VOLUME 1 HIGHWAYS STRUCTURES, APPROVAL PROCEDURES AND GENERAL DESIGN

SECTION 3 GENERAL DESIGN

PART 3

BD 49/01

DESIGN RULES FOR AERODYNAMIC EFFECTS ON BRIDGES

SUMMARY

This Standard sets out the design requirements for bridges with respect to aerodynamic effects including provisions for wind-tunnel testing. It updates and supersedes BD 49/93.

INSTRUCTIONS FOR USE

1. This document supersedes BD 49/93, which is now withdrawn.

2. Remove BD 49/93, which is superseded by BD 49/01 and archive as appropriate.

3. Insert BD 49/01 in Volume 1, Section 3, Part 3.

4. Archive this sheet as appropriate.

Note: A quarterly index with a full set of Volume Contents Pages is available separately from The Stationery Office Ltd.

THE HIGHWAYS AGENCY

SCOTTISH EXECUTIVE DEVELOPMENT DEPARTMENT

THE NATIONAL ASSEMBLY FOR WALES
CYNULLIAD CENEDLAETHOL CYMRU

THE DEPARTMENT FOR REGIONAL DEVELOPMENT*

Design Rules for Aerodynamic Effects on Bridges

* A Government Department in Northern Ireland

Summary: This Standard sets out the design requirements for bridges with respect to aerodynamic effects including provisions for wind-tunnel testing. It updates and supersedes BD 49/93.

Volume 1 home page

REGISTRATION OF AMENDMENTS

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

REGISTRATION OF AMENDMENTS

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

VOLUME 1 HIGHWAYS STRUCTURES, APPROVAL PROCEDURES AND GENERAL DESIGN

SECTION 3 GENERAL DESIGN

PART 3

BD 49/01

DESIGN RULES FOR AERODYNAMIC EFFECTS ON BRIDGES

Contents

1. INTRODUCTION

1.1 Scope

This Standard specifies design requirements for bridges with respect to aerodynamic effects, including provisions for wind-tunnel testing. It supersedes clause 5.3.9 of BS 5400: Part 2(1) and the previous version of this standard BD 49/93. All references to BS 5400: Part 2 are intended to imply the document as implemented by BD 37(2) (DMRB 1.3).

This Standard is applicable to all highway bridges and foot/cycle-track bridges. However its provisions shall only apply to bridges which comply with the constraints outlined herein.

1.2 Design requirements

The aerodynamic aspects of bridge design shall be carried out in accordance with the rules including associated annexes when applicable.

The requirements, in the form of design rules, are given in the following sections. Proximity effects are covered in Annex A. Formulae for the prediction of fundamental frequencies in bending and in torsion are given in Annex B, and further requirements for wind tunnel testing are given in Annex C.

1.3 Background

The original version of these rules first appeared as the "Proposed British Design Rules" in 1981(3). A modified version was included in the TRL Contractor Report 36(4), which also contained the associated partial safety factors and guidance on the use of the rules.

1.4 Major changes in 1993 version of BD 49

In the light of their use in bridge design from 1981, further consideration was deemed necessary with respect to a number of items. The more notable aspects embodied in the 1993 version of BD 49 were the rules which determined whether the designs of certain footbridges and steel plate-girder bridges needed to be based on wind tunnel testing.

Background information on these modifications is available in TRL Contractor Report 256(5).

1.5 Major changes in this version of BD 49

Since the 1993 version of this standard further wind tunnel tests have been carried out and other studies have been undertaken, which have led to further amendments to the Rules. Background is provided in reference (6). The present version of the rules incorporates the outcome of this work, including:

(a) more reliable criteria for plate girder bridges, based on a comprehensive series of tests on wind tunnel models; and

(b) a review of the Rules in the light of experience leading to:

- (i) improved considerations of edge details;
- (ii) amendments to all critical wind speeds;
- (iii) improved accuracy of vortex shedding amplitudes;
- (iv) more accurate criteria for aerodynamic susceptibility; and
- (v) initial guidance on proximity effects.

1.6 Additional guidance

Guidance on the use of the design rules is available in TRL Contractor Report 36(4). Actual bridge configurations being designed that may correlate with sections physically tested previously may benefit from use of the archived test data which are held in the library of the Institution of Civil Engineers(7). Benefit may also be gained from proven theoretical or computational fluid dynamics (CFD) procedures.

1.7 Implementation and Mandatory Requirements

1.7.1 Implementation

This Standard shall be used forthwith for all schemes currently being prepared provided that, in the opinion of the Overseeing Organisation, this would not result in significant additional expense or delay progress. Design organisations shall confirm its application to particular schemes with the Overseeing Organisation.

1.7.2 Mandatory Requirements

Sections of this Standard which are mandatory requirements of the Overseeing Organisation are highlighted by being contained within boxes. The remainder of the document contains advice and enlargement which is commended for consideration.

1.8 General requirements

The adequacy of the structure to withstand the dynamic effects of wind, together with other coincident loadings, shall be verified in accordance with the appropriate parts of BS 5400, as implemented by the Overseeing Organisation. Partial load factors to be used in considering ultimate and serviceability limit states are defined in 4.

Bridges are prone to several forms of aerodynamic excitation which produce motions in isolated vertical bending or torsional modes or, more rarely, in coupled vertical bending-torsional modes. Depending on the nature of the excitation the motions that shall be considered in design are as follows:

(1) limited amplitudes which could cause unacceptable stresses or fatigue damage, see 2.1.1 and 2.1.2;

(2) divergent amplitudes increasing rapidly to large values, which must be avoided, see 2.1.3; and

(3) non-oscillatory divergence due to a form of aerodynamic torsional instability which must also be avoided, see 2.1.4.

1.8.1 Limited amplitude response

Two types of response can occur:

(i) Vortex-induced oscillations: These are oscillations of limited amplitude excited by the periodic cross-wind forces arising from the shedding of vortices alternatively from the upper and lower surfaces of the bridge deck. They can occur over one or more limited ranges of wind speeds. The frequency of excitation may be close enough to a natural frequency of the structure to cause the resonance and, consequently, cross-wind oscillations at that frequency. These oscillations occur in isolated vertical bending and torsional modes.

(ii) Turbulence response: Because of its turbulent nature, the forces and moments developed by wind on bridge decks fluctuate over a wide range of frequencies. If sufficient energy is present in frequency bands encompassing one or more natural frequencies of the structure, vibration may occur.

1.8.2 Divergent amplitude response

Identifiable aerodynamic mechanisms leading to oscillations of this type that can occur include:

(i) galloping and stall flutter - galloping instabilities arise on certain shapes of deck cross-section because of the characteristics of the variation of the wind drag, lift and pitching moments with angle of incidence or time; and

(ii) classical flutter - this involves coupling (i.e. interaction) between the vertical bending and torsional oscillations.

1.8.3 Non-oscillatory divergence

Divergence can occur if the aerodynamic torsional stiffness (i.e. the rate of change of pitching moment with rotation) is negative. At a critical wind speed the negative aerodynamic stiffness becomes numerically equal to the structural torsional stiffness resulting in zero total stiffness.

1.9 Notation

A_j Enclosed area of cell of box girder bridge at mid-span

b Overall width of bridge deck

$b^/$ Overall width of neighbouring bridge

b* Effective width of bridge deck

c Amplitude correction factor

C_g Parameters used in determination of V_{Rg} and V_g

C_s Coefficient to take account of the extent of wind speed range over which oscillation may occur

C_θ Relative frequency of occurrence of winds within ±10° of normal to the longitudinal centre line of the bridge in strong winds

d_4 Depth of bridge deck

$d^/$ Depth of neighbouring bridge

E Modulus of elasticity

f_B Natural frequency in bending

f_T Natural frequency in torsion

g Acceleration due to gravity

G Clear gap between parallel bridges

G_1 Minimum gap between parallel bridges

G_2 Maximum gap between parallel bridges

h Height of bridge parapet or edge member above deck level

I_b Second moment of mass of the bridge cross section for vertical bending

I_j Second moment of mass of individual box for vertical bending at mid-span

I_P Polar second moment of mass of the bridge cross section at mid-span

J_j Torsion constant of individual box at mid-span

k Depth of fascia beam or edge slab

k_s Modal bending moment factor

K Factor used in the calculation of natural frequency

K_{1A} Probability coefficient

K_D Dynamic sensitivity factor

L Length of main span of bridge

L_1 Length of longer side span of bridge

L_2 Length of shorter side span of bridge

m Mass per unit length of bridge

n Number of stress cycles per annum

p Frequency of occurrence of wind speeds within ±2½% of the critical wind speed

P_b Aerodynamic susceptibility parameter

P_T Turbulence susceptibility parameter

P_1, P_2, P_3 Factors used in calculation of fundamental torsional frequency of box girder bridge

r Polar radius of gyration of the effective bridge cross section

R_e Reynold's number

S_c Hourly speed factor, as per BD 37 (DMRB 1.3)

S_g Gust factor, as per BD 37 (DMRB 1.3)

S_m Hourly mean speed factor, as per BD 37 (DMRB 1.3)

t Thickness of box

V_d Maximum wind gust speed derived for the relevant maximum span as per BD 37 (DMRB 1.3)

V_r Hourly mean wind speed for relieving areas, as per BD 37 (DMRB 1.3)

V_s Site hourly mean wind speed (10m above ground level) as per BD 37 (DMRB 1.3)

V_{cr} Critical wind speed for vortex shedding

$V^/_{cr}$ Critical wind speed for vortex shedding for the estimation of fatigue damage

V_f Critical wind speed for classical flutter

V_g Critical wind speed for galloping and stall flutter

V_{vs} Reference wind speed for vortex shedding

V_{Rf} Reduced critical wind speed for classical flutter

V_{Rg} Reduced critical wind speed for galloping and stall flutter

V_{WO} Wind speed criteria for section model testing of divergent amplitude response

V_{WE} Wind speed criteria for full model testing of divergent amplitude response

$V_{W\alpha}$ Wind speed criteria for section model testing of divergent amplitude response when considering inclined wind

w Weight per unit length of bridge (dead load plus superimposed dead load) at mid-span

w_D Weight per unit length of deck only at mid-span

w_j Weight per unit length of individual box of box girder bridge

y_{max} Maximum amplitude of vibration of the deck

α Inclination of wind to horizontal to be considered in wind tunnel tests

$\overline{\alpha}$ Inclination of wind to horizontal due to local topography

δ_s Structural damping expressed as logarithmic decrement

γ_{fL} Partial load factor

σ_c Reference stress

σ_{f1m} Peak stress in structure per unit deflection in the first mode of vibration

σ_r Stress range

ϕ Solidity ratio of parapet, or ratio of net total projected area presented to the wind to the total area encompassed by the outer boundaries of the deck for trusses

ρ Density of air (for the United Kingdom ρ may be taken as 1.225 kg/m^3)

ν Poisson's ratio

NOTE: Consistent units should be used for m and ρ.

