLE 11 – NUMBER OF LANES FOR CARRIAGEWAY WIDTHS

Carriageway width	No. of lanes
18.5m up to and including 22.2m	6

4.1.2.3 Bridges having a carriageway width of less than 4.6 metres. The number of traffic lanes shall be determined in accordance with Clause 4.1.2.2 of the Standard. The number of traffic lanes may be a non-integral number in which case the loading is to be obtained pro rata.

4.1.2.4 Two superstructures carried on one substructure. Where two superstructures are provided for dual carriageways and are carried on a unified substructure, the number of traffic lanes carried by the substructure shall be taken as the sum of the traffic lanes on both superstructures.

4.1.3 Type HA uniformly distributed loading. Type HA udl shall be applied in accordance with A1(1) of the Standard except that for loaded lengths up to 6.5 metres the HA udl shall be taken as 31.5 kN per linear metre of traffic lane.

* The exclusions result from the absence of a hardstrip on the typical cross-section of the road approaches to the bridge.

** As opposed to reversed fall where surface water drains towards an offside hardstrip which is also provided on the road approaches to the bridge. In this case, the continuous hardstrip is included.

4. Type HA knife edge load. The knife edge load of A.1.(2) of the Standard shall not be applied in accordance with A.3.1 but as follows:

a. On longitudinal members and stringers, in a direction at right angles to the span of the member.

b. On cross-members, including transverse cantilever brackets, in a direction in line with the span of the member.

c. On plates, right slabs and skew slabs, whether or not they span longitudinally, transversely or as cantilevers, in a direction which produces the most severe effect. The knife edge load to be applied shall be 40 kN/m (full value) or $\frac{40}{3}$ kN/m ($\frac{1}{3}$ full value) irrespective of the width of lane.

4.1.5 Type HA 112kN wheel loads. The two 112kN wheel loads are applied in line transversely to the direction of traffic flow, spaced at 0.9 metre centres as in A.1.(3) of the Standard. The contact area of each wheel shall, however be distributed in accordance with 4.1.6 below.

The type HA wheel loading is to be used only where the member supports a small area of roadway, such that it may be called upon to carry the weight of one or two wheels, and where the proportion of distributed load and knife edge load which would be allocated to it is small (see also Clause 8).

4.16. Contact Area of HA and HB wheels. The contact area of each type HA and HB wheel load shall be assumed to be uniformly distributed over a circular or square area taking an effective pressure of 1.4 N/mm^2.

4.2 Application of standard highway live loading

4.2.1 Class of road

The following structures shall be designed to resist the more severe effects of either of the loadings given in Table 12 in combination with other loads.

TABLE 12 - HIGHWAY LOADING FOR VARIOUS CLASSES OF ROAD

Class of road carried by structure	Standard highway loading
Motorways and trunk roads (or principal road extensions of trunk routes, eg in County or Metropolitan Boroughs).	(1) Type HA loading or (2) Type HA loading combined with 45 units of type HB loading.
Principal roads	(1) Type HA loading or (2) Type HA loading combined with 37.5 units of type HB loading
Other public roads	(1) Type HA loading or (2) Type HA loading combined with 30 units of type HB loading.
Accommodation roads, bridleways and byways.	Type HA loading only (This supersedes the 2nd sentence of para 9.8 of the Motorway Design Memorandum).

For all public highway bridges the minimum number of units of type HB loading which shall normally be considered is given in the above Table, but this shall be such greater number up to 45 as directed by the appropriate authority.

4.2.2 Type of construction

Where type HB loading or the two 112 kN wheels of type HA loading or the accidental wheel loading is combined with other loading, basic permissible stresses may be increased, in accordance with the provisions of the relevant documents listed in Table 13.

TABLE 13 - DESIGN DOCUMENT TO BE USED FOR VARIOUS TYPES OF CONSTRUCTION

Type of construction	Design document
Steel girder	BS 153: Part 3B
Composite structural steel and concrete	CP 117: Part 2
Reinforced concrete	Technical Memorandum (Bridges) BE 1/73
Prestressed concrete	Technical Memorandum (Bridges) BE 2/73

4.2.3 Highway live loading on a single superstructure

4.2.3.1 Type HA loading On single carriageways and on dual carriageways on single superstructures, type HA loading shall be applied in accordance with Clause 4.1.3 of the Standard.

4.2.3.2 Type HB loading wholly within one traffic lane On single carriageways and on dual carriageways on single superstructures, type HB loading shall be applied in accordance with Clause 4.1.3 of the Standard.

4.2.3.3 Type HB loading straddling two traffic lanes Where an element can be more severely affected by the HB vehicle straddling any two traffic lanes, this shall be considered. In this case, the vehicle substitutes for type HB loading over a traffic lane width. The remainder of the straddled lanes on each side of this lane width shall be loaded with $\frac{1}{3}$ type HA loading.

On a single carriageway, all other traffic lanes are loaded with $\frac{1}{3}$ type HA loading.

On a dual carriageway on a single superstructure, full type HA loading shall be considered in two other traffic lanes on the carriageway not carrying HB loading. All other traffic lanes on the superstructure shall be loaded with $\frac{1}{3}$ type HA loading.

4.2.4 Omission of parts of the highway loading Attention is drawn to the requirement that the effects of live load shall not be taken into account where these are opposite in sign to the total effect or where the most severe effect on the element will be diminished by its presence. Apart from the simple application of this provision to positive and negative areas of influence lines for a continuous member, this requirement is also applicable to type HB loading; any one axle or bogey (ie two axles 1.8m apart) shall be ignored if by so doing a more severe effect on the element will result.

4.2.5 Multilevel superstructures Where multilevel superstructures are carried on common substructure members (as, eg columns of a multilevel interchange), superstructures shall be loaded so as to produce the most severe effect in the member under consideration. In determining the intensity of HA u.d.l, the portions of the multilevel loaded length which produce the most severe effect shall be taken as the total combined length of the loaded portions.

Only one type HB vehicle shall be considered.

4.2.6 Dispersal of type HA and type HB wheel loads. Dispersal of wheel loads shall be in accordance with Clauses A.3.6 and A.5.4 of the Standard. Dispersal means the spread of loading through surfacing, fill and the like, as opposed to distribution which means the sharing of load from directly loaded members to other members as a consequence of the stiffness of the interconnecting construction (deck slabs).

4.2.7 The type HB vehicle This shall normally be considered in the most severe position laterally within the width of the traffic lane it occupies; however in the case of the nearside traffic lane the vehicle shall be positioned so that there is at least 200mm between the centreline of the wheels and the kerb.

4.2.8 Central reserves and hard strips excluded from the carriageway by 4.1.2.1 These shall not be loaded with live load in considering the overall design of the structure. However, these areas shall be capable of carrying the more severe of the following loads:

The accidental wheel loading (see 4.7)

OR

Full type HA loading.

This provision shall be satisfied if the area being considered is supported by structural members which are of the same form and strength as those designed to carry the loading applied to the adjacent traffic lane.

4.2.9 Load distribution analysis Advantage should be taken of the transverse distribution of both type HA and type HB loading where overall economy can be achieved. Approved methods for the determination of bending moments and shear forces include the Morice and Little (up to 20° skew), Hendry and Jaeger methods and two - and three - dimensional Frame and Finite Element analyses. The reduction factors in Clause A5 of the Standard are acceptable for skews up to 20°. Approved computer programs are listed in Technical Memoranda BE 2/76, BE 1/75 and BE 8/73.

The local effects of wheel loads on slabs may be calculated by the method of Westergaard (1) or Pucher (2).

4.2.10 Combined effects Where elements of a structure can sustain the effects of live load in two ways, ie as elements in themselves and also as parts of the main structure reference shall be made to Clause A.3.10 of the Standard.

4.3 Longitudinal load

The horizontal longitudinal load resulting from the traction or braking of vehicles shall be in accordance with Clause 10 of the Standard. Longitudinal loads shall not be considered in combination with centrifugal load.

(1) 'Computation of Stresses in bridge slabs due to wheel loads', HM Westergaard, Public Roads, Vol 11 No.1 March 1930.

(2) 'Influence Surfaces of Elastic Plates', A Pucher, published by Springer - Verlag, Vienna and New York, 1964, Translation by H Juhl.

4.4 Centrifugal load

Radial lateral loads P shall be applied at 50 metre centres in each of two traffic lanes where

$$P = \frac{30\ 000}{r + 150}\ \text{kN}$$

r = radius of curvature of the traffic lane in metres.

Each load P may be distributed along 5 metres loaded length or divided into two parts of $\frac{1}{3}$P and $\frac{2}{3}$P at 5 metre centres longitudinally.

