NRA ADDENDUM TO

BD 49/93

DESIGN RULES FOR AERODYNAMIC EFFECTS ON BRIDGES

Standard BD 49/93 - Design Rules for Aerodynamic Effects on Bridges - is applicable in Ireland with the following amendments:

1. At several locations:

   For: "Overseeing Department"
   Read: "National Roads Authority";

   For: "highway"
   Read: "road".

2. Page 1/1, Paragraph 1.4:
   Delete Paragraph 1.4 and replace with:
   "1.4 This Standard should be used forthwith for all schemes for the construction and/or improvement of national roads. The Standard should be applied to the design of schemes already being prepared unless, in the opinion of the National Roads Authority, application would result in significant additional expense or delay progress. In such cases, Design Organisations should confirm the application of this Standard to particular schemes with the National Roads Authority."

3. Page 4/1, Section 4:
   Delete text and replace with:
   "4.1 All technical enquiries or comments on this Standard should be sent in writing to:

   Head of Project Management and Engineering
   National Roads Authority
   St Martin's House
   Waterloo Road
   Dublin 4"

Eoghan O'Connor

....

E O'CONNOR
Head of Project Management and Engineering

THE HIGHWAYS AGENCY BD 49/93

THE SCOTTISH OFFICE DEVELOPMENT DEPARTMENT

THE WELSH OFFICE
Y SWYDDFA GYMREIG

THE DEPARTMENT OF
THE ENVIRONMENT FOR NORTHERN IRELAND

Design Rules for Aerodynamic Effects on Bridges

Summary: This Standard sets out the design requirements for highway bridges and footbridges with respect to aerodynamic effects.

REGISTRATION OF AMENDMENTS

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

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

VOLUME 1	HIGHWAY STRUCTURES, APPROVAL PROCEDURES AND GENERAL DESIGN
SECTION 3	GENERAL DESIGN

PART 3

BD 49/93

DESIGN RULES FOR AERODYNAMIC EFFECTS ON BRIDGES

Contents

1. INTRODUCTION

General

1.1 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. All references to BS 5400: Part 2 are intended to imply the document as implemented by BD 37 (DMRB 1.3).

1.2 The requirements, in the form of design rules, are given 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. The original version of these rules first appeared as the "Proposed British Design Rules" in 1981 in reference (3). A modified version was included in the Transport and Road Research Laboratory Contractor Report 36 (4), which also contained the associated partial safety factors and guidance on the use of the rules. In the light of their use in bridge design in recent years, further consideration was deemed necessary with respect to a number of items, the more notable ones being 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 later modifications is available in TRL Contractor Report 256 (5). The present version of the rules is the outcome of this further study.

1.3 Guidance on the use of the design rules is available in TRRL Contractor Report 36 (4).

Implementation

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

2. REQUIREMENTS

Scope

2.1 This Standard is applicable to all highway bridges and foot/cycle-track bridges. However its provisions will affect only certain categories of bridges as explained in the rules.

Design requirements

2.2 The aerodynamic aspects of bridge design shall be carried out in accordance with the rules given in Annex A.

3. REFERENCES

1. Design Manual for Roads and Bridges

Volume 1: Section 3 General Design

BD 37/88 Loads for Highway Bridges
(DMRB 1.3)

2. BS 5400: Steel, concrete and composite bridges: Part 2: 1978: Specification for loads and Amendment No. 1, 31 March 1983.

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 and Road 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.

4. ENQUIRIES

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

Chief Highway Engineer The Department of Transport St Christopher House Southwark Street London SE1 OTE	T A ROCHESTER Chief Highway Engineer

The Deputy Chief Engineer, Roads Directorate The Scottish Office Industry Department New St. Andrew's House Edinburgh EH1 3TG	J INNES Deputy Chief Engineer

The Director of Highways Welsh Office Y Swyddfa Gymreig Government Buildings Ty Glas Road Llanishen Cardiff CF4 5PL	K J THOMAS Director of Highways

Chief Engineer - Roads Service Dept. of the Environment for Northern Ireland Commonwealth House Castle Street Belfast BT1 1GU	W J McCOUBREY Chief Engineer - Roads Service

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DESIGN RULES FOR BRIDGE AERODYNAMICS