2. SUSCEPTIBILITY TO AERODYNAMIC EXCITATION

This section shall be used to determine the susceptibility of a bridge to aerodynamic excitation. If the structure is found to be susceptible to aerodynamic excitation then the additional requirements of 3 shall be followed.

2.1 Criteria for applicability and consideration of aerodynamic effects

The aerodynamic susceptibility parameter, P_b, shall be derived in order to categorise the structure using the equation:

$$P_b = \left(\frac{\rho b^2}{m}\right)\left(\frac{16V_r^2}{bLf_B^2}\right)$$

where

ρ is the density of air (see NOTE 1);

b is the overall width of the bridge deck (see Figure 1);

m is the mass per unit length of the bridge (see NOTE 1);

V_r is the hourly mean wind speed (for relieving areas) as per BD 37 (DMRB 1.3);

L is the length of the relevant maximum span of the bridge;

f_B is the natural frequency in bending (see NOTE 2).

The bridge shall then be categorised as follows:-

(a) Bridges designed to carry the loadings specified in BD 37 (DMRB 1.3), built of normal construction, are considered to be subject to insignificant effects in respect of all forms of aerodynamic excitation when $P_b < 0.04$. However the Rules can still be applied if required, provided the constraints of 2.3 are satisfied.

(b) Bridges having $0.04 \leq P_b \leq 1.00$ shall be considered to be within the scope of these rules, provided the geometric constraints of 2.3 are satisfied, and shall be considered adequate with regard to each potential type of excitation if they satisfy the relevant criteria given in 2.1.1, 2.1.2 and 2.1.3.

(c) Bridges with $P_b > 1.00$ shall be considered to be potentially very susceptible to aerodynamic excitation: see 2.2.

For the purpose of this categorisation, normal construction may be considered to include bridges constructed in steel, concrete, aluminium or timber, including composite construction, and whose overall shape is generally covered by Figure 1.

Normal highway bridges of less than 25m span should generally be found to be category (a). Bridges of spans greater than 250m are likely to be category (c).

Covered footbridges, cable supported bridges and other structures where any of the parameters b, L or f_B cannot be accurately derived shall be considered as category (c).

The application of these Rules to bridges of novel design shall be agreed with the Overseeing Organisation.

The calculation of V_r should take account of sites where the wind flow may be abnormally affected by steep sloping valleys, unusual terrain or topography. The treatment for the application of the Rules for twin deck configurations and the treatment of proximity effects are given in Annex A.

NOTE 1: Units shall be applied consistently, particularly with respect to ρ and m; preferably ρ should be in kg/m^3, and the material density used for the structure should also be in kg/m^3, with other parameters all in consistent units.

NOTE 2: Frequencies should be derived by dynamic/eigenvalue analysis of the structure; see Annex B which contains approximate formulae for standard bridge arrangements.

NOTE 3: For the purposes of initial/ preliminary categorisation, the following may be used to given an indicative range for P_b:

V_r between 20 and 40 m/s;

m/b between 600 and 1200 kg/m^2;

f_B between $50/L^{0.87}$ and $100/L^{0.87}$, but see also NOTE 2.

Appropriate upper bound and lower bound values should be derived.

2.1.1 Limited amplitude response - vortex excitation

2.1.1.1 General

Estimates of the critical wind speed for vortex excitation for both bending and torsion (V_{cr}) shall be derived according to 2.1.1.2. For certain truss girder bridges see 2.1.1.3(c). The limiting criteria given in 2.1.1.3 shall then be satisfied.

2.1.1.2 Critical wind speeds for vortex excitation

The critical wind speed for vortex excitation, V_{cr}, is defined as the velocity of steady air flow or the mean velocity of turbulent flow at which maximum aerodynamic excitation due to vortex shedding occurs and shall be calculated as follows for both vertical bending and torsional modes of vibration of box and plate girder bridges. Alternatively V_{cr} may be determined by appropriate wind tunnel tests on suitable scale models. For truss bridges with solidity $\phi < 0.5$, refer to 2.1.1.3(c). When $\phi \geq 0.5$ the equations for plate girders may be used conservatively, but taking the depth d_4 as ϕd_4 (see 2.3 and Figures 1 and 2).

b^*/d_4	V_{cr} for bridge types 1, 1A, 3, 3A 4, 4A	V_{cr} for bridge types 2, 5, 6
≤5	$6.5fd_4$	$6.5fd_4$
>5 <10	$fd_4(1.1b^*/d_4+1.0)$	$fd_4(0.7b^*/d_4+3.0)$
≥10	$12fd_4$	$10fd_4$

In these equations:

b^* is the effective width of the bridge as defined in Figure 1;

d_4 is the depth of the bridge shown in Figure 1. Where the depth is variable over the span, d_4 shall be taken as the average value over the middle third of the longest span;

f is either f_B or f_T as appropriate, i.e. the natural frequencies in bending and torsion respectively (Hz) calculated under dead and superimposed dead load. Methods of calculating approximate values of f_B and f_T, within certain constraints, are given in Annex B.

2.1.1.3 Limiting criteria

The following conditions shall be used to determine the susceptibility of a bridge to vortex excited vibrations:-

(a) Any bridge whose fundamental frequency is greater than 5Hz shall be considered stable with respect to vortex excitation.

(b) Any bridge, including truss bridges (see also (c)), shall be considered stable with respect to vortex excited vibrations if the lowest critical wind speeds, V_{cr}, for vortex excitation in both bending and torsion, as defined in 2.1.1.2, exceed the value of reference wind speed V_{vs}, where:

V_{vs} = $1.25 V_r$;

V_r is the hourly mean wind speed in accordance with BD 37 (DMRB 1.3) for relieving areas of the bridge deck derived in accordance with BD 37 (DMRB 1.3).

(c) In addition, truss girder bridges shall be considered stable with regard to vortex excited vibrations provided $\phi < 0.5$, where ϕ is the solidity ratio of the front face of the windward truss, defined as the ratio of the net total projected area of the truss components to the projected area encompassed by the outer boundaries of the truss (i.e. excluding the depth of the deck). For trusses with $\phi \geq 0.5$, refer to 2.1.1.2.

If any one of (a), (b) or (c) is satisfied, then the bridge shall be deemed stable with respect to the effects of vortex excitation. If none of these conditions is satisfied, then the effects of vortex excitation shall be considered in accordance with 3.1.

2.1.2 Limited amplitude response - turbulence

Provided the fundamental frequencies in both bending and torsion calculated in accordance with 2.1.1.2 are greater than 1Hz, the dynamic magnification effects of turbulence may be ignored.

The dynamic magnification effects of turbulence may also be neglected if:

$P_T \leq 1.0$

where

$$P_T = \left(\frac{\rho b^2}{m}\right)\left(\frac{V_s}{f_B b}\right)^2 \frac{\sigma_{flm}.b}{\sigma_c}$$

ρ, b, m, f_B are all as defined in 2.1;

V_s is the site hourly mean wind speed (10m above ground level) as per BD 37 (DMRB 1.3);

σ_{flm} is the peak stress in the structure per unit deflection in the first mode of vibration, derived for the most highly stressed location in the relevant element:

σ_c is a reference stress as follows:

for steel beam elements, $\sigma_c = 600$ N/mm² for the longitudinal flange bending stress; or

for truss bridges, $\sigma_c = 750$ N/mm² for the chord axial stress; or

for concrete elements (composite or concrete bridges), $\sigma_c = 80$ N/mm² for the primary bending concrete stress; or

for cable-stayed bridges the peak stay axial stress should additionally be examined, with σ_c = 1200 N/mm².

If these conditions are not satisfied the dynamic effects of turbulence response shall be considered in accordance with 3.3.

2.1.3 Divergent amplitude response

2.1.3.1 General

Estimates of the critical wind speed for galloping and stall flutter for both bending and torsional motion (V_g) and for classical flutter (V_f) shall be derived according to 2.1.3.2 and 2.1.3.3 respectively. Alternatively values of V_g and V_f may be determined by wind tunnel tests (see 6). The limiting criteria given in 2.1.3.4 shall then be satisfied.

2.1.3.2 Galloping and stall flutter

(a) Vertical motion

Vertical motion need be considered only for bridges of types 3, 3A, 4 and 4A as shown in Figure 1, and only if $b < 4d_4$.

Provided the constraints (i) to (iii) in 2.3 are satisfied V_g shall be calculated from the reduced velocity, V_{Rg}, using the formula below:

$V_g = V_{Rg} f_B d_4$

where

$$V_{Rg} = \frac{C_g\,(m\delta_s)}{\rho d^2_4}$$

where

f_B, m and ρ are as defined in 2.1;

C_g is 2.0 for bridges of type 3 and 4 with side overhang greater than $0.7d_4$ or 1.0 for bridges of type 3, 3A, 4 and 4A with side overhang less than or equal to $0.7d_4$;

δ_s is the logarithmic decrement of damping, as specified in 3.1.2,

d_4 is the reference depth of the bridge shown in Figure 1, as defined in 2.1.1.2.

Alternatively, wind tunnel tests shall be undertaken to determine the value of V_g.

(b) Torsional motion

Torsional motion shall be considered for all bridge types. Provided the fascia beams and parapets comply with the constraints given in 2.3, then V_g shall be taken as:

V_g $= 3.3\, f_T\, b$ for bridge types 1, 1A, 2, 5 and 6;

V_g $= 5\, f_T\, b$ for bridge types 3, 3A, 4 and 4A.