The loads shall be applied at the surface of the road and parallel to it, and shall be considered in combination with a vertical load of 300 kN distributed along 5 metres loaded length in each of these traffic lanes at each point of application of the load P. Where P is divided into two parts the vertical loading shall be divided in the same proportions and applied at the same points.

The centrifugal load shall not be considered in combination with longitudinal loads, or vertical live loads other than the 300 kN loads directed here.

4.5 Loads due to vehicular collision with parapets.

4.5.1 Parapet post anchorages. Requirements for highway bridge parapets are given in Technical Memorandum BE5. The factor of 1.5 on parapet post anchorages is intended to safeguard the permanent structure so that it would not require expensive repair as a result of parapet damage. If the cost of providing this strength at the anchorages throughout a long structure is greater than the cost of repairing local damage due to a severe impact, this factor may be reduced to 1.0 except at end posts or positions of discontinuity in parapet rails.

4.5.2 Structures supporting vehicle parapets. In the design of a structure supporting a vehicle parapet, the effect of a collision may be taken as a single load 1.2 times the load P, specified in Technical Memorandum BE5 for the parapet. This factor is an allowance for the parapets having a resistance higher than is specified. The load should be shared equally between the main longitudinal members of the parapet and applied at the position in plane where it would produce the most severe effect.

4.5.3 Associated vertical loading on supporting structures. The vertical loading adjacent to the collision point shall be assumed to be the accidental wheel loading placed in the position which will have the most severe effect. (See 4.7).

4.5.4 Effect of combining horizontal and vertical loading. Where the effects of 4.5.1 and 4.5.2 are diminished by the effects of 4.5.3 the latter shall be ignored.

4.6 Loading for fatigue investigations

Refer to Technical Memorandum (Bridges) No. BE 16.

4.7 Accidental wheel loading.

4.7.1 Areas subject to this loading.

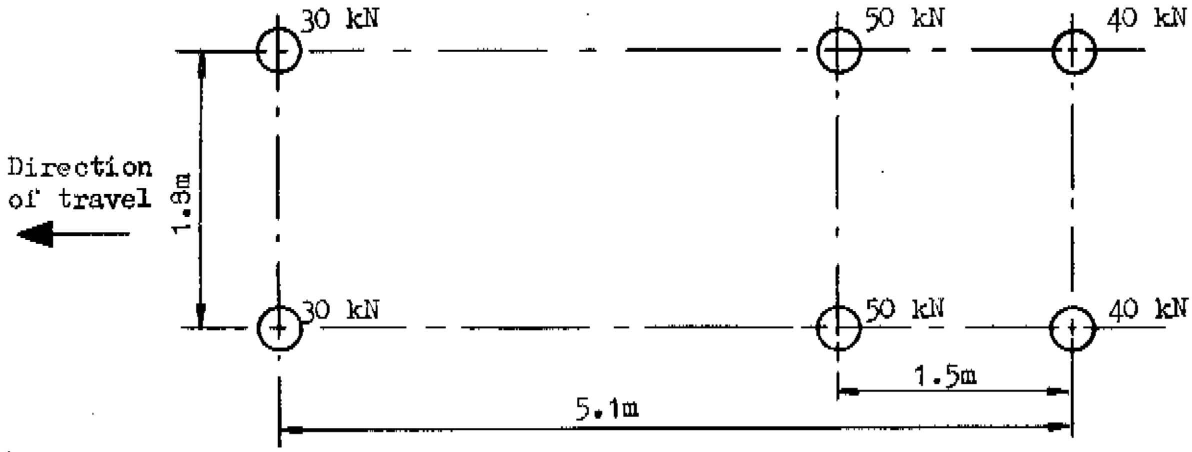
a. Verges behind raised kerbs.

b. Central reserves and hardstrips excluded from the carriageway by 4.1.2.1 (For other loading see 4.2.8).

c. Footways and cycle tracks in certain circumstances (see 4.5 and 4.8).

4.7.2 Wheel loading and disposition. The wheel load arrangement, shown in Figure 11, shall be placed in the position which will have the most severe effect. This loading applies to local effects and shall not be taken into account in determining global effects on the deck.

FIGURE 11 - ACCIDENTAL WHEEL LOADING



4.7.3 Associated type HA loading. Where the effects of this accidental wheel loading on main members supporting the carriageway are being considered the carriageway loading shall be assumed to consist of full type HA loading on one traffic lane and one third type HA loading on the remaining traffic lanes.

4.7.4 Contact area of wheel loads. Each wheel load shall be deemed to include impact and to be distributed over a circular contact area 250mm in diameter.

4.7.5 Increases in permissible stresses. Basic permissible stresses may be increased when accidental loading is combined with parapet forces and other loading in accordance with 4.2.2.

4.8 Footway and cycle track live loading.

4.8.1 Bridges supporting footway and/or cycle tracks only. The live load shall be taken as 5.0 kN/m^2.

4.8.2 Bridges supporting highway, footway and/or cycle tracks. The live loading shall be taken as the more severe of either (a) or (b) below.

a. For loaded lengths up to 23 metres, a uniformly distributed live load of 4.0 kN/m^2.
For loaded lengths in excess of 23 metres, a uniformly distributed live load of K x 4.0 kN/m^2 where

$$K = \frac{\text{HA UDL for the appropriate loaded length}}{31.5 \text{ kN/m}}$$

OR

b. Where the footway or cycle track is not protected from traffic live loads by an effective barrier, the accidental wheel loading specified in 4.7 in the most severe position.

4.9 Sign/signal gantry live loading.

Gantry live loading includes for maintenance and ice/snow loads.

4.9.1 Global effects. A live load of 1.5 kN/m run of gantry.

4.9.2 Local effects. A live load of 2.0 kN/m^2 on walkways and their supporting members.

5. DYNAMIC LOADING

5.1 Highway bridges

The dynamic effects of standard highway loading on commonly occurring types of highway bridges are deemed to be covered by the impact allowance included in traffic live loading. Special consideration, however may need to be given to unusually flexible bridges (eg suspension, cable-stayed etc) where dynamic behaviour may be critical.

5.2 Footbridges

The dynamic behaviour of footbridges should be such that no discomfort or alarm is caused to the majority of pedestrian users.

For footbridges where f, the fundamental natural frequency of vibration for the unloaded bridge exceeds 5 Hz, this requirement is deemed to be satisfied.

For footbridges having a value of f equal to or less than 5 Hz, the maximum vertical acceleration of any part of the deck shall be limited to $\pm\frac{1}{2}\sqrt{f}$ m/sec^2. The maximum acceleration shall be derived in accordance with Appendix C.

5.2.1 Damage from forced vibration. Consideration should be given to the possibility of permanent damage to a footbridge by a group of pedestrians deliberately causing resonant oscillation of the bridge. 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.

5.3 Sign/signal gantries.

Dynamic effects arising from live loads due to maintenance may be disregarded. Consideration may need to be given however to dynamic effects arising from other sources (eg wind induced oscillations).

6. LOADS DUE TO VEHICLE COLLISION WITH HIGHWAY STRUCTURES

6.1 Displacement of highway structure supports.

The consequences of impact on supports adjacent to the carriageway edge shall be considered in the design of all highway structures to ensure that the superstructure will not collapse under dead load. Connections between structural members shall be designed to resist or accommodate relative movements without collapse. Supports complying with 6.1.1 and 6.1.2 or 6.1.1 and 6.1.3 as appropriate are deemed to satisfy this requirement.

6.1.1 Protection of highway structure supports. Safety fences shall be provided to protect all supports subject to possible vehicular collision. Strength, minimum length requirements etc are given in Tech Memo H9/73.

6.1.2 Highway bridge over road. Highway bridge supports shall be capable of resisting all of the impact loads in Table 14, applied concurrently.

TABLE 14 - IMPACT LOADS ON HIGHWAY BRIDGE SUPPORTS

	Load in kN normal to c'way	Load in kN parallel to c'way	Point of Application
Impact from guard-rail	225	75	Any one bracket attachment point or, for free standing fences, any one point 750mm above the carriageway
Residual impact above guard-rail	150	150	At the most severe point between 1 and 3 metres above the carriageway.

6.1.3 Footbridge or sign/signal gantry over road. Footbridge and gantry supports shall be designed to resist an impact of 50 kN acting in the most severe direction and at the most severe point up to 3 metres above the carriageway.

6.2 Impact loading on highway and footbridge superstructures from overheight vehicles

For all bridge structures with a minimum headroom clearance less than 5.5 metres the superstructure and its connections shall be capable of resisting an impact of 50 kN acting in the most severe direction between the horizontal and the vertically upward direction, applied to the most severe point on the soffit.

6.3 Highway or footbridge over British Rail tracks

Impact loading shall be subject to approval by British Rail and the Railway Inspectorate.