Contents

Notation

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

b Overall width of bridge deck

b^* Effective width of bridge deck

C_g Parameter 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 $\pm 10°$ of normal to the longitudinal centre line of the bridge in strong winds

d_4 Depth of bridge deck

E Young's modulus

f Frequency or natural frequency

f_B Natural frequency in bending

f_T Natural frequency in torsion

g Acceleration due to gravity

h Height of bridge parapet or edge member above deck level

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

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

I_p Polar moment of inertia of the bridge cross section at mid-span

J_j Torsion constant of individual box at mid-span

K Factor used in the calculation of natural frequency

K_1 Wind coefficient, related to return period, as per BS 5400: Part 2

K_2	Wind coefficient, related to height of deck, as per BS 5400: Part 2
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_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
t	Thickness of box
V	Mean hourly wind speed
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_r	Reference wind speed
V_{Rf}	Reduced critical wind speed for classical flutter
V_{Rg}	Reduced critical wind speed for galloping and stall flutter
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
δ_s	Structural damping expresses as logarithmic decrement
γ_{fL}	Partial load factor
σ_r	Stress range
ϕ	Solidity ratio, or ratio of net total projected area presented to the wind to the total area encompassed by the outer boundaries of the deck
ρ	Density of air
ν	Poisson's ratio

1. General

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 Department. 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 may result in 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 may be of:

(1) Limited amplitudes which could cause unacceptable stresses or fatigue damage,

(2) Divergent amplitudes increasing rapidly to large values, which must be avoided,

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

1.1 Limited amplitude response

(i) Vortex-induced oscillations - oscillations of limited amplitude may be excited by the periodic cross-wind forces arising from the shedding of vortices alternatively from the upper and lower surfaces of the bridge deck. 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 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, the structure may be forced to oscillate.

1.2 Divergent amplitude response

Identifiable aerodynamic mechanisms leading to oscillations of this type 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.

(ii) Classical flutter - this involves coupling (ie, interaction) between the vertical bending and torsional oscillations.

1.3 Non-oscillatory divergence

Divergence can occur if the aerodynamic torsional stiffness (ie, 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.

2. Susceptibility to aerodynamic excitation

This section can 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 Bridges of span up to 200m

Bridges designed to carry the loadings specified in BS 5400: Part 2, built at normal heights above ground and of normal construction, in either of the following categories, may be considered to be subject to insignificant effects in respect of all forms of aerodynamic excitation:

a) Highway or railway bridges having no span greater than 50 m

b) Footbridges having no span greater than 30 m

Other bridges having no span greater than 200 m may be considered adequate with regard to each potential type of instability if they satisfy the relevant criteria given in 2.1.1, 2.1.2 and 2.1.3.

For the purpose of these rules, normal height may be considered to be less than 10 metres above ground level, and 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.

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 other than for certain truss girder bridges - see 2.1.1.3(a). The limiting criteria given in 2.1.1.3 should 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. It should either by determined by appropriate wind tunnel tests on suitable scale models or it may be calculated as follows for both vertical bending and torsional modes of vibration of box and plate girder bridges. For truss bridges with solidity $\phi < 0.5$, refer to 2.1.1.3(a). When $\phi \geq 0.5$ these equations can be used for vertical bending modes of vibration only - torsion modes should be the subject of special investigation, eg, appropriate wind tunnel tests.

$V_{cr} = 6.5 f d_4$ for $b^*/d_4 < 5.0$

$V_{cr} = f d_4 (1.1\, b^*/d_4 + 1.0)$ for $5.0 \leq b^*/d_4 < 10$

$V_{cr} = 12 f d_4$ for $b^*/d_4 \geq 10$

In these equations:

- b^* is the effective width in metres as defined in Figure 1,
- d_4 is the depth in metres shown in 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,
- f is either f_B or f_T as appropriate, ie, the natural frequencies in bending and torsion respectively (Hz) calculated under dead and superimposed dead load. A means of calculating approximate values of f_B and f_T, within certain constraints, is given in Annex B.

2.1.1.3 Limiting criteria

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

(a) Truss girder bridges may be considered stable with regard to vortex excited vibrations provided that $\phi < 0.5$, 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.