For bridges of type 3, 3A, 4 and 4A (see Figure 1) having $b < 4d_4$, V_g shall be taken as the lesser of $12\, f_T\, d_4$ or $5\, f_T\, b$

where

f_T is the natural frequency in torsion in Hz as defined in 2.1.1.2;

b is the total width of bridge;

d_4 is as given in Figure 1.

2.1.3.3 Classical flutter

The critical wind speed for classical flutter, V_f, shall be calculated from the reduced critical wind speed:

$$V_{Rf} = \frac{V_f}{f_T b}$$

ie $V_f = V_{Rf} f_T b$

where

$$V_{Rf} = 1.8 \left(1\text{-}1.1 \left(\frac{f_B}{f_T}\right)^2\right)^{1/2} \left(\frac{mr}{\rho b^3}\right)^{1/2}$$

but not less than 2.5.

f_T, f_B, m, ρ and b are defined in 2.1;

r is the polar radius of gyration of the effective bridge cross section at the centre of the main span (polar second moment of mass/mass)$^{1/2}$.

Alternatively the value of V_f may be determined by wind tunnel tests; see 6.

2.1.3.4 Limiting criteria

The bridge shall be shown to be stable with respect to divergent amplitude response in wind storms up to wind speed V_{WO}, given by:

$$V_{WO} = \frac{1.10}{3} (V_r + 2V_d)\, K_{1A}$$

where

V_r is the hourly mean wind speed derived in accordance with BD 37 (DMRB 1.3);

V_d is the maximum wind gust speed derived in accordance with BD 37 (DMRB 1.3) for the relevant maximum span;

K_{1A} is a coefficient selected to give an appropriate low probability of occurrence of these severe forms of oscillation. For locations in the UK, $K_{1A} = 1.25$*.

(*Note: a higher value is appropriate for other climatic regions, eg typically K_{1A} = 1.4 for a tropical cyclone-prone location).

Where the values of V_g or V_f derived in accordance with 2.1.3.2 or 2.1.3.3 respectively are lower than V_{WO} further studies as noted in 1.6 or wind-tunnel tests in accordance with 3.2 shall be undertaken.

2.1.4 Non-oscillatory divergence

A structure shall be considered stable for this motion if the criteria in 2.1.3 above are satisfied.

2.2 Bridges requiring special consideration

The stability of all bridges exceeding the susceptibility criteria (c) in 2.1 are outside the scope of the rules provided and shall be verified by means of further studies as noted in 1.6, or through wind tunnel tests on scale models in accordance with 6.

2.3 Geometric constraints

For applicability of the reduced velocities for divergent amplitude response (2.1.3.2) and the vortex shedding maximum amplitude derivation (3.1), the following constraints shall be satisfied:

(i) Solid edge members, such as fascia beams and solid parapets shall have a total depth less than $0.2d_4$ unless positioned closer than $0.5d_4$ from the outer girder when they shall not protrude above the deck by more than $0.2d_4$ nor below the deck by more than $0.5d_4$. In defining such edge members, edge stiffening of the slab to a depth of 0.5 times the slab thickness may be ignored.

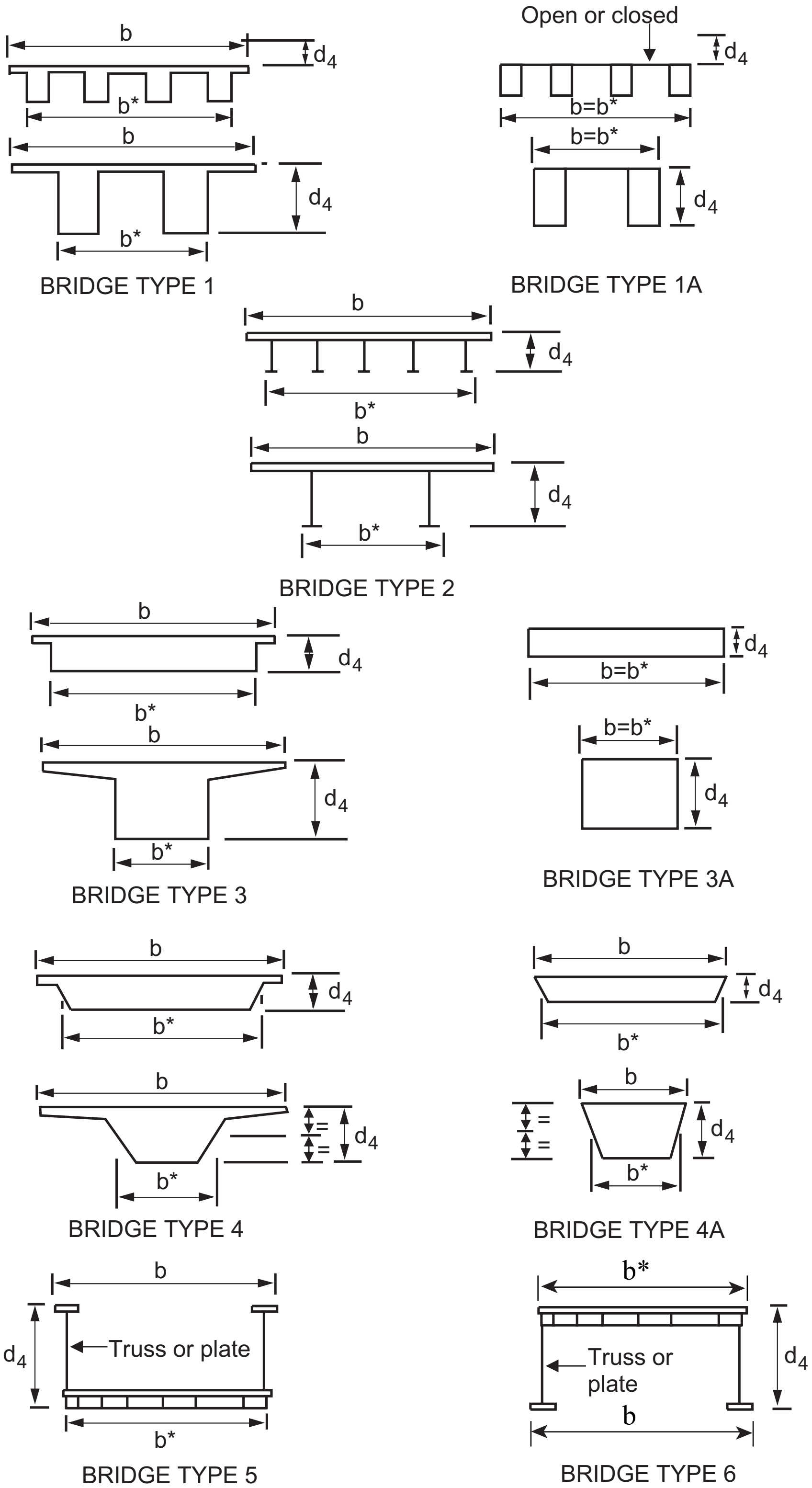
(ii) Other edge members such as parapets, barriers, etc., shall have a height above deck level, h, and a solidity ratio, ϕ, such that ϕ is less than 0.5 and the product $h\phi$ is less than $0.35d_4$ for the effective edge member. The value of ϕ may exceed 0.5 over short lengths of parapet, provided that the total length projected onto the bridge centre-line of both the upwind and downwind portions of parapet whose solidity ratio exceeds 0.5 does not exceed 30% of the bridge span.

(iii) Any central median barrier shall have a shadow area in elevation per metre length less than $0.5m^2$. Kerbs or upstands greater than 100mm deep shall be considered as part of this constraint by treating as a solid bluff depth; where less than 100mm the depth shall be neglected, see Figure 2.

In the above, d_4 is the reference depth of the bridge deck (see Figures 1 and 2). Where the depth is variable over the span, d_4 shall be taken as the average value over the middle third of the longest span.

2.4 Proximity effects

Guidance on the effect of obstacles in the path of the wind is given in Annex A. The guidance for ‘twin-deck’ bridges should be particularly noted.



Note: For truss bridges of type 5 or 6,
d_4 taken as $Ød_4$, where Ø is the truss solidity.

Fig. 1 Bridge types and reference dimensions
Note: For twin-deck bridges, see Annex A