6.4 Highway or footbridge over nagivable waterways

Impact loading shall be agreed with the relevant authority.

6.5 Loading combination

6.5.1 Impact loading on highway structure supports. Impact on supports shall be considered in combination with dead load plus superimposed dead load plus 0.5 x wind load.

6.5.2 Impact loading on bridge superstructure. Impact on the superstructure shall be considered in combination with dead load plus superimposed dead load only.

6.5 Method of analysis

The capacity of members including the connections and the foundations which resist the impact loads shall be based on an analysis at ultimate conditions. The safety factor against collapse shall be not less than 1.15.

7. EARTH PRESSURES ON RETAINING STRUCTURES

7.1 Effects of filling material

Where filling material is retained by part of the structure, the loads calculated by soil mechanics principles shall be taken into account. Where the superimposed dead load comprises filling, as eg on spandrel filled arches or fill behind earth retaining structures, consideration should be given to the possibility that the filling material may become saturated. Consideration shall also be given to the effects of the removal in whole or in part of the filling material which has an effect algebraically opposite in sign to the total effect being calculated.

7.2 Live load surcharge.

In the absence of more exact calculations, the normal allowance of 0.6 metres of surcharge may be assumed to cover full type HA loading but this should be increased to 1.2 metres for 45 units of type HB loading. These values are based on a density of 1900 kg/m^3 and should be increased in inverse proportion for lesser densities, eg for a backfill of density 1500 kg/m^3: 0.75 metres of surcharge for full type HA loading.

8. LOADING ON BURIED STRUCTURES

8.1 General.

The following clauses apply only to buried structures of span or internal diameter greater than 0.9 metres which have a minimum depth of fill greater than 0.6 metres. For buried structures having a span or internal diameter less than or equal to 0.9 metres, the design data in Building Research Station document "Simplified tables of external loads on buried pipelines" may be used.

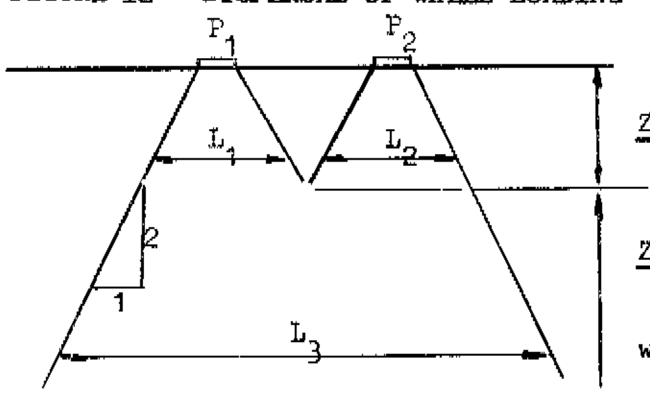
These clauses do not apply to spandrel filled arches.

8.2 Highway live loading.

The structure shall be designed to resist the more severe effects of the type HA and type HB wheel loading, in combination with other loads as appropriate.

8.2.1 Dispersal of type HA and type HB wheel loading. Dispersal of the wheel loads may be assumed to occur from the contact area on the carriageway (see 4.1.6) to the level of the top or crown of the buried structure and shall be taken at a slope of 2 vertically to 1 horizontally as shown in Fig 12. Wheel loads not directly over the structure must nevertheless be considered if the dispersed loading overlaps the edge of the structure.

FIGURE 12 - DISPERSAL OF WHEEL LOADING



Zone 1: P_1 or P_2 uniformly distributed over length L_1 or L_2 respectively.

Zone 2: $P_1 + P_2$ uniformly distributed over length L_3.

where P_1 and P_2 represent wheel loads.

8.2.2 Application of Boussinesq theory. As an alternative to dispersal of the wheel loading, where the depth of fill exceeds 1 metre, the pressure intensity may be derived from Boussinesq theory.

8.2.3 Longitudinal loads. The structure shall be designed to resist longitudinal loads resulting from traction or braking of vehicles unless provision has been made for the transfer of such loads by the road slab.

Traction or braking loads may be disregarded on buried structures covered by a fill depth exceeding the span or diameter of the structure.

8.3 Temperature effects

8.3.1 During construction. Temperature effects shall be considered for the erection condition of all buried structures in accordance with 3.4.

8.3.2 In service. Buried structures of a length up to 5 times the span or diameter, are considered to be open to the atmosphere and temperature effects are derived from 3.4.

For buried structures of a length exceeding 5 times their span or diameter, the requirements of 3.4 may be modified as in Table 15.

TABLE 15 TEMPERATURE EFFECTS IN BURIED STRUCTURES

Span or diameter metres	Fill depth metres	Temperature C Range	Difference
≤ 3	≥ 0.6	Disregard temp. effects	
≥ 3	$\geq 0.6 - 0.75$	10 ± 10	$\frac{1}{2}$ Fig 10 values
	$\geq 0.75 - 1.0$	10 ± 6	$\frac{1}{3}$ Fig 10 values
	$\geq 1.0 - 2.0$	10 ± 3	Disregard
	≥ 2.0	Disregard temp. effects	

Structures subject to thermal effects other than from atmospheric sources (eg roof slab over a metro) or closed structures at steady internal temperatures but variable external temperatures or vice-versa, require special consideration and advice shall be sought in such cases.

8.4 Reinforced concrete cylindrical pipes

Pipes shall either be designed to comply with the Department's standards or shall be load tested in accordance with British Standard 556:Part 2:1972 and manufactured under BSI kitemark licence.

Elliott

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14th February 1977

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APPENDIX A

The following clauses of BS 153: Part 3A: 1972 as amended by the appropriate clauses of this memorandum are particularly relevant to the loading to be considered in the design of highway structures.

BS 153: SPECIFICATION FOR STEEL GIRDER BRIDGES: PART 3A LOADS: 1972

1. Scope

3. Dead Load;

See also clause 3.1 of this Memorandum.

4.1.1 Standard highway loading;

Only the clauses of Appendix A of the Standard listed below;
See also clause 4.2.1 of this Memorandum.

4.1.2 Width and number of traffic lanes;

Sub-clauses amended as below:

4.1.2.1 Bridges having a carriageway width of 4.60m or more;

See also clause 4.1.2.2 of this Memorandum.

4.1.2.2 Bridges having a carriageway width of less than 4.60m;

See also clause 4.1.2.3 of this Memorandum.

4.1.2.3 Number of lanes;

1st paragraph only,
See also clauses 4.1.2.1 and 4.2.8 of this Memorandum.

4.1.3 Application of standard loading on a single superstructure;

See also clause 4.2.3 of this Memorandum.

10. Longitudinal force on highway bridges;

See also clause 4.3 of this Memorandum.

15. Forces on parapets.

16. Combination of forces;

This clause shall be taken to refer to steel girder bridges only, (other than steel box girder bridges).

For other types of structure see clause 2.6 of this Memorandum.

18. Anchorage;

See also clause 2.2 of this Memorandum.

A.1 Type HA loading;

See comments on sub-clause below.

A.1(1) Uniformly distributed loading;

See also clause 4.1.3 of this Memorandum.

A.1(2) Knife edge loading;

Delete reference to clause A.3.1 of the Standard and substitute clause 4.1.4 of this Memorandum.

A.1(3) Two wheel loads;

For details of contact area refer to clause 4.1.6 of this Memorandum.

Delete "and on cantilever projections not exceeding 1.80 m."

Delete para b and last para.

A.3 Application of type HA loading;

Sub-clauses amended as below.

A.3.3 Total end live load shear.

A.3.4 No allowance for impact or dispersal of UD loads or KE load;

But see clause 8.2.1 of this Memorandum.

A.3.5 No allowance for impact under wheel loads.

A.3.6 Dispersal under wheel loads;

See also clauses 4.2.6 and 8.2.1 of this Memorandum

A.3.8 Reinforced concrete slab design;

See also clauses 4.1.3 and 4.2.9 of this Memorandum.

A.3.10 Combined effects of live load;

Delete second para; refer to clause 4.2.2 of this Memorandum for increases in permissible stresses.

Table 1 and Figure 1:

Type HA equivalent uniformly distributed load;

For loaded lengths up to 6.5m refer to clause 4.1.1.1 of this Memorandum.

Slab and beam loading;

Where the traffic lane width is over 3m, the lane loading and the load/sq metre to be applied to slabs shall be the same as the loading to be applied to beams. The knife edge load per metre, irrespective of the width of traffic lane, to be applied to plates, right slabs and skew slabs shall be in accordance with the requirements of 4.1.4(c) of this Memorandum.

A.4 Type HB loading;

Delete reference to clause A.5.3 of the Standard.
For contact area of wheels see clause 4.1.6 of this Memorandum.

A.5 Application of type HB loading;

Sub-clauses amended as below.