(b) All bridges, including truss bridges, may 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_r , where

$V_r = 1.25 K_1 K_2 V$,

- V is the mean hourly wind speed (see clause 5.3.2.1.1 of BS 5400: Part 2),
- K_1 is the wind coefficient related to return period (see clause 5.3.2.1.2 of BS 5400: Part 2),
- K_2 is the hourly speed factor, to adjust to deck level of the bridge (clause 5.3.2.2, Table 2 and modification where appropriate as in clause 5.3.2.1.5 of BS 5400: Part 2),

(c) Any bridge whose fundamental frequency is greater than 5Hz may be considered stable with respect to 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 that the fundamental frequencies in both bending and torsion, calculated in accordance with 2.1.1.2, are greater than 1 Hz, then the effects of turbulence may be ignored. If this condition is not satisfied the dynamic effects of turbulence response should 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 $b < 4d_4$.

V_g may be calculated from the reduced velocity, V_{Rg}, using the formula below, provided that the following limits are 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 $0.5d_4$.

(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 $0.25d_4$ for each 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 15% of the bridge span.

(iii) Any central median barrier shall have a shadow area in elevation per metre length less than 0.5m^2.

If these conditions are fulfilled, V_g can be obtained from

$$V_{Rg} = \frac{C_g\ (m\delta_s)}{\rho d_4^2} = \frac{V_g}{f_B\ d_4}$$

where

f_B is the natural frequency in Hz in vertical bending motion as defined in 2.1.1.2,

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 and 4 with side overhang less than or equal to $0.7d_4$,

m is the mass per unit length of the bridge (in kg/m),

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

ρ is the density of air (1.2 kg/m^3),

d_4 is the reference depth of the bridge in metres (see Figure 1) as defined in 2.1.1.2.

If the constraints (i) to (iii) above are not satisfied, wind tunnel tests should be undertaken to determine the value of V_g .

(b) Torsional motion

Torsional motion shall be considered for all bridge types. Provided that the fascia beams and parapets comply with the constraints given in (a) above, then V_g may be taken as:

$$V_g = 5\, f_T\, b$$

In addition for bridges of type 3, 3A, 4 and 4A (see Figure 1) having b < $4d_4$, V_g may be taken as the lesser of:

$$V_g = 12\, f_T\, d_4 \textit{ or } 5\, f_T\, b$$

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 in metres,

d_4 is as defined in (a) above,

2.1.3.3 Classical flutter

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

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

given by

$$V_{Rf} = 4\left(1 - \frac{f_B}{f_T}\right)\left(\frac{m\, r}{\rho\, b^3}\right)^{½}$$

but not less than 2.5

where f_T, f_B, m, ρ and b are defined in 2.1.3.2,

r is the polar radius of gyration of the effective bridge cross section at the centre of the main span in metres (polar second moment of mass/mass)^{½}

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

NOTE: In wind tunnel tests allowance must be made for the occurrence in practice of a value of the frequency ratio f_B/f_T which is less favourable than that predicted from the nominal mass and stiffness parameters of the structure. In general an increase of at least 0.05 to the nominal value of f_B/f_T should be allowed for, subject to a maximum value of

$$0.95 - \frac{0.5}{mr/\rho b^3}$$

2.1.3.4 Limiting criteria

Values of V_g and V_f derived in accordance with 2.1.3.2 and 2.1.3.3 respectively shall satisfy the following:

$V_g \nless 1.3\ V_r$
$V_f \nless 1.3\ V_r$

where V_r is the reference wind speed defined in 2.1.1.3(b). Where these criteria are not satisfied, the additional requirements outlined in 3.2 shall be followed.

2.1.4 Non-oscillatory divergence

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

2.2 Bridges of span greater than 200m

The stability of all bridges having any span greater than 200m shall be verified by means of wind tunnel tests on scale models in accordance with 6.

3. Additional requirements

If the bridge is found to be susceptible to aerodynamic excitation, then the following additional requirements shall be fulfilled.

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)

For bridges having no span greater than 200 m, the amplitudes of vibration y_{max} , in metres from mean to peak, for flexural and torsional modes 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) Edge and centre details conform with the constraints given in 2.1.3.2(a).

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

For vertical flexural vibrations,

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

For torsional vibrations,

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

This latter equation should be used with care for plate girder bridges until further wind tunnel results are available to verify the rule. Torsional vibrations of truss bridges should be the subject of special investigation, eg, appropriate wind tunnel tests.

For bridge types 1A, 3A, 4A, 5 and 6, and for bridge types 1, 3 and 4 during erection, (Figure 1) with no continuous solid overhang over more than 2/3 of the span, the amplitudes obtained from the above formulae shall be multiplied by a factor of 3.