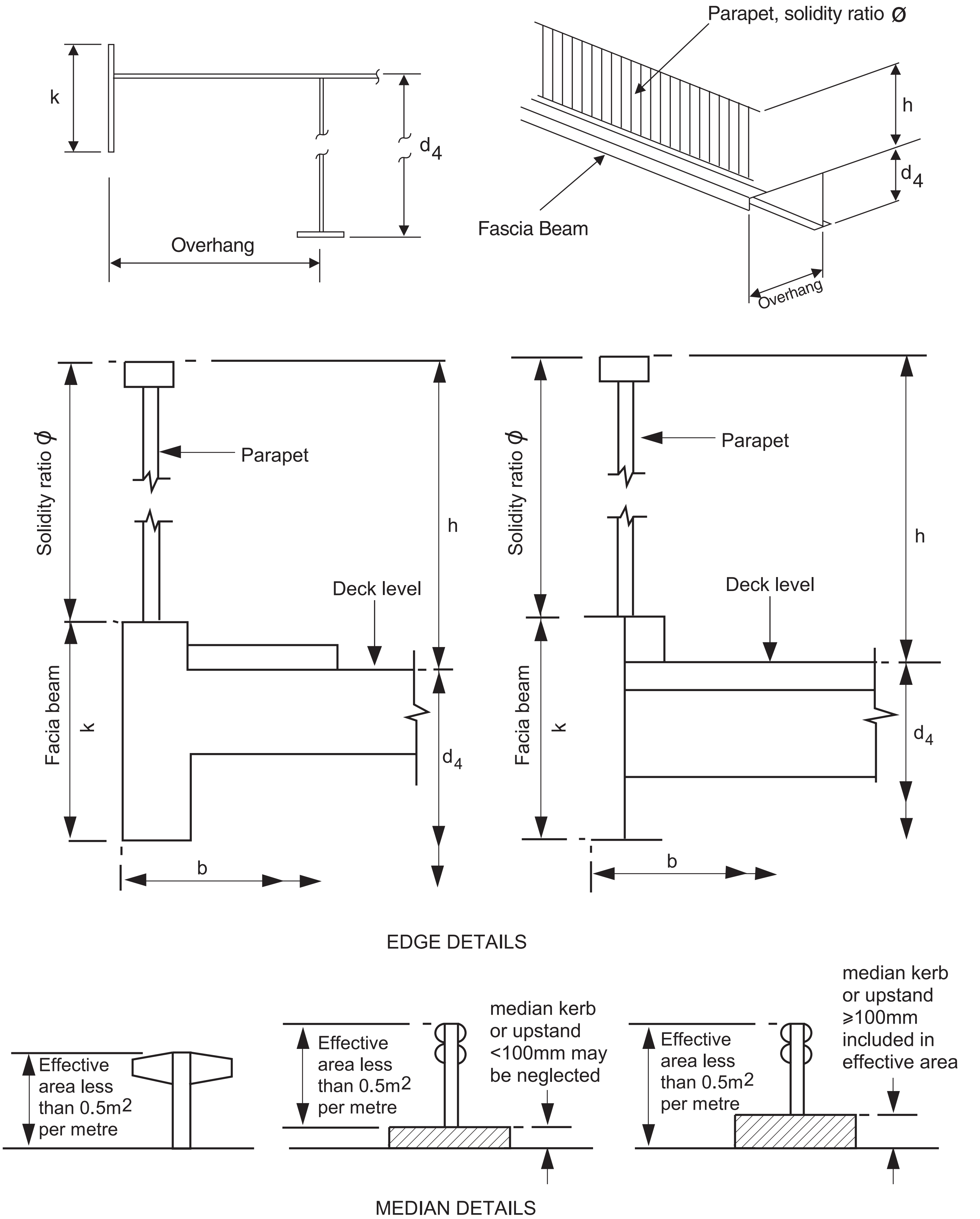


Fig. 2 Bridge deck details
for geometric constraints, see 2.3

3. ADDITIONAL REQUIREMENTS

If the bridge is found to be susceptible to aerodynamic excitation in accordance with the criteria in 2.1, then the following additional requirements shall be considered (see also 4).

3.1 Vortex excitation effects

3.1.1 General

Where the bridge cannot be assumed to be aerodynamically stable against vortex excitation in accordance with 2.1.1 above, consideration shall be given to:

(i) the effects of maximum oscillations of any one of the motions considered singly, calculated in accordance with 3.1.2 together with the effects of other coincident loading (see 4);

(ii) fatigue damage, assessed in accordance with 5 summated with damage from other loading.

3.1.2 Amplitudes

The maximum amplitudes of flexural and torsional vibrations, y_{max}, shall be obtained for each mode of vibration for each corresponding critical wind speed less than V_r as defined in 2.1.1.3(b).

The amplitudes of vibration, y_{max}, from mean to peak, for flexural and torsional models of vibration of box and plate girders and for flexural modes of vibration of trusses may be obtained from the formulae below provided that the following conditions are satisfied:

(a) For all bridge types, edge and centre details conform with the constraints given in 2.3.

(b) The site, topography and alignment of the bridge are such that the consistent vertical inclination of the wind to the deck of the bridge, due to ground slope, does not exceed ±3°.

NOTE: The formulae below provide an approximate value to the amplitudes. However if the consequences of such values in the design are significant then wind tunnel tests shall be considered.

For vertical flexural vibrations:

$$y_{max} = \frac{c b^{0.5} d_4^{\,2.5} \rho}{4 m \delta_s}$$

for bridge types 1 to 6

For torsional vibrations:

$$y_{max} = \frac{c b^{1.5} d_4^{\,3.5} \rho}{8 m r^2 \delta_s}$$

for bridge types 1, 1A, 3, 3A, 4 and 4A.

y_{max} may be ignored for torsional vibrations for bridge types 2, 5 and 6.

In these equations:

$$c = \frac{3\,(k + h\phi)}{d_4} \quad \text{but} \nless 0.5$$

b, m and ρ are defined in 2.1;

r is as defined in 2.1.3.3;

δ_s is the logarithmic decrement due to structural damping;

h, d_4 and ϕ are as defined in 2.3; and

k is the depth of fascia beam or edge slab (see Figure 2).

The following values of δ_s shall be adopted unless appropriate values have been obtained by measurements on bridges similar in construction to that under consideration and supported on bearings of the same type. If the bridge is cable supported the values given shall be factored by 0.75.

Material of construction	δ_s
Steel	0.03
Steel and Concrete Composite	0.04
Concrete	0.05
Timber (see NOTE 2)	0.06-0.12
Aluminium Alloy	0.02
Glass or Fibre Reinforced Plastic (see NOTE 2)	0.04-0.08

NOTE 1: Low wind speeds, where V_{cr} is less than about 10 m/s, may need special study; an approximate way to cater for this is for δ_s to be factored by $(V_{cr}/1.25V_r)^{½}$ but $\ngtr$ 1.00, but with a limit of $\delta_s \nless 0.02$, where V_{cr} and V_r are as defined in 2.1.1.2 and 2.1.1.3 respectively.

NOTE 2: i) The values for timber and plastic composites are indicative only; in cases where aerodynamic effects are found to be significant in the design, more exact figures should be obtained from specialist sources and agreed with the Overseeing Organisation.

ii) Alternatively, maximum amplitudes of all bridges may be determined by appropriate wind tunnel tests on suitable scale models, or from previous results on similar sections (see 1.6).

The amplitudes so derived shall be considered as maxima and be taken for all relevant modes of vibration. To assess the adequacy of the structure to withstand the effects of these predicted amplitudes, the procedure set out in 3.1.3 shall be followed.

3.1.3 Assessment of vortex excitation effects

A dynamic sensitivity parameter, K_D, shall be derived, as given by:

$K_D = y_{max}\, f_B^2$ for bending effects
$K_D = y_{max}\, f_T^2$ for torsional effects

where

y_{max} is the predicted bending or torsional amplitude (in mm) obtained from 3.1.2,

f_B, f_T are the predicted frequencies (in Hz) in bending and torsion respectively.

Table 1 then gives the equivalent static loading that shall be used, if required, dependent on the value of K_D, to produce the load effects to be considered in accordance with 4 and 5.

Table 1 gives an indication of the relative order of discomfort levels for pedestrians according to the derived value of K_D and indicates where a full discomfort check may be required.

In particular, if K_D is greater than 30mm/s^2 and the critical wind speed for excitation of the relevant mode is less than 20m/s, detailed analysis should be carried out to evaluate K_D. If K_D is still found to be greater than 30mm/s^2, pedestrian discomfort may be experienced and the design should be modified, unless agreed otherwise with the Overseeing Organisation.

3.2 Divergent amplitude effects

3.2.1 Galloping and stall flutter

If the bridge cannot be assumed to be stable against galloping and stall flutter in accordance with 2.1.3.2 it shall be demonstrated by means of a special investigation (or use of previous results, see 1.6) that the wind speed required to induce the onset of these instabilities is in excess of V_{WO} (see 2.1.3.4 and chapter 6). It shall be assumed that the structural damping available corresponds to the values of δ_s given in 3.1.2.

3.2.2 Classical flutter

If the bridge cannot be assumed to be stable against classical flutter in accordance with 2.1.3.3 it shall be demonstrated by appropriate wind tunnel tests on suitable scaled models (see 6) (or use of previous results, see 1.6) that the critical wind speed, V_f, for classical flutter is greater than V_{WO} (see 2.1.3.4 and chapter 6).

3.3 Turbulence response

If the dynamic response to gusts cannot be ignored (see 2.1.2) a dynamic analysis shall be carried out to calculate the peak amplitudes and modes of vibration under an hourly mean wind speed of V_r (see BD 37 (DMRB 1.3)). These shall be used to assess the adequacy of the structure in accordance with 4. Proximity effects (wake buffeting) shall be considered and specialist advice should be sought where indicated for gaps in the range given in A4.

<table>
<tr><td rowspan="3">K_D mm/s^2
(See Note 1)</td><td colspan="2">Vertical load due to vortex excitation expressed as a percentage (α) of the total unfactored design dead plus live load on the bridge</td><td rowspan="2">Motion discomfort only for V_{cr} < 20m/s (see Note 2)</td></tr>
<tr><td>A</td><td>B</td></tr>
<tr><td>All bridges except those in B</td><td>Simply supported highway bridge and all concrete footbridges</td><td>All bridges</td></tr>
<tr><td rowspan="5">≥100
50
30
20
10
5
3
2
1</td><td>α may be greater than 20%: Assess by analysis using derived y_{max}</td><td rowspan="2">α may be greater than 25%. Assess by analysis using derived y_{max}</td><td>Pedestrian discomfort possible (see Note 2)</td></tr>
<tr><td>Assess by analysis using derived y_{max} or for simplicity use upper bound load, $\alpha = 0.4K_D$</td><td>Unpleasant</td></tr>
<tr><td rowspan="3">α is less than 4% and may be neglected</td><td rowspan="2">Assess by analysis using derived y_{max} or for simplicity use upper bound load, $\alpha = 2.5K_D$</td><td>Tolerable</td></tr>
<tr><td>Acceptable</td></tr>
<tr><td>α is less than 5% and may be neglected</td><td>Only just perceptible</td></tr>
</table>

Table 1 Assessment of Vortex Excitation Effects

Note 1: $K_D = f^2 y_{max}$ where f is the natural frequency in Hz, y_{max} is the maximum predicted amplitude in mm, α is the percentage of the total nominal dead plus live load to be applied as the loading due to vortex excitation.

Note 2: When the critical wind speed for excitation in the relevant mode is greater than 20 m/s, motion discomfort is generally not experienced by any pedestrians still using the bridge due to the strength and buffeting effects of the associated gale force winds. For more information see references 4 and 5.