A.5.1 No allowance for impact.

A.5.2 Increase in permissible stresses;

Refer to clause 4.2.2 of this Memorandum.

A.5.3 Contact areas of wheels;

Refer to clause 4.1.6 of this Memorandum.

A.5.4 Dispersion or distribution of wheel loads;

See also clause 4.2.6 of this Memorandum.

A.5.5 Reduction factors for type HB wheel loading.

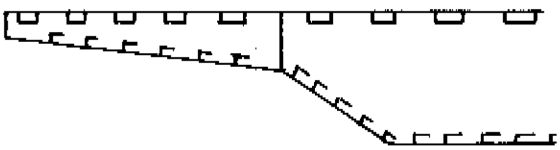
NOTE: Clause A6 has not been included because it is no longer valid for the constant intensity of the uniformly distributed part of type HA loading on loaded lengths up to 6.5 m and because the 25 per cent overstress is no longer generally applicable. See clause 4.2.2 of this Memorandum.

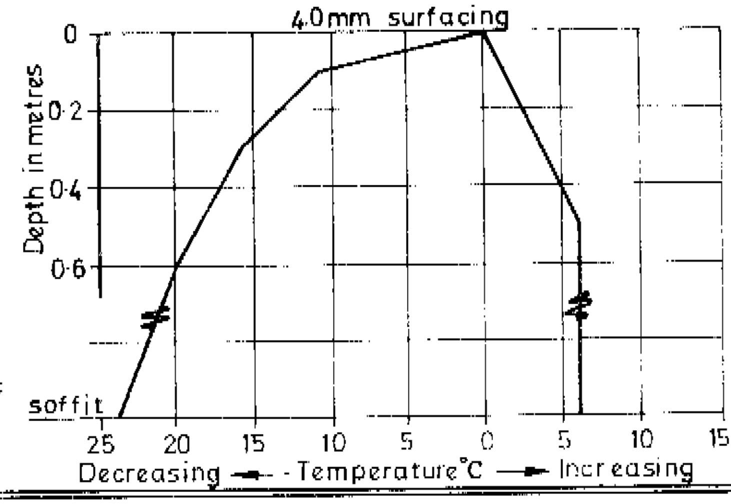
APPENDIX B

FIG. B1

MEASURED TEMPERATURE DIFFERENCES

GROUP	TYPE OF CONSTRUCTION
1.	Steel deck on steel box girders.

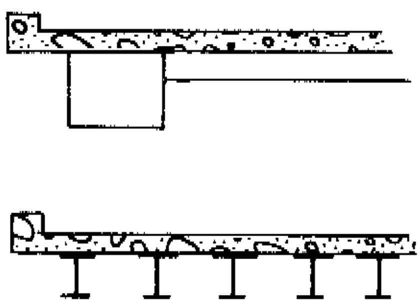


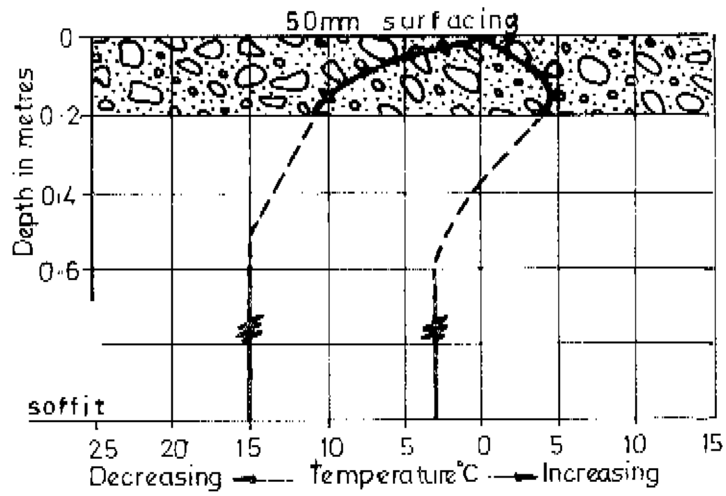


2.	Steel deck on steel truss or plate girders.

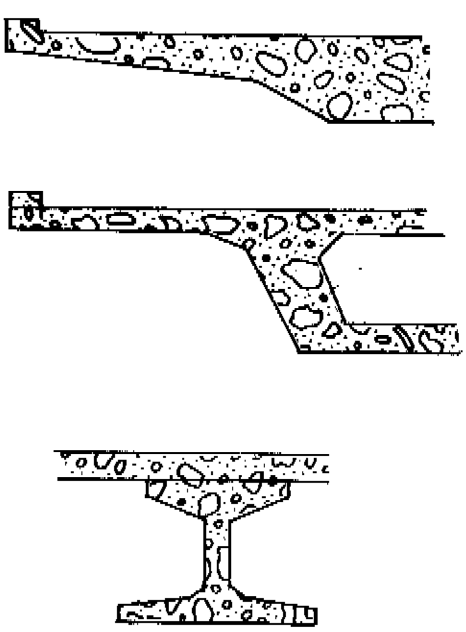
Use differences as for group 1.

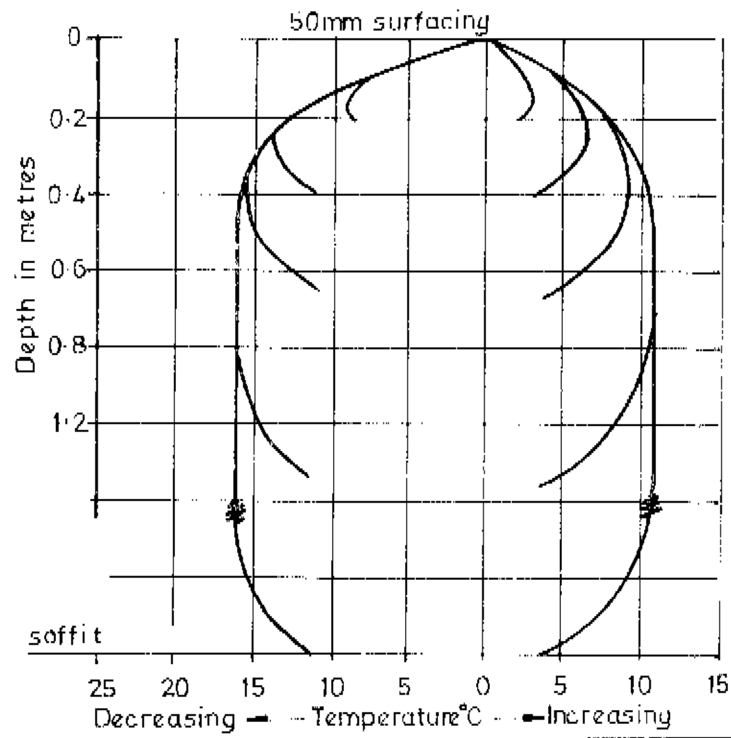
3.	Concrete deck on steel box, truss or plate girders.





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





APPENDIX C

DERIVATION OF MAXIMUM VERTICAL ACCELERATION FOR FOOTBRIDGES

C.1 <u>Approximate method</u>. This method is valid only for single span or two or three span continuous symetric superstructures. In addition the deck must be of constant cross-section and supported on bearings which can be idealised as knife-edged supports.

The maximum vertical acceleration a, in m/sec^2 shall be taken as :

$$a = 4\pi^2 f^2 y_s K \psi \quad \text{where}$$

f	= fundamental natural frequency in Hz	see C. 1.3
y_s	= static deflection in metres	see C. 1.4
K	= configuration factor	see C. 1.5
ψ	= dynamic response factor	see C. 1.6

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

C.1.1 <u>Modulus of elasticity.</u> In calculating the values of f and y_s, dynamic modulii of elasticity shall be used for concrete and for steel. For steel, in the absence of a dynamic value, the static modulus may be considered sufficiently accurate. Dynamic modulii are given in Clause 4.1 of BE 1/73.

C.1.2 <u>Second moment of area.</u> In calculating the values of f and y_s, the second moment of area for sections of discrete concrete members may be based on the gross concrete section ignoring the presence of reinforcement.

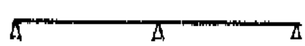
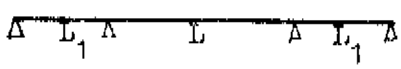
C.1.3 <u>Fundamental natural frequency f.</u> The natural frequency is evaluated for the bridge including superimposed dead load but excluding pedestrian live loading.

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

C.1.4 <u>Static deflection y_s.</u> The static deflection is taken at the mid-point of the main span for a vertical concentrated load of 0.7 kN applied at this point. For three span bridges, the centre span is the main span.