In these equations,

b, m and ρ are as defined in 2.1.3.2,

r is as defined in 2.1.3.3,

δ_s is the logarithmic decrement due to structural damping.

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:

Material of construction	δ_s
Steel	0.03
Steel and Concrete Composite	0.04
Concrete	0.05
Timber	0.15
Aluminium Alloy	0.02

Alternatively, maximum amplitudes of all bridges may be determined by appropriate wind tunnel tests on suitable scale models.

The amplitudes so derived should 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.

In addition 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.

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 that the wind speed required to induce the onset of these instabilities is in excess of 1.3 V_r (see 2.1.1.3). It should 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 should be demonstrated by appropriate wind tunnel tests on suitable scaled models (see 6) that the critical wind speed, V_f, for classical flutter is greater that 1.3 V_r (see 2.1.1.3).

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 a mean hourly wind speed of V (see clause 5.3.2 of BS 5400: Part 2). These shall be used to assess the adequacy of the structure in accordance with 4.

4. Design values of aerodynamic effects

When vibrations are predicted to occur due to vortex excitation (see 3.1) and 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	For ultimate limit state	For serviceability limit state
(a) Erection	1.2	1.0
(b) Dead load, superimposed dead load and wind load	1.2	1.0
(c) Other appropriate combination 2 loads	1.2	1.0

The effects due to each of these aerodynamic wind effects taken separately shall be combined with the static wind load effects, appropriate to V_{cr} for the mode of vibration under consideration for vortex shedding.

5. Fatigue damage

All bridges which fail to satisfy the requirements of 2.1.1 shall be assessed for fatigue damage due to vortex excited vibration in accordance with 5.1 in addition to fatigue damage due to other load effects.

5.1 Vortex excitation causing fatigue

An estimate of the cumulative fatigue damage shall be made in accordance with BS 5400: Part 10 by considering the stress range and number of cycles specified below, for each mode in which V_{cr} is less than V_r

where V_{cr} is defined in 2.1.1.2
V_r 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 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}, may 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 $1.25 \leq b^*/d_4 < 10$

$V'_{cr} = 13.5\,fd_4$ for $b^*/d_4 \geq 10$

where b^*, f and d_4 are defined in 2.1.1.2.

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

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, eg parapets, and shall be provided with a representative range of natural frequencies 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. Further specific requirements for wind tunnel testing are given in Annex C.

<table>
<tr><td>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>Motion discomfort
Only for V_{cr} < 20 m/s</td></tr>
<tr><td></td><td>A

All bridges except those in B</td><td>B
Simply supported highway bridges and all concrete footbridges</td><td>All Bridges</td></tr>
<tr><td>100</td><td rowspan="2">α may be greater than 20%: Assess by analysis using derived y_{max}</td><td rowspan="4">α may be greater than 25%. Assess by analysis using derived y_{max}</td><td rowspan="3">Pedestrian discomfort possible
(See Note 2)</td></tr>
<tr><td>50</td></tr>
<tr><td>30</td><td rowspan="2">Assess by analysis using derived y_{max} or for simplicity use upper bound load, $\alpha = 0.4\ K_D$</td></tr>
<tr><td>20</td><td>Unpleasant</td></tr>
<tr><td>10</td><td rowspan="5">α is less than 4% an may be neglected</td><td rowspan="3">Assess by analysis using derived y_{max} or for simplicity use upper bound load, $\alpha = 2.5\ K_D$</td><td rowspan="3">Tolerable</td></tr>
<tr><td>5</td></tr>
<tr><td>3</td></tr>
<tr><td>2</td><td rowspan="2">α is less than 5% and may be neglected.</td><td>Acceptable</td></tr>
<tr><td>1</td><td>Only just Perceptible</td></tr>
</table>

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 unfactored design dead plus live load to be applied as the loading due to vortex excitation

Note 2: If K_D is greater than 30 mm/s^2 and the critical wind speed for excitation of the relevant mode is less than 20 m/s, detailed analysis should be carried out to evaluate K_D, If K_D is still found to be greater than 20 mm/s^2, pedestrian discomfort may be experienced and the design should be modified.

TABLE 1 ASSESSMENT OF VORTEX EXCITATION EFFECTS