4. DESIGN VALUES FOR WIND LOADS INCLUDING AERODYNAMIC EFFECTS

4.1 Load Combinations

The load combinations at ultimate limit state (ULS) and serviceability limit state (SLS) specified in clause 5.3.6 and Table 1 of BD 37 (DMRB 1.3) shall be considered, as modified for aerodynamic effects below.

When vibrations are predicted to occur due to vortex excitation (see 3.1) and/or turbulence response (see 3.3), the global aerodynamic load effects to be applied to the bridge structure shall be derived in accordance with 3.1.3 for the mode of vibration under consideration, using the maximum amplitude as obtained from 3.1.2 and 3.3 as appropriate. These load effects shall then be multiplied by the partial load factor, γ_{fL}, given below:

Load Combination		ULS	SLS
(a)	*Wind loads derived in accordance with BD 37 (DMRB 1.3) or turbulence response derived in accordance with 3.3 according to the following case with which they are considered*		
(i)	erection	1.1	1.0
(ii)	dead loads plus superimposed dead load only, and for members primarily resisting wind loads	1.4*	1.0
(iii)	appropriate combination 2 loads	1.1	1.0
(b)	*Aerodynamic effects (vortex shedding) derived in accordance with 3.1 considered with cases (i) to (iii) in (a) but using wind loads appropriate to V_{cr} for the mode of vibration under consideration for vortex excitation*	1.2	1.0
For relieving effects of wind in (a) or (b)		1.0	1.0

NOTE: The factor γ_{fL} on permanent and live loads associated with (a) or (b) shall be as per combination 2 in table 1 of BD 37 (DMRB 1.3).

NOTE*: A higher value is appropriate for other climatic regions, eg the factor γ_{fL} for ULS shall be separately derived and is likely to be increased to the order of 1.7 to 2.3 for tropical cyclone locations. Specialist advice should be sought before application to other climatic regions.

5. FATIGUE DAMAGE

5.1 Fatigue damage requirements

All bridges which fail to satisfy the requirements of 2.1.1 shall be assessed for fatigue damage due to vortex excited vibration in addition to fatigue damage due to other load effects.

5.2 Fatigue damage due to vortex excitation

An estimate of the cumulative fatigue damage due to vortex excitation shall be made in accordance with BS 5400: Part 10 as implemented by BA 9/81 (DMRB 1.3) by considering the stress range and number of cycles specified below, for each model in which V_{cr} is less than V_{vs}.

where

V_{cr} is defined in 2.1.1.2,

V_{vs} is defined in 2.1.1.3.

The stress range σ_r shall be taken as 1.2 times the unfactored stress determined from the load effects derived in 3.1.3. The effective number of cycles per annum, n, shall be calculated from:

$n = 2500\, f\, p\, C_\theta C_s$

where

f is the natural frequency of the given mode and p, C_θ and C_s are given in Figures 3, 4 and 5 respectively,

p is the frequency of occurrence, in hours per year, of wind speeds within ±2½% of the critical wind speed, V'_{cr} defined below irrespective of direction,

C_θ is the relative frequency of occurrence of winds within ±10° of normal to the longitudinal centre line of the bridge in strong winds,

C_s takes account of the extent of the range of wind speeds over which oscillation may occur.

The critical wind speed for the estimation of fatigue damage, V'_{cr} for all bridge types in Figure 1, shall be increased to:

$V'_{cr} = 6.5\, fd_4$	for $b^*/d_4 < 1.25$
$V'_{cr} = (0.8\, b^*/d_4 + 5.5)\, fd_4$	for $1.25 \le b^*/d_4 < 10$
$V'_{cr} = 13.5\, fd_4$	for $b^*/d_4 \ge 10$

where

b*, f and d_4 are defined in 2.1.1.2 but noting that d_4 is replaced by ϕd_4 for trusses with $\phi > 0.5$.

Alternatively V'_{cr} shall be assessed from appropriate wind tunnel tests.

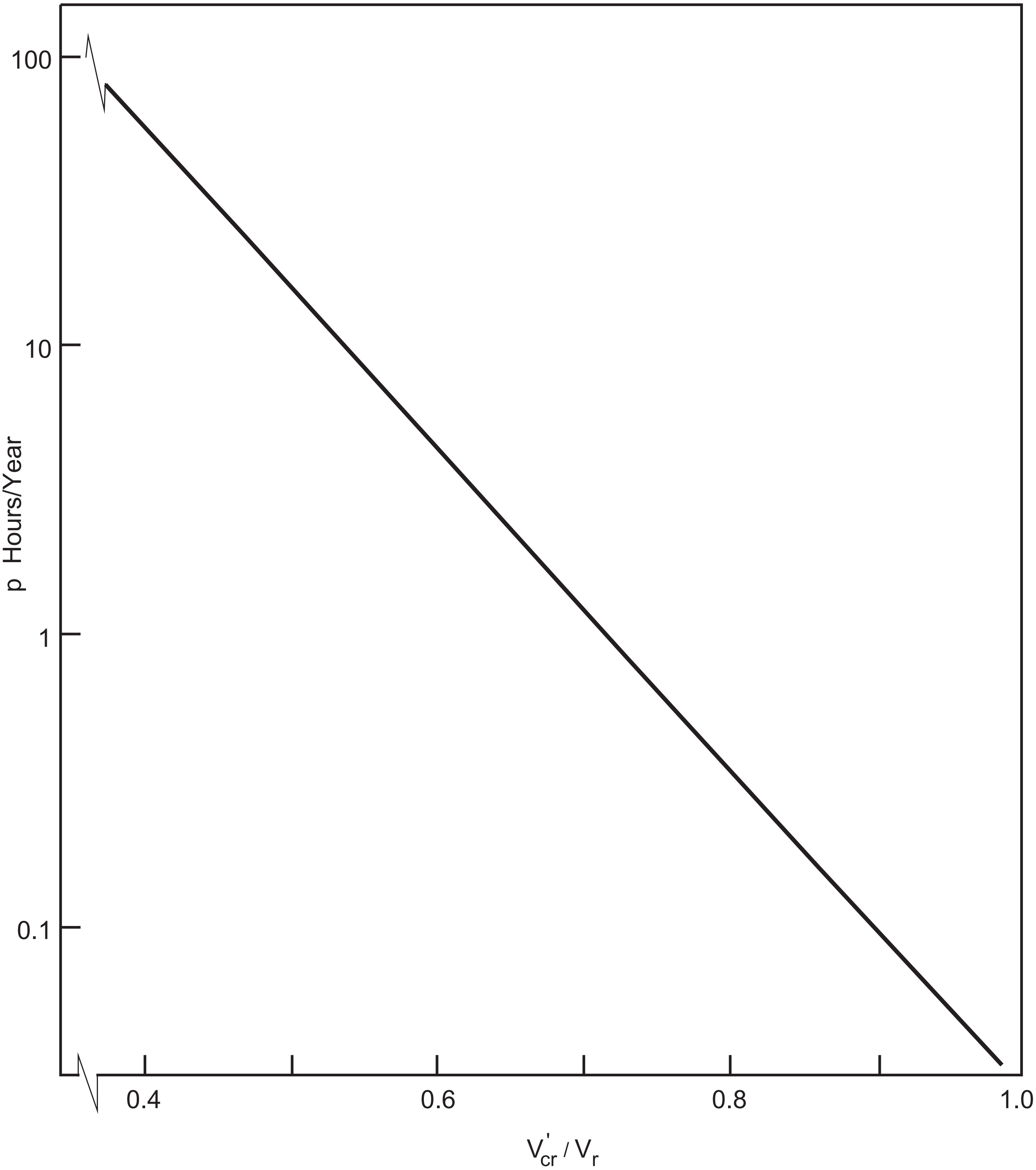


Fig. 3 Expected frequency of occurrence of critical wind speed
(Hours per annum of occurrence of speed within ± 2.5% of critical value)