C.1.5 <u>Configuration factor K.</u> Values of K shall be taken from Table C.1 below:

TABLE C.1 – CONFIGURATION FACTOR K

Bridge configuration	Ratio L_1/L	K
	–	1.0
	–	0.7
	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.1.6 Dynamic response factor ψ. Values of ψ are given in Figure C.1

In the absence of more precise information, values of δ, the logarithmic decrement of the decay of vibration due to structural damping given in Table C.2 should be used.

TABLE C.2 - LOGARITHMIC DECREMENT OF DECAY OF VIBRATION - δ

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

C.2 Rigorous method. For footbridges which do not conform to those of C.1, the maximum 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 bridge at a constant speed v as follows:

$F = 180 \sin 2\pi f t$ (Newtons) where t = time in seconds.

$v = 0.9 f$ (m/s)

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

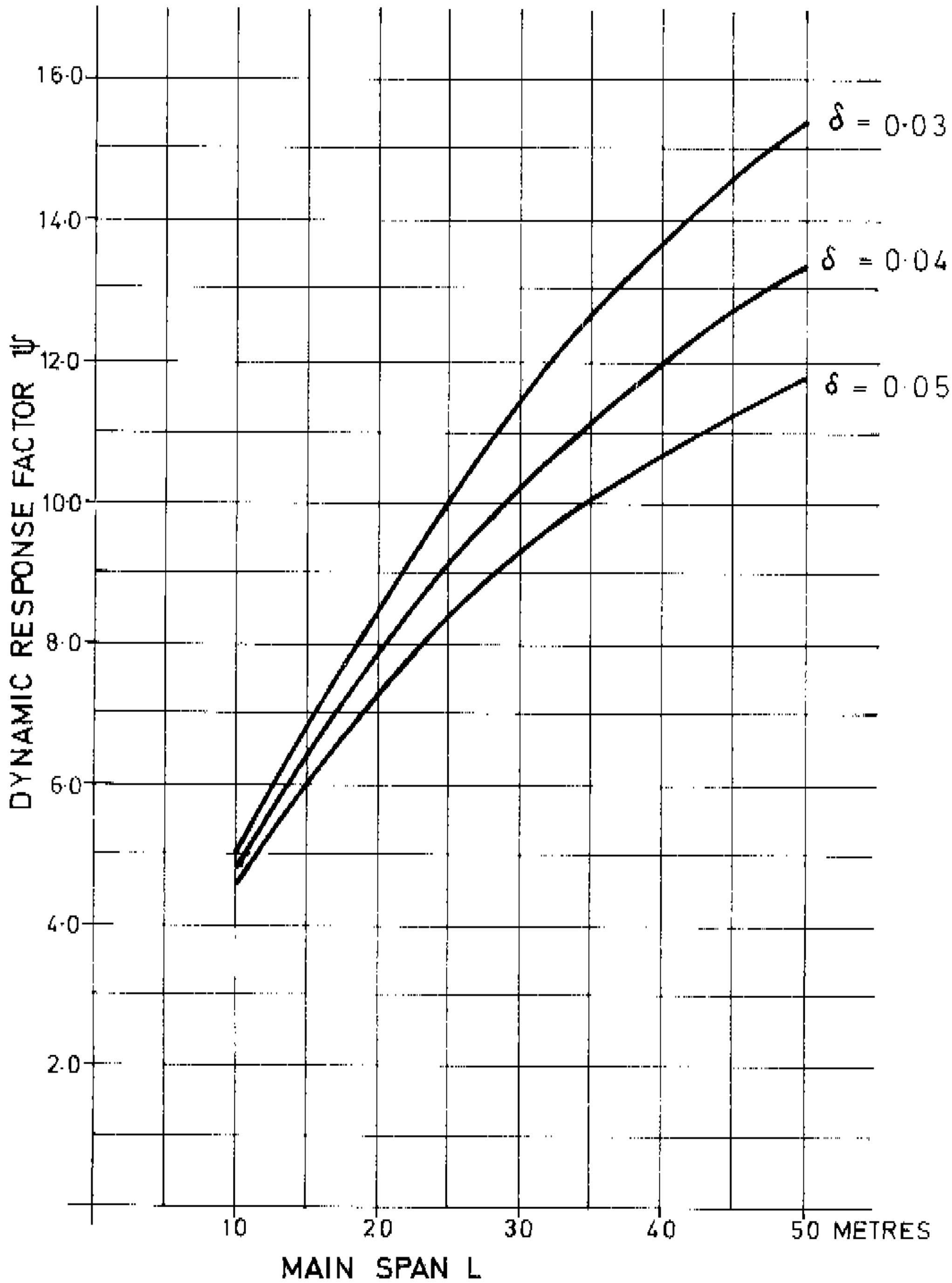


FIG. C1. DYNAMIC RESPONSE FACTOR ψ

NOTE: 1. Main span L is shown in Table C1 of C.1.5

2. Values of δ for different types of construction are given in C.1.6

DEPARTMENT OF TRANSPORT

ROADS AND LOCAL TRANSPORT GROUP

TECHNICAL MEMORANDUM (BRIDGES) BE 1/77: STANDARD HIGHWAY LOADINGS

AMENDMENT LIST NO 1

The following amendments shall be made to the above document to update the Department's requirements pending the general adoption of BS 5400: Part 2: July 1978.

Those pages requiring substantial amendment have been reproduced in full so that they may be included as amended pages.

PAGE	CLAUSE	AMENDMENT
i	2.5.5	Amend clause title to: "Frictional restraint at bearings"
iii		Delete page iii, insert amended page iii attached
iv	4.2.1	Amend clause title to: "Number of units of type HB loading"
	4.2.2	Delete clause title (not used)
vi	6.1.1	Amend clause title to: "Protection at highway structure supports"
	8.4	Amend clause title to: "Precast concrete pipes"
	Appendix B	Amend Appendix title to: "VALUES OF 'T' FOR A RANGE OF SURFACING DEPTHS"
vii	Table 7	Delete table (not used)
	Table 10	Amend title to: "Adjustment to effective bridge temperature for depth of surfacing"
	Table 12	Amend title to: "Number of units of type HB loading for various classes of road"
	Table 13	Delete title (not used)
	Figure 1	Amend title to: "Isotachs of mean hourly wind speed (m/s)"

PAGE	CLAUSE	AMENDMENT
	Figure 8	Amend title to: "Isotherms of minimum shade air temperature (^{o}C)"
	Figure 9	Amend title to: "Isotherms of maximum shade air temperature (^{o}C)".
1	1.vi	Insert additional item as follows: "vi. Traffic signs - Traffic signs manual"
4	2.5.4.3	Delete clause, insert amended clause as follows: "2.5.4.3 <u>Ultimate load combination</u>. For prestressed concrete highway bridges, dead load, superimposed dead load and live load due to traffic shall be combined in accordance with Clause 7 of Technical Memorandum BE 2/73: Prestressed concrete for highway structures. For prestressed concrete footbridges and gantries, dead load, superimposed dead load, live load and wind load shall be combined in accordance with Clause 5 of Technical Memorandum BE 1/78: Design criteria for footbridges and sign/signal gantries."
	2.5.5	Delete clause, insert amended clause as follows: "2.5.5 <u>Frictional restraint at bearings</u>. The restraint at roller and sliding bearings shall be derived from the dead load and the superimposed dead load."
5	3.3.1, para 2, lines 7 and 8	Delete lines, insert the following: "Figure 1. These wind speeds are appropriate to a height above ground level of 10 metres in open level country and a 120 year return period."
	3.3.2, line 6	Delete line, insert the following: "K_1 = a coefficient related to return period See 3.3.2.2"
6	Fig 1, title	Amend title to: "Isotachs of mean hourly wind speed (m/s)"
7		Delete page, insert amended page 7 and additional page 7a attached
8	3.3.2.5, para 1, line 5	Delete 'windbreaks', insert "obstructions"
	3.3.2.6	Delete whole clause, insert amended clause as follows: "3.3.2.6 <u>Hourly speed factor K_2 for minimum wind gust speed</u>. Where wind on any part of a structure or element gives relief to the member under consideration, the effective co-existent value of

PAGE	CLAUSE	AMENDMENT

minimum wind gust speed on the parts affording relief (and on any adverse areas not subject to maximum wind) shall be taken as follows:

for unloaded structures: $v_c = v\ K_1\ K_2$

for live loaded structures: v_c = the lesser of $35 \times \frac{K_2}{S_2}$

and $v\ K_1\ K_2$

Where v, K_1 and S_2 are defined in 3.3.2.1, 3.3.2.2 and 3.3.2.4 respectively, and

K_2 = hourly speed factor as given in Table 1

S_2 and K_2 may be modified, where appropriate in accordance with 3.3.2.5."

3.3.2.7, lines 2 and 4
Delete '33', insert "35"

9 3.3.3.1.2i
Delete sub-clause, insert the following:

"i Unloaded bridge with open parapets. The transverse wind load P_t is derived separately for the following:

a. deck

b. windward parapet and where appropriate, the safety fence

c. leeward parapet and where appropriate, the safety fence

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

10 3.3.3.2.1, title
Amend title to "Single beam or box girder"

3.3.3.2.2
Delete clause, insert amended clause as follows:

"3.3.3.2.2 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 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 3.3.3.2.1 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 C_D derived as specified in 3.3.3.2.1 for the single beam or box, where n is the number of beams or box girders."

PAGE	CLAUSE	AMENDMENT
	3.3.3.2.3, title	Amend title to: <u>"Single plate girder"</u>
	3.3.3.2.4, line 1	Delete line, insert the following: "3.3.3.2.4 <u>Two or more plate girders.</u> 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"
16	3.3.3.4.2	Delete clause, insert amended clause as follows: "3.3.3.4.2 <u>Live loaded bridge.</u> The drag coefficient for each truss and for the deck shall be as derived in 3.3.3.4.1. C_D on unshielded parts of the live load shall be taken as 1.45."
18	Table 6, column 2, row 3	Delete '$\frac{1}{2}$', insert "$\frac{1}{3}$"
	Table 6, note 2, line 1	Expression should read "$t/b \geq 1/3$"
	Table 6, note 3, line 2	Delete 'lesser', insert "greater"
19	3.3.3.9.2, last line	Delete last line, insert the following: "d_L = 2.5 m above the carriageway for highway live loading and 1.25 m above the footway for footway and cycle track live loading."
20		Delete page, insert amended page 20 and additional page 20a attached
21	3.3.5, line 5	After 'Fig 7', insert "for structures where the angle of superelevation is less than 1°."
	Figure 7	After title to abscissa, insert "(See fig. 4)."
22		Delete page, insert amended page 22 attached.
23	Figure 8, title	Amend title to: "FIG. 8 ISOTHERMS OF MINIMUM SHADE AIR TEMPERATURE ($^{\circ}$C)"

PAGE	CLAUSE	AMENDMENT
24	Figure 9	title Amend title to: "FIG 9. ISOTHERMS OF MAXIMUM SHADE AIR TEMPERATURE (^{o}C)"
25–28 inclusive		Delete page, insert amended pages 25-28 attached.
29	3.5.2, line 1 3.5.3, line 2 3.5.4, lines 1 and 3	Delete 'accomodate', insert "accommodate"
31	4.1.4	Delete clause, insert amended clause as follows:

"4.1.4 Type HA knife edge load (kel). The HA kel of A1(2) of the Standard shall be 120 kN per traffic lane for full HA or 40 kN per traffic lane for ⅓ HA as appropriate. The kel shall always be taken as acting uniformly on a straight line extending over the width of the traffic lane except as in iii below, where the intensity of 120/W or 40/W as appropriate shall be used, where W is the traffic lane width. For the design of the structural elements given below, the orientation of the kel shall be as follows:

i. for longitudinal members and stringers - in a direction parallel to the supports;

ii. for cross members, including transverse cantilever brackets - in a direction in line with the span of the member;

iii. for plates, right slabs and skew slabs, spanning or cantilevering longitudinally or transversely - in a direction which produces the most severe effect; and

iv. for piers, abutments and other members supporting the superstructure - in a direction parallel to the supports."

4.2.1 Delete clause, insert amended clause as follows:

"4.2.1 Number of units of type HB loading. The number of units of type HB loading according to the class of road shall be as given in Table 12.

PAGE	CLAUSE	AMENDMENT

TABLE 12 - NUMBER OF UNITS OF TYPE HB LOADING FOR VARIOUS CLASSES OF ROAD

Class of road carried by structure	Number of units of type HB loading
Motorways and Trunk Roads (or principal road extensions of trunk routes - eg in County or Metropolitan Boroughs)	45
Principal roads	37.5
Other public roads	30
Accommodation roads, bridleways and byways	0

For all public highway bridges, the minimum number of units of type HB loading which shall normally be considered is given in the above Table, but this shall be such greater number up to 45 as directed by the appropriate authority."

32 4.2.2

Delete clause and Table 13 (not used)

4.2.3, line 1

Insert additional sentence after title as follows:

"This clause applies to a superstructure carrying not more than seven working lanes."

4.2.4, line 4

After 'element', insert "or structure"

33 4.2.8, line 8

Delete 'shall', insert "is deemed to"

4.2.9, lines 7 and 8

Delete last sentence, insert "Approved computer programs are given in the appropriate documents listed in BD 2/79."

37 6.1.1 Delete clause, insert amended clause as follows:

"6.1.1 Protection at highway structure supports. Regardless of the requirements given in 6.1.2 and 6.1.3 below, safety fences shall be provided in accordance with Technical Memorandum H9/73."

6.2, title

After 'highway', insert "bridge"

PAGE	CLAUSE	AMENDMENT
	6.4, title	Delete 'nagivable', insert "navigable"
	6.5 Method of analysis	Amend this clause number to "6.6"
39	8.2, line 3	After 'HA', insert "wheel loading"
40	8.4	Delete clause, insert amended clause as follows: "8.4 <u>Precast concrete pipes</u> The loading data given in Building Research Station publication "Simplified Tables of External Loads on Buried Pipelines" (HMSO 1970) may be used, subject to the provisions of BE 1/73."
41	Item 16, line 4	Delete '2.6', insert "2.5"
42	Table 1 and Fig 1, line 2	Delete '4.1.1.1', insert "4.1.3"
44	Appendix B	Delete Appendix, insert amended Appendix B attached (pages 44 and 44a)
45	Appendix C Clause C.1.1, line 4	Delete last sentence, insert: "Dynamic moduli are given in BE 1/73."
	Clause C.1.2, line 3	Insert additional sentence as follows: "For the purpose of this Appendix, the effects of shear lag need not be taken into account in steel and concrete bridges."
	Clause C.1.3, para 1, line 2	Delete line, insert the following: "bridge having due regard to the number of spans, continuity and nature of supports. Superimposed dead load shall be included, but not pedestrian live loading."
46	Appendix C Clause C.2, para 1, line 2	After 'calculated', insert "from the equations of motion "
	Clause C.2, para 1, line 4	After 'speed v', insert "where F and v may be taken "

ENQUIRIES

Technical enquiries arising from the application of these amendments to a particular design should be addressed to the TAA for that scheme.

General technical enquiries or comments should be addressed to:

Assistant Chief Engineer
Bridge Engineering Standards Division
Department of Transport
St Christopher House
Southwark Street
LONDON
SE1 OTE

Distribution enquiries should be addressed to:

Highways Procedures and Legislation 2 Division
Room P3/030
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31 May 1979

Page

(Amendment No 1 May 1979)

3.3.4 Derivation of longitudinal wind load. The method of deriving longitudinal wind load differs from 3.3.3 in that P_L for live loaded bridges shall be obtained separately for the bridge and for the live load. The total longitudinal wind load on the superstructure is then obtained by summing the separate values of P_L.

3.3.4.1 All bridges other than truss girder bridges

$P_L = 0.25\ q\ C_D\ A_1$ where

q = dynamic pressure head = $0.613\ v_c^2$ in N/m^2 where v_c in m/s is derived for the unloaded or live loaded bridge as appropriate.

C_D = drag coefficient (excluding reduction for inclined webs) as defined in 3.3.3.3 and 3.3.3.5 for unloaded bridges, but not less than 1.3.

A_1 = as defined in 3.3.3.9 for unloaded bridges.

3.3.4.2 Truss girder bridges

$P_L = 0.5\ q\ C_D\ A_1$ where

q = dynamic pressure head = $0.613\ v_c^2$ in N/m^2 where v_c in m/s is derived for the unloaded or live loaded bridge as appropriate.

C_D = drag coefficient as defined in 3.3.3.4.1.

A_1 = as defined in 3.3.3.9 for unloaded bridges

3.3.4.3 Live load on all bridges

$P_L = 0.5\ q\ C_D\ A_1$ where

q = dynamic pressure head = $0.613\ v_c^2$ in N/m^2 where v_c is in m/s

C_D = drag coefficient, taken as 1.45

A_1 = area of live load based on height d_L as given in figure 6

3.3.4.4 Parapets and safety fences.

P_L = $0.8\ P_t$, for parapets with vertical infill members

P_L = $0.4\ P_t$, for parapets and fences with 2 or 3 horizontal rails only

P_L = $0.6\ P_t$, for parapets with mesh panels

where P_t = the appropriate transverse wind load on the parapet or fence.

3.3.4.5 Cantilever brackets extending outside main girders or trusses

P_L = 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 brackets. The drag coefficient C_D shall be taken from Table 5.