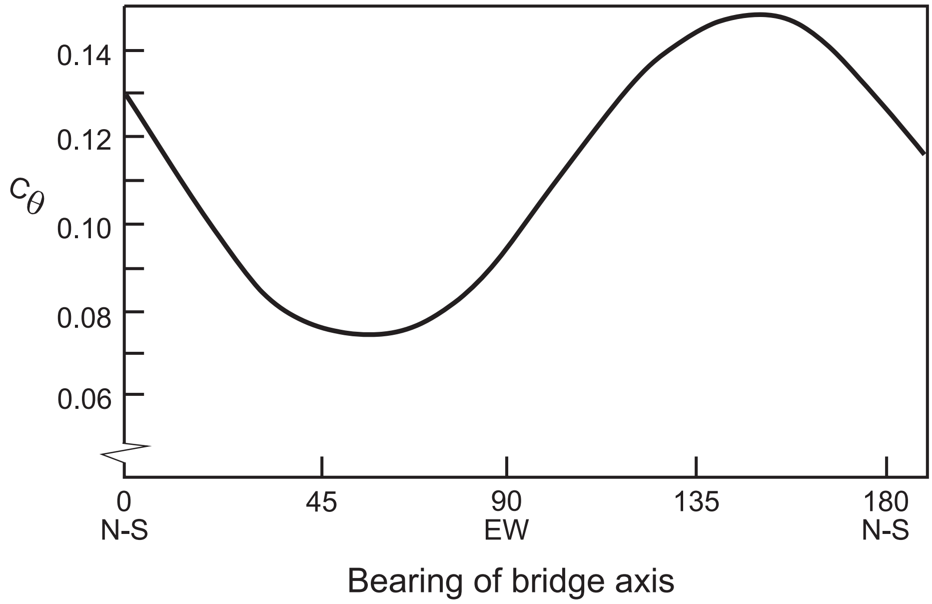


Fig. 4 Factor for orientation of bridge in plan

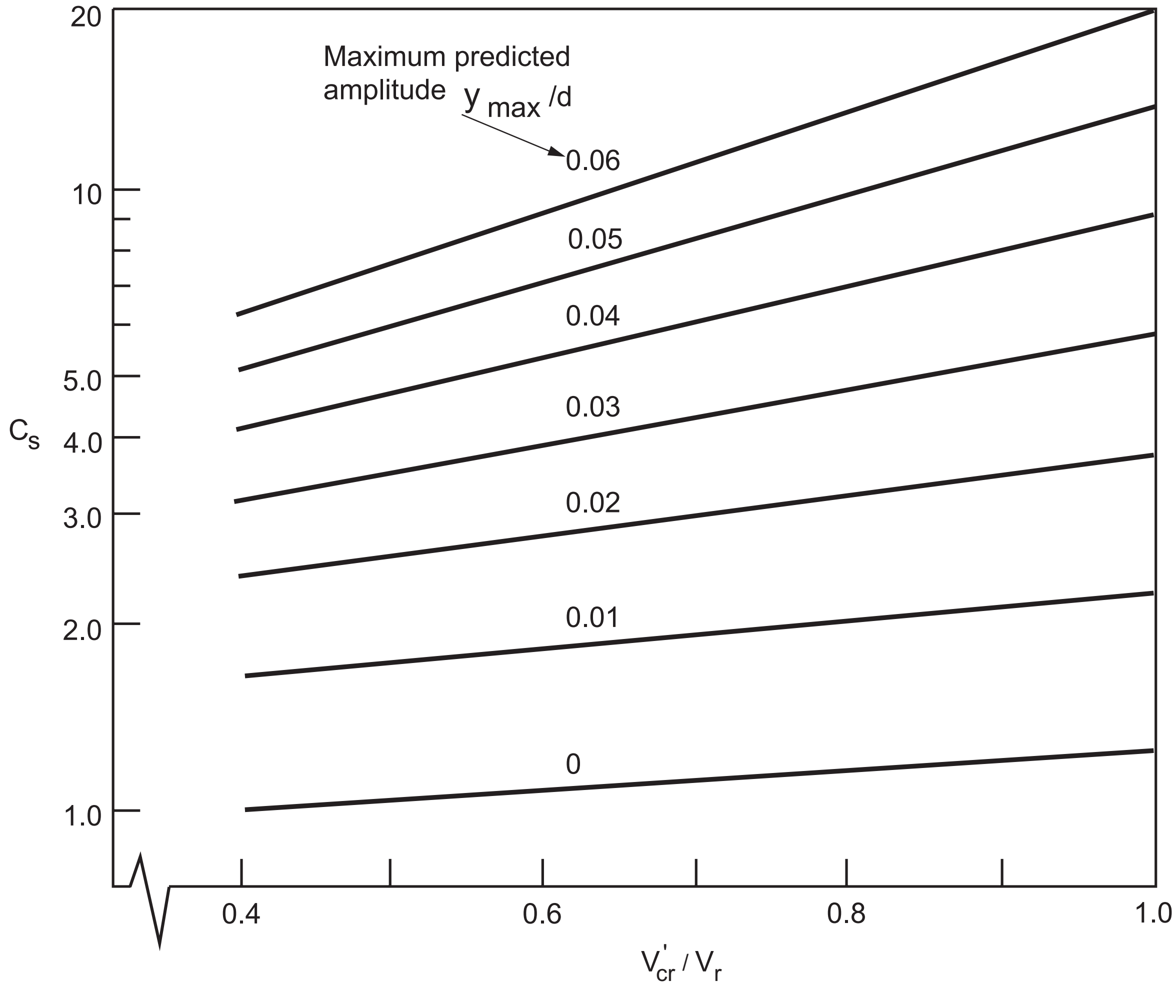


Fig.5 Speed range factor

6. WIND TUNNEL TESTING

Where a design is subject to wind tunnel testing, the models shall accurately simulate the external cross sectional details including non-structural fittings, e.g. parapets, and shall be provided with a representative range of natural frequencies, mass, stiffness parameters and damping appropriate to the various predicted modes of vibration of the bridge.

Due consideration shall be given to the influence of turbulence and to the effect of wind inclined to the horizontal, both appropriate to the site of the bridge. Tests in laminar flow may, however, be taken as providing conservative estimates of critical wind speeds and amplitudes caused by vortex shedding.

Where stability with respect to divergent amplitude response is established by section-model testing (see Annex C) stability shall be demonstrated up to the wind speed criterion V_{WO} (see 2.1.3.4) given by:

$$V_{WO} = \frac{1.10}{3} (V_r + 2V_d)K_{1A}$$

This shall be treated as a horizontal wind, or as inclined to the horizontal by an angle $\overline{\alpha}$ as a consequence of local topography. Although this occurs rarely for most locations in the United Kingdom, in cases where there are extensive slopes of the ground in a direction perpendicular to the span which suggest a significant effect on inclination of the mean flow, a separate topographical assessment (which may include wind tunnel studies) shall be made to determine $\overline{\alpha}$ Stability shall also be demonstrated in wind inclined to the horizontal by an angle α (in degrees) with speed criterion $V_{w\alpha}$ given by:

$$V_{w\alpha} = 1.10\, V_r K_{1A}$$

where

$$\alpha = 7 \left(\frac{S_g}{S_m} -1 \right) + \overline{\alpha}$$

S_g, S_m are derived from BD 37 (DMRB 1.3) for a loaded length equal to the longest span; and

K_{1A} is given in 2.1.3.4.

For full-model testing under the conditions given in Annex C, the criterion shall be wind speed V_{WE} given by:

$$V_{WE} = \frac{1.10}{2} (V_r + V_d) K_{1A}$$

The factor 1.10 in each of V_{WO}, $V_{W\alpha}$ and V_{WE} allows for the range of possible bridge span configurations and locations for which response is to be established. This factor may be reduced to a minimum of 1.00 for certain configuration/location combinations (typically spans greater than 500m at height above ground level less than above 30m in coastal or estuarial locations); such reductions shall only be adopted following further studies.

Further guidance on wind tunnel testing is given in Annex C.

7. REFERENCES

1. BS 5400: Steel, concrete and composite bridges:

Part 2: 1978: Specification for loads including Amendment No. 1, 31 March 1983, and Amendment agreed by BSI Committee; and

Part 10: 1980: Code of practice for fatigue.

2. Design Manual for Roads and Bridges:

Volume 1: Section 3: General Design:

BD 37 Loads for Highway Bridges (DMRB 1.3); and

BA 9 The Use of BS 5400: Part 10 - Fatigue (DMRB 1.3).

3. Bridge aerodynamics. Proceedings of Conference at the Institution of Civil Engineers, London 25-26 March, 1981. Thomas Telford Limited.

4. Partial safety factors for bridge aerodynamics and requirements for wind tunnel testing. Flint and Neill Partnership. TRRL Contractor Report 36, Transport Research Laboratory, Crowthorne, 1986.

5. A re-appraisal of certain aspects of the design rules for bridge aerodynamics. Flint and Neill Partnership. TRL Contractor Report 256, Transport Research Laboratory, Crowthorne, 1992.

6. ‘Wind tunnel tests on plate girder bridges’. Flint and Neill Partnership in association with BMT Fluid Mechanics Limited and TRL – 290/2/3/96, May 1996. (To be published by TRL in due course.)

7. Wind tunnel tests on box girder and plate girder bridges: Archived results: Library of Institution of Civil Engineers.

8. ENQUIRIES

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

Chief Highway Engineer The Highways Agency St Christopher House Southwark Street London SE1 0TE	J KERMAN Chief Highway Engineer

Chief Road Engineer Scottish Executive Development Department Victoria Quay Edinburgh EH6 6QQ	J HOWISON Chief Road Engineer

Chief Highway Engineer The National Assembly for Wales Cynulliad Cenedlaethol Cymru Crown Buildings Cathays Park Cardiff CF10 3NQ	J R REES Chief Highway Engineer

Director of Engineering Department for Regional Development Roads Service Clarence Court 10-18 Adelaide Street Belfast BT2 8GB	G W ALLISTER Director of Engineering

ANNEX A - PROXIMITY EFFECTS

A1. Introduction

Most obstacles in the path of the wind contribute to the creation of turbulence, either directly by vortex shedding or indirectly through the build-up of the profile of mean wind speed with height which in turn provides more severe velocity differentials when the flow is further perturbed. The basic turbulence is the statistically steady (or developing slowly over distance of many kilometres) summation of the effect of a broadly random scatter of such obstacles over a substantial region upwind of the reference point. Where there are identifiable outstanding obstacles, further specific allowance may be necessary.

The turbulence generated by such identifiable objects decays on translation downwind into a more random structure comprising a widening range of gust sizes (or spectral frequencies), eventually being subsumed into the basic random 'background'. There is thus a range of potential effects. Where there are obstacles (topographic or man-made) that are large compared with the cross-section of the bridge, wind tunnel tests can be used to check on the consequences of any change in turbulence affecting the bridge.

A parallel, or near-parallel, prismatic obstacle such as another bridge must always be given specific consideration, and should be included in any wind tunnel tests. However, where the gap is small compared to the characteristic dimension of the 'vortex street' (say, less than the structure depth) the formation and shedding or vortices becomes strongly linked. Assessments for small and moderate separation are given below.

A2. Twin-deck configurations

The term 'twin-deck bridge' is used here to describe a bridge with parallel decks each supported by the same structural form with equal structural depth, with gap between the decks not exceeding 1m, and the deck edges bordering the gap (or each gap) not differing in level by more than 250mm. The gap may be closed by an apron, or left open. The deck cross-falls may be in the same sense or reversed. Considerations relating to proximity effects for other parallel structures are given in A3 and A4.

A3. Evaluation of parameters for vortex shedding

For the evaluation of the critical wind speed for vortex excitation (2.1.1.2), the reference width b* should be determined according to Figure 1 applied to the overall cross-section ignoring the existence of the gap when the gap complies with the twin-deck configuration in A2 above. For all other provisions in this Annex, the evaluations should be based on the parameters for the upwind deck. Additionally, the prediction made of response amplitude y_{max} for vertical motion caused by vortex shedding (3.1.2) should be increased by a factor of 1.4 to conservatively allow for the interactive response of the twin-deck system.

Where the gap exceeds G_2 (see A4 below) each bridge deck may be treated separately with respect to vortex excitation. For gaps in the ranges of G_1 to G_2 (see A4 below), the estimate of the limiting response amplitude to vortex shedding, y_{max}, given in 3.1.2 should be doubled. For gaps between 1m and G_1 (see A4 below) for twin deck configurations and less than G_1 (see A4 below) for all other configurations, special investigations should be made to investigate the interactive vortex response of the dual system.

A4. Other proximity effects

Proximity effects in relation to turbulence should also be considered. The limiting value of P_T should be halved if there is a parallel structure with a clear gap G such that $G_1 < G < G_2$, where:

G_1 is the lesser of d′ or b′/3; and
G_2 is the greater of 24d′ or 6b′

in which d′ and b′ are the overall depth and breadth respectively of the neighbouring structure.

Where the gap is less than G_1 the parallel structures may be considered as a single structure for turbulence effects. Where the gap is greater than G_2, turbulence effects may be considered independently on each structure.

ANNEX B - FORMULAE FOR THE PREDICTION OF THE FUNDAMENTAL BENDING AND TORSIONAL FREQUENCIES OF BRIDGES

B1. General

To obtain accurate values of bending and torsional frequency it is recommended that dynamic analyses are undertaken to determine both fundamental and higher modes. Finite element methods or other recognised analytical procedures may be used.

For composite bridges, concrete should be assumed uncracked for simply-supported spans and cracked for continuous spans adjacent to internal supports.

Approximate formulae to obtain the fundamental bending and torsional frequencies for bridges within defined constraints are given below.

B2. Bending frequency

The fundamental bending frequency of a plate or box girder bridge may be approximately derived from:

$$f_B = \frac{K^2}{2\pi L^2}\sqrt{\frac{EI_b g}{w}}$$

where

L	=	length of the main span;
E	=	Young's Modulus;
g	=	gravitational acceleration;
I_b	=	second moment of area of the cross-section for vertical bending at mid-span; and
w	=	weight per unit length of the full cross-section at mid-span (for dead and super-imposed dead load).

Note: If the value of $\sqrt{I_b / w}$ at the support exceeds twice the value at mid-span, or is less than 80% of the mid-span value, then the formula should not be used except for obtaining very approximate values.

K is a factor depending on span arrangement defined below.

a) For single span bridges:

$K = \pi$ if simply supported;

or

$K = 3.9$ if propped cantilever;

or

$K = 4.7$ if encastre.

b) For two-span continuous bridges:

K is obtained from Figure B1, using the curve for two-span bridges, where

L_1 = length of the side span and $L > L_1$.

c) For three-span continuous bridges:

K is obtained from Figure B1 using the appropriate curve for three-span bridges, where

L_1 = length of the longest side span;

L_2 = length of the other side span and $L > L_1 > L_2$.

This also applies to three-span bridges with a cantilevered/suspended main span.

If $L_1 > L$ then K may be obtained from the curve for two-span bridges neglecting the shortest side span and treating the largest side span as the main span of an equivalent two-span bridge.

d) For symmetrical four-span continuous bridges (i.e. bridges symmetrical about the central support):

K may be obtained from the curve for two-span bridges in Figure B1 treating each half of the bridge as an equivalent two-span bridge.

e) For unsymmetrical four-span continuous bridges and bridges with greater than four continuous spans:

K may be obtained from Figure B1 using the appropriate curve for three-span bridges, choosing the main span as the greatest internal span.

Note on units:

Care should be taken when choosing the units for the parameters in the formula for f_B. Any consistent set may be used to give f_B in cycles per second (units: seconds^{-1}) but the following are recommended examples:

L	M	mm
I_b	m^4	mm^4
I_p	kgm^2	$kg\ mm^2$
I_j	kgm^2	$kg\ mm^2$
E	N/m^2	kN/mm^2
w	N/m	kN/mm
g	m/s^2	mm/s^2

B3. Torsional frequency

B3.1 Plate girder bridges

It may be assumed that the fundamental torsional frequency of plate girder bridges is equal to the fundamental bending frequency calculated from B2 above, provided the average longitudinal bending inertia per unit width is not less than 100 times the average transverse bending inertia per unit length.

B3.2 Box girder bridges

The fundamental torsional frequency of a box girder bridge may be approximately derived from:

$$f_T = f_B\sqrt{P_1(P_2 + P_3)}$$

where

$$P_1 = \frac{wb^2}{gI_p} \quad ;$$

$$P_2 = \frac{\Sigma r_j^2 I_j}{b^2 I_p} \quad ;$$

$$P_3 = \frac{L^2 \Sigma J_j}{2K^2 b^2 I_b (1 + \nu)}$$

f_B, w, I_b, L, g and K are as defined in B2 above;

b = total bridge width

I_p = polar moment of mass of cross-section at mid span (see NOTE 1);

ν = Poisson's ratio of girder material;

r_j = distance of individual box centre-line from centre-line of bridge;

I_j = second moment of mass of individual box for vertical bending at mid-span, including an associated effective width of deck;

J_j = torsion constant of individual box at mid-span (see NOTE 2);

Σ represents summation over all the box girders in the cross-section.

NOTES:

1) $$I_p = \frac{w_D b^2}{12g} + \sum\left(I_{pj} + \frac{w_j r_j^2}{g}\right)$$

where

w_D = weight per unit length of the deck only, at mid-span;

I_{pj} = polar moment of mass of individual box at mid-span;

w_j = weight per unit length of individual box only, at mid-span, without associated portion of deck.

2) $$J_j = \frac{4A_j^2}{\oint \frac{ds}{t}}$$ for a single closed cell

where

A_j = enclosed cell area at mid-span;

$\oint \frac{ds}{t}$ = integral around box perimeter of the ratio length/thickness for each portion of box wall at mid-span.

3) Slight loss of accuracy may occur if the proposed formula is applied to multi-box bridges whose plan aspect ratio (= span/width) exceeds 6.

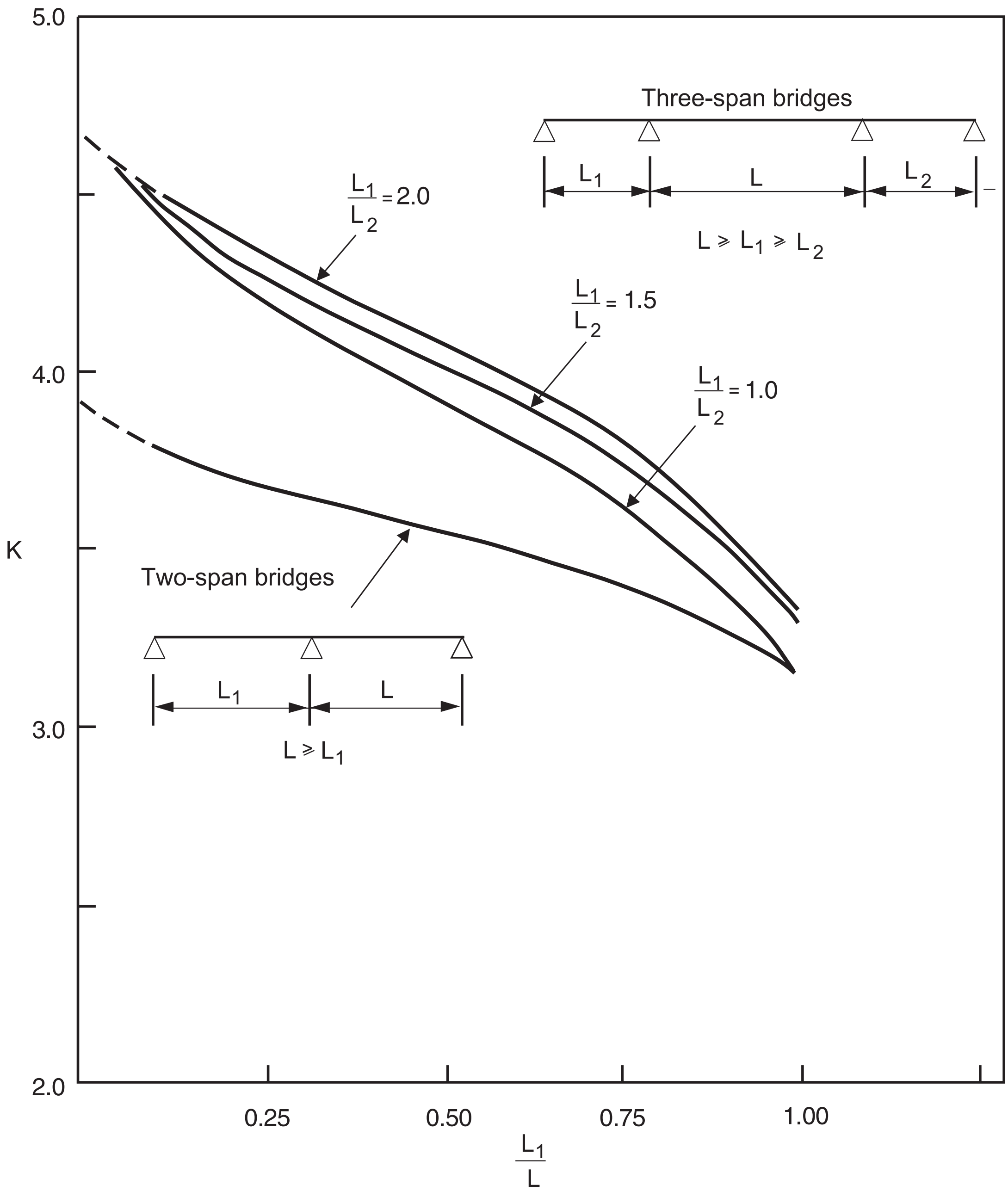


Figure B1 Factor K used for the derivation of fundamental bending frequency

ANNEX C - REQUIREMENTS FOR WIND TUNNEL TESTING

C1. Introduction

This Annex provides some guidelines to assist the engineer who intends to make use of wind tunnel model testing. These guidelines should not be regarded as complete as testing techniques are continually being developed. Other publications should be referred to for more extensive details of the theory and practice of wind tunnel testing.

In providing relatively comprehensive procedures it is recognised that sometimes it becomes necessary to relax modelling requirements in order to obtain practical information. It is important to stress the need for an awareness of the limitations of wind tunnel model tests in general with special caution in situations where partial or approximate models are used.