(Amendment No 1 May 1979)

3.3.4.6 Piers and structure supports

$P_L = q\ C_D\ A_2$ where

q = dynamic pressure head = $0.613\ v_c^2$ in N/m^2 where v_c is in m/s

C_D = drag coefficient, taken from Table 6, with values of b and t interchanged.

A_2 = net projected area in the longitudinal wind direction (in m^2).

3.3.4.7 Sign/signal gantries

$P_L = q\ C_D\ A_2$ where

q = dynamic pressure head = $0.613\ v_c^2$ in N/m^2 where v_c is in m/s

C_D = drag coefficient, taken as 6.0

A_2 = the net exposed area in m^2 in end view of the sign and gantry structure.

(Amendment No 1 May 1979)

3.3.2.1 Mean hourly wind speed v. Values of v in metres/second for the location of the structure shall be obtained from the map of isotachs shown in Fig 1.

3.3.2.2 Coefficient K_1. The coefficient shall be taken as 1.0 for highway bridges for a return period of 120 years.

For footbridges, sign/signal gantries and other ancillary highway structures, a return period of 50 years is acceptable 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 two days or less and for which reliable wind speed forecasts are available, this predicted speed may be used as the mean hourly wind speed, in which case the value of K_1 shall be taken as 1.0.

For all highway structures, when maximum temperature effects are considered in conjunction with wind, K_1 shall be reduced to 0.3 (see 3.4.3.2 for reduced temperature effects in combination with maximum wind effects).

3.3.2.3 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 structure is sited near hills, buildings or other obstructions which cause local acceleration of wind, a value not less than 1.1 shall be taken, subject to advice from the Meteorological Office who shall be consulted wherever terrain conditions are abnormal.

3.3.2.4 Gust factor S_2. Values of S_2 are given in Table 1. These are valid for sites not exceeding 300 metres above sea level.

Where the structure 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 structures over tidal water, the height above ground shall be measured from the mean water level.

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

The horizontal wind loaded length shall be that giving the most severe effect. Where there is only one adverse area (see 3.3.2.4.1) for the element or structure under consideration the 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 (see 3.3.2.4.1) are subjected to wind having a gust speed as specified in 3.3.2.6.

(Amendment No 1 May 1979)

TABLE 1. VALUES OF GUST FACTOR S_2 AND HOURLY SPEED FACTOR K_2

Height above ground level Metres	HORIZONTAL WIND LOADED LENGTH IN METRES									Hourly speed factor K_2
	20 or less	40	60	100	200	400	600	1000	2000	
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

3.3.2.4.1 Adverse and relieving areas 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 the negative areas of the influence line are referred to as relieving areas.

Conversely, in the consideration of loading effects which are negative, the negative areas of the influence line are referred to as adverse areas and the positive areas of the influence line are referred to as relieving areas.

3.4 Temperature

3.4.1 General. Daily and seasonal fluctuations in shade air temperature, solar radiation, re-radiation, etc cause:

i. Changes in the overall temperature of the bridge. The overall temperature of the bridge is referred to as the effective bridge temperature and is defined as the temperature which governs the longitudinal movement of the deck. Over a prescribed period, there will be a minimum and maximum value of effective bridge temperature. Effective bridge temperatures are derived from the isotherms of shade air temperature shown in Figures 8 and 9. These shade air temperatures are appropriate to mean sea level in open country and a 120 year return period.

ii. Differences in temperature between the top surface and other levels through the depth of the superstructure, referred to as temperature difference. Maximum temperature differences for different types of construction and for a range of depths of surfacing are derived from the information given in 3.4.4.

3.4.2 Minimum and maximum shade air temperatures. Subject to the reservations in 3.4.2.1 these shall be taken from Figs 8 and 9 respectively.

3.4 2.1 Divergence from minimum shade air temperature. There are locations where the minimum values diverge from the values given in Fig 8, for example, frost pockets and sheltered low lying areas where the minimum may be substantially lower, or urban areas (except London) and coastal sites, where the minimum may be higher than that indicated by Fig 8. (In coastal areas, values are likely to be 1°C higher than map values.) These divergences should be investigated and taken into consideration. Consultation with the local Meteorological Office may be helpful in these conditions.

3.4.2.2 Adjustment for height above mean sea level. The values given in Figs 8 and 9 are related to mean sea level and shall be adjusted for height above mean sea level by subtracting 0.5°C per 100 m of height for minimum shade air temperatures and 1.0°C per 100 m of height for maximum shade air temperatures.

3.4.2.3 Adjustment for return period

3.4.2.3.1 Highway bridges. For all highway bridges, a return period of 120 years shall be taken and no adjustment made to the shade air temperatures.

3.4.2.3.2 Footbridges and sign/signal gantries. For footbridges and sign/signal gantries, a return period of 50 years may be adopted and the shade air temperatures adjusted in accordance with 3.4.2.3.5.

3.4.2.3.3 Equipment having a design life less than that of the structure. Carriageway joints and similar equipment which 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 temperatures adjusted in accordance with 3.4.2.3.5.

3.4.2.3.4 During erection. For all highway structures, during erection, a 50 year return period may be adopted and the shade air temperatures adjusted in accordance with 3.4.2.3.5.

In circumstances where a particular erection can 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.

3.4.2.3.5 Adjustment for a 50 year return period. The minimum shade air temperature shall be adjusted by adding 2°C to the appropriate value taken from Fig 8.

The maximum shade air temperature shall be adjusted by subtracting 1°C from the appropriate value taken from Fig 9.

3.4.3 Minimum and maximum effective bridge temperatures. For all highway structures, the minimum and maximum effective bridge temperatures for different types of construction, shall be derived from the minimum and maximum shade air temperatures by reference to Tables 8 and 9 respectively. The different types of construction are shown in Fig 10.

TABLE 8 MINIMUM EFFECTIVE BRIDGE TEMPERATURE °C

Minimum shade air temperature	Minimum effective bridge temperature		
	Group 1 and 2	Group 3	Group 4
- 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 9 MAXIMUM EFFECTIVE BRIDGE TEMPERATURE °C

Maximum shade air temperature	Maximum effective bridge temperature		
	Group 1 and 2	Group 3	Group 4
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

3.4.3.1 Adjustment for depth of surfacing. The effective bridge temperatures are dependent on the depth of surfacing on the bridge deck and the values given in Tables 8 and 9 assume depths of 40 mm for Groups 1 and 2 and 100 mm for Groups 3 and 4. For other depths of surfacing, the minimum and maximum effective bridge temperatures may be adjusted by the amounts given in Table 10.

TABLE 10 ADJUSTMENT TO EFFECTIVE BRIDGE TEMPERATURE FOR DEPTH OF SURFACING

Deck Surface	Addition to Minimum effective bridge temperature °C			Addition to Maximum effective bridge temperature °C		
	Group 1&2	Group 3	Group 4	Group 1&2	Group 3	Group 4
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 †	--	0	0	--	0	0
200 mm surfacing †	--	+ 3	+ 1	--	- 4	- 2

† Surfacing depths include waterproofing.

(Amendment No 1 May 1979)

3.4.3.2 Combination with wind effects. Where loads due to changes in the effective bridge temperature are to be considered in combination with maximum wind loads, the minimum and maximum effective bridge temperatures shall be taken at 0°C and 20°C respectively for all localities and all types of construction. (See 3.3.2.2 for reduced wind effects in combination with extremes of effective bridge temperature.)

3.4.4 Temperature difference. A temperature difference is said to be positive if the temperature of the surface of the deck is higher than the temperature of the deck at the specified depth. Conversely, a temperature difference is said to be reversed if the temperature of the surface of the deck is lower than the temperature of the deck at the specified depth.

For highway and foot bridges, load effects resulting from temperature differences within the superstructure shall be derived from the data given in Fig 10, which **are based on the temperature difference distributions given in LR765*. The distributions in LR765 may, if desired, be used in preference to Fig 10.**

A method of calculating the distribution of temperature is given in LR561** and this method may, if desired be used to obtain temperature differences.

Effects of temperature difference need not be considered in the design of sign/signal gantries.

3.4.4.1 Adjustment for depth of surfacing. Temperature difference distributions are sensitive to the depth of surfacing and the data in Fig 10 relate only to the given depths. For other depths of surfacing, temperature differences may be derived using the data in Appendix B. Alternatively, the appropriate temperature difference distributions given in LR765 may be used.

3.4.4.2 Combination with wind effects. Effects of temperature difference need not be considered in combination with wind effects.