There are three basic reasons for undertaking wind tunnel tests:-

1) The first is to obtain static coefficients to be used in the basic static design checks for wind or for input to the analysis of turbulence response according to 3.3.

2) The second is to obtain coefficients for checks on vortex excitation effects or divergent amplitude effects according to 3.1 and 3.2 respectively. Such tests require dynamic models, and can also yield either direct estimation of turbulence response or 'derivative' coefficients which enable more sophisticated analysis of turbulence response to be carried out.

3) The third is to examine the influence of topography or other perturbations of the incident wind such as large structures or other obstacles nearby. A potentially important effect is inclination of the mean wind to the horizontal (quantity $\overline{\alpha}$ in 6).

For most studies in the first two categories it is necessary to use large scale models to accurately simulate the structure, deck furniture and, possibly, highway or railway traffic, and wind tunnels operating with uniform laminar flow (aeronautical wind tunnels) are used. More accurate measurements of mean loads require a simulation of the turbulence characteristics of wind, but this would require a model whose scale would be too small to be practicable. Smooth flow tests are thus generally acceptable for these measurements providing upper bound values to the coefficients when compared to those appropriate to the natural wind.

Studies in the third category require simulation of the salient properties of the wind. Wind tunnels designed to develop this type of flow are classified as boundary-layer wind tunnels (BLWT). The required small scale of the topography is such that a realistic model of the bridge itself would be impracticable.

Both types of tunnels use air at atmospheric pressure and operate in a low-speed range of 10-50 m/s.

If relevant, proximity effects need to be considered and adjacent structures modelled (see Annex A).

C2. Use of smooth flow (Laminar) tests to determine time average coefficients

Tests on sectional models of bridge decks can be used to determine the mean or static components of the overall wind load on the model. These wind loads can be obtained using rigid models with geometrically scaled features.

Accurate measurements of both the mean and the dynamic components of the overall loads can only be obtained if both the approach flow and the local environment are properly simulated. For the scale of model bridge required this becomes impracticable.

Approaches towards evaluating overall wind loads include the spatial averaging of instantaneous pressures acting on the elements of the bridge structure and the direct measurement of such loads with force balances or transducers capable of providing accurate information on both their mean and time-varying components. Sections comprising circular section members or other curved surfaces are likely to be Reynolds number (R_e) sensitive and adjustments based on full scale data and/or theoretical considerations may be necessary. Modelling adjustments are commonly needed for very small elements such as handrails to avoid local R_e effects below about 500.

The effect of wind inclination in elevation should be examined, the extent of which should be judged, depending on the site topography, any planned superelevation of the bridge and predicted torsional deflections under traffic loads. Generally tests up to ±5° are adequate.

C3. Section model tests to determine aerodynamic stability

The primary objective of such tests is to determine the aerodynamic stability of the bridge deck, mounted with deck furniture, using a geometrically scaled model of a section of the bridge elastically mounted in a wind tunnel. Typically, such models simulate the lowest bending and torsional vibration frequencies, and are tested in uniform laminar flow. The requirements of geometric scaling and Reynold's number limitations outlined in C2, still apply. In more advanced or refined stages, section models are tested in simulated turbulent flow in order to provide estimates of the responses at sub-critical wind speeds. As the simulated turbulence generally has a preponderance of the smaller-size eddies most likely to influence flow features such as vortex-shedding or re-attachment, the total intensity of turbulence should be selected with care. Generally this should be significantly lower than the standard atmospheric value for full scale. Reliance on beneficial effects from turbulence should not be allowed to reduce the likely aerodynamic effects.

In addition to modelling the geometry in accordance with C2, it is necessary to maintain a correct scaling of inertia forces, the time scale, the frequency, and the structural damping. The time scale is normally set indirectly by maintaining the equality of the model and full scale reduced velocities of particular modes of vibration. The reduced velocity is the ratio of a reference wind speed and the product of a characteristic length and the relevant frequency of vibration, see V_{Rg}, V_{Rf} in 2.1.3 for galloping and flutter. The numerical coefficient for vortex excitation in 2.1.1.2 is also derived from use of a similar ratio.

Measurements should be carried out through the range of wind speeds likely to occur at the site to provide information on both relatively common events, influencing serviceability, and relatively rare events, which govern ultimate strength behaviour. Wind inclination in elevation should be examined. Measurements of vortex excitation require careful control of the wind speed around the critical velocity, and care should be exercised if divergent amplitudes are predicted, to ensure that these do not become so violent as to destroy the model.

C4. Aeroelastic simulations of bridges

Ideally a dynamic model of the full bridge is used in the wind tunnel, commonly referred to as an aeroelastic model, to provide information on the overall wind induced mean and/or dynamic loads and responses of bridges. Such models are particularly valuable for slender, flexible and dynamically sensitive structures, where dynamic response effects may be significant. However to be representative, such tests must consistently model the salient characteristics of natural wind at the site and the aerodynamically significant features of the bridge's geometry. It is also necessary to correctly model the stiffness, mass and damping properties of the structural system. It is only possible to model the full spectrum of atmospheric turbulence in a wind tunnel at small scale; together with the obvious constraint of fitting a full bridge model within the tunnel, this is generally irreconcilable with the scale desirable to ensure correct behaviour, which is commonly sensitive to small changes in cross-section. For this reason the primary study should be made by section model tests; where non-uniformity of section or of incident flow conditions, complex dynamics or erection considerations, necessitate the use of a full model, particular care is needed in its design and interpretation of the test results.

As the modelling of dynamic properties requires the simulation of the inertia, stiffness and damping characteristics of only those modes of vibration which are susceptible to wind excitation, approximate or partial models of the structural system are often sufficiently accurate.

C5. Studies of the wind environment

C5.1 Topographic models

Information on the characteristics of the full scale wind may not be available in situations of complex topography and/or terrain. Small scale topographic models, with scales in the range of 1:2000, can be used in such situations to provide estimates of the subsequent modelling of the wind at a larger scale and are suitable for studying particular wind effects on the bridge.

C5.2 Local environment

Nearby buildings, structures, and topographic features of significant relative size influence the local wind flow and hence should be allowed for in simulations of wind at particular locations. For bridges in urban settings this requires the scaled reproduction (usually in block outline form) of all major buildings and structures within about 500 to 800m of the site. Also of particular importance is the inclusion of major nearby existing and projected buildings which could lead to aerodynamic interference effects, even though they may be outside this "proximity" model.

Corrections are generally required if the blockage of the wind tunnel test section by the model and its immediate surroundings exceeds about 5 to 10%. Typical geometric scales used in studies of overall wind effects or for local environment tests range between about 1:300 to 1:600.

C5.3 Use of boundary layer wind tunnels (BLWT)

A BLWT should be capable of developing flows representative of natural wind over different types of full-scale terrain. The most basic requirements are as follows:

a) To model the vertical distribution of the mean wind speed and the intensity of the longitudinal turbulence

b) To reproduce the entire atmospheric boundary layer thickness, or the atmospheric surface layer thickness, and integral scale of the longitudinal turbulence component to approximately the same scale as that of the modelled topography

In some situations a more complete simulation including the detailed modelling of the intensity of the vertical components of turbulence becomes necessary.

C6. Instrumentation

The instrumentation used in wind tunnel model tests of all aforementioned wind effects should be capable of providing adequate measures of the mean and, where necessary, the dynamic or time varying response over periods of time corresponding to about 1 hour in full scale. In the case of measurements of wind induced dynamic effects, overall wind loads and the response, the frequency response of the instrumentation system should be sufficiently high to permit meaningful measurements at all relevant frequencies, and avoid magnitude and phase distortions.

Furthermore, all measurements should be free of significant acoustic effects, electrical noise, mechanical vibration and spurious pressure fluctuations, including fluctuations of the ambient pressure within the wind tunnel caused by the operation of the fan, opening of doors and the action of atmospheric wind. Where necessary, corrections should be made for temperature drift.

Most current instrumentation systems are highly complex and include on-line data acquisition capabilities which in some situations are organised around a computer which also controls the test.

Nevertheless, in some situations it is still possible to provide useful information with more traditional techniques including smoke flow visualisation. Although difficult to perform in turbulent flow without proper photographic techniques, flow visualisation remains a valuable tool for evaluating the overall flow regime and, in some situations, on the potential presence of particular aerodynamic loading mechanisms.

C7. Quality assurance

The reliability of all wind tunnel data should be established and should include considerations of both the accuracy of the overall simulation and the accuracy and hence the repeatability of the measurements. Checks should be devised where possible to assure the reliability of the results. These should include basic checking routines of the instrumentation including its calibration, the repeatability of particular measurements and, where possible, comparisons with similar data obtained by different methods. For example, mean overall force and/or aeroelastic measurements can be compared with the integration of mean local pressures.

Ultimate comparisons and assurances of data quality can be made in situations where full-scale results are available. Such comparisons are not without difficulties as both the model and full-scale processes are stochastic. It is also valuable to make credibility crosschecks with the code requirements and previous experience.

C8. Interpretation of test data and prediction of full-scale behaviour

The objective of all wind tunnel simulations is to provide direct or indirect information on wind effects during particular wind conditions.

For time average effects this would relate to the appropriate design wind speed either with or in the absence of traffic as appropriate. Dynamic response will require prediction of the full-scale wind speeds at which vertical and/or torsional vortex excitation occurs as well as the speed at which divergent response is likely to start.

Particular care is required in relation to simulation and scaling such as, for example, with respect to wind speed, turbulence (intensity and length scales), frequency and damping (see C2, C3 and C5) as well as the bridge geometry and properties (see C4). The range of wind angles considered needs to take due account of the requirements in Chapter 6. If measurements have been undertaken in turbulent flow (see C3 and C4), the intensity of turbulence and associated length scales need to be reported for both the reduced and full size intensities and length scales.

C9. Typical scales

The following typical scales for the various types of wind tunnel tests are recommended:

TYPE OF TEST	TYPICAL SCALE
Topographic models	1:2000
Local environment	1:600 to 1.300
Aeroelastic models	1:200 to 1:100
Section models (stability or time average coefficients)	1:80 to 1:40
Models of ancillaries	> 1:20