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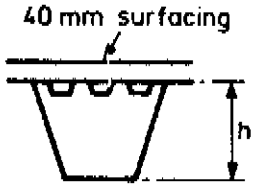
*TRRL Report LR765 'Temperature differences in bridges: basis of design requirements'

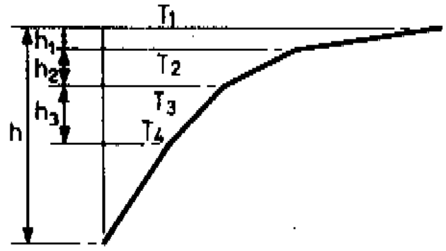
**TRRL Report LR561 'The calculation of the distribution of temperature in bridges'

These reports are available from Transport and Road Research Laboratory, Crowthorne, Berkshire

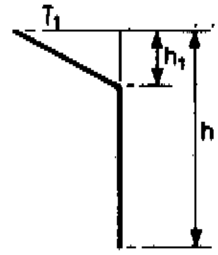
GROUP	TYPE OF CONSTRUCTION	POSITIVE TEMPERATURE DIFFERENCE	REVERSED TEMPERATURE DIFFERENCE

1. Steel deck on steel box girders





$h_1 = 0{\cdot}1$m $T_1 = 24°$C
$h_2 = 0{\cdot}2$m $T_2 = 14°$C
$h_3 = 0{\cdot}3$m $T_3 = 8°$C
$T_4 = 4°$C

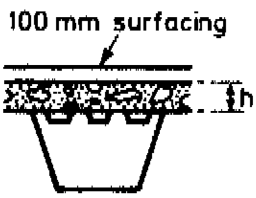


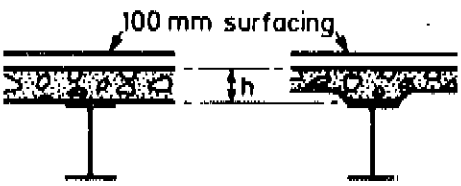
$h_1 = 0{\cdot}5$m
$T_1 = 6°$C

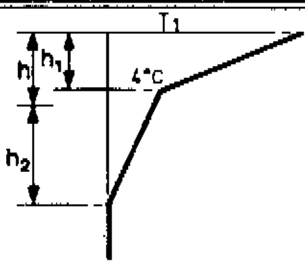
2. Steel deck on steel truss or plate girders

Use differences as for Group 1

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

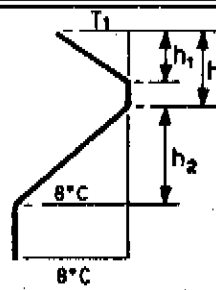






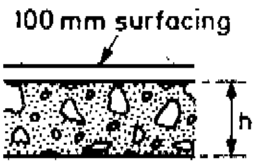
$h_1 = 0{\cdot}6$h
$h_2 = 0{\cdot}4$m

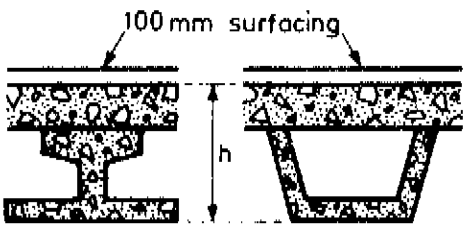
h	T_1
m	°C
0·2	13
0·3	16

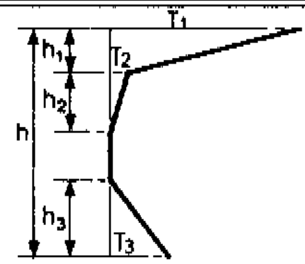


h	T_1
m	°C
0·2	3·5
0·3	5·0

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

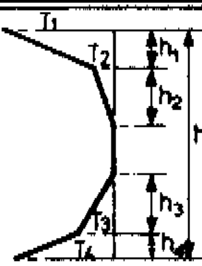






$h_1 = 0{\cdot}3$h, but $\leqslant 0{\cdot}15$m
$h_2 = 0{\cdot}3$h, but $0{\cdot}10\text{m} \leqslant h_2 \leqslant 0{\cdot}25$m
$h_3 = 0{\cdot}3$h, but $\leqslant 0{\cdot}10$m + surfacing in m.
and $\leqslant h - h_1 - h_2$

h	T_1	T_2	T_3
m	°C	°C	°C
⩽ 0·2	8·5	3·5	0·5
0·4	12·0	3·0	1·5
0·6	13·0	3·0	2·0
⩾ 0·8	13·5	3·0	2·5



$h_1 = h_4 = 0{\cdot}2$h, but $\leqslant 0{\cdot}25$m
$h_2 = h_3 = 0{\cdot}25$h, but $\leqslant 0{\cdot}20$m

h	T_1	T_2	T_3	T_4
m	°C	°C	°C	°C
⩽ 0·2	2·0	0·5	0·5	1·5
0·4	4·5	1·4	1·0	3·5
0·6	6·5	1·8	1·5	5·0
0·8	7·6	1·7	1·5	6·0
1·0	8·0	1·5	1·5	6·3
⩾ 1·5	8·4	0·5	1·0	6·5

Figure 10. Temperature difference for different types of construction

(Amendment No 1 May 1979)

3.4.4.3 Combination with effective bridge temperatures. Maximum positive temperature differences can occur during Spring, Summer and Autumn and shall therefore be considered to co-exist with effective bridge temperatures at or above 25°C (Groups 1 and 2) or 15°C (Groups 3 and 4).

Maximum reversed temperature differences can occur at any time of day or night throughout the year but they are not coincident with maximum effective bridge temperatures. This effect shall therefore be considered to co-exist with effective bridge temperatures in the range between the minimum effective bridge temperature and 8°C below the maximum for Groups 1 and 2, 4°C below the maximum for Group 3 and 2°C below the maximum for Group 4.

3.4.5 Coefficient of thermal expansion. For structural steel, the coefficient shall be taken as 12×10^{-6} per °C.

For reinforced and prestressed concrete, values of the coefficient are given in BE 1/73.

3.5 Restraint at bearings. Loads due to expansion and contraction shall be taken into account in accordance with the following clauses.

3.5.1 Roller and sliding bearings. Loads shall be derived using the coefficients of friction given below (the coefficients are based partly on TRRL Report LR382: Notes on Bridge Bearings). Where an appropriate coefficient of friction is not given, this shall be established by testing.

Roller bearings*	μ
Through-hardened special steel with finely ground finish 425-450 HB.	0.01
1 or 2 rollers with as-turned finish in: Mild steel to BS 4360 grade 43 110-150 HB. High tensile steel to BS 4360 grade 50 160-190 HB. Grey cast iron to BS 1452 grade 23 190-220 HB.	0.03
As above but with more than 2 rollers.	0.05

Sliding bearings

For steel on polytetrafluoroethylene (PTFE), the coefficient of friction varies with the average contact pressure and may be taken as follows:

average contact pressure N/mm²	coefficient of friction μ
10	0.06
20	0.04
30	0.03

*The rollers shall be turned so that opposite faces shall be parallel within a tolerance of ± 0.05 mm and the seatings shall be ground in the direction of travel of the rollers so that the variation in flatness at any section parallel to the axis of the roller shall not exceed ± 0.05 mm.

APPENDIX B: VALUES OF 'T' FOR A RANGE OF SURFACING DEPTHS

The values of 'T' for various surfacing thicknesses given below have been derived from TRRL Report LR765. Refer to Figure 10 for the values of 'h' to be used in conjunction with the appropriate values of 'T'.

TABLE B1: VALUES OF T FOR GROUPS 1 AND 2

SURFACING DEPTH (mm)	POSITIVE TEMPERATURE DIFFERENCE °C				REVERSED TEMPERATURE DIFFERENCE °C
	T_1	T_2	T_3	T_4	T_1
Unsurfaced	30	16	6	3	8
20	27	15	9	5	6
40	24	14	8	4	6

TABLE B2: VALUES OF T FOR GROUP 3

DEPTH OF SLAB h (m)	SURFACING DEPTH (mm)	POSITIVE TEMPERATURE DIFFERENCE °C T_1	REVERSED TEMPERATURE DIFFERENCE °C T_1
0.2	Unsurfaced	16.5	5.9
	Waterproofed	23.0	5.9
	50	18.0	4.4
	100	13.0	3.5
	150	10.5	2.3
	200	8.5	1.6
0.3	Unsurfaced	18.5	9.0
	Waterproofed	26.5	9.0
	50	20.5	6.8
	100	16.0	5.0
	150	12.5	3.7
	200	10.0	2.7

(Amendment No 1 May 1979)

TABLE B3: VALUES OF T FOR GROUP 4

DEPTH OF SLAB h (m)	SURFACING DEPTH (mm)	POSITIVE TEMPERATURE DIFFERENCE °C			REVERSED TEMPERATURE DIFFERENCE °C			
		T_1	T_2	T_3	T_1	T_2	T_3	T_4
$\leqslant 0.2$	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
$\geqslant 1.5$	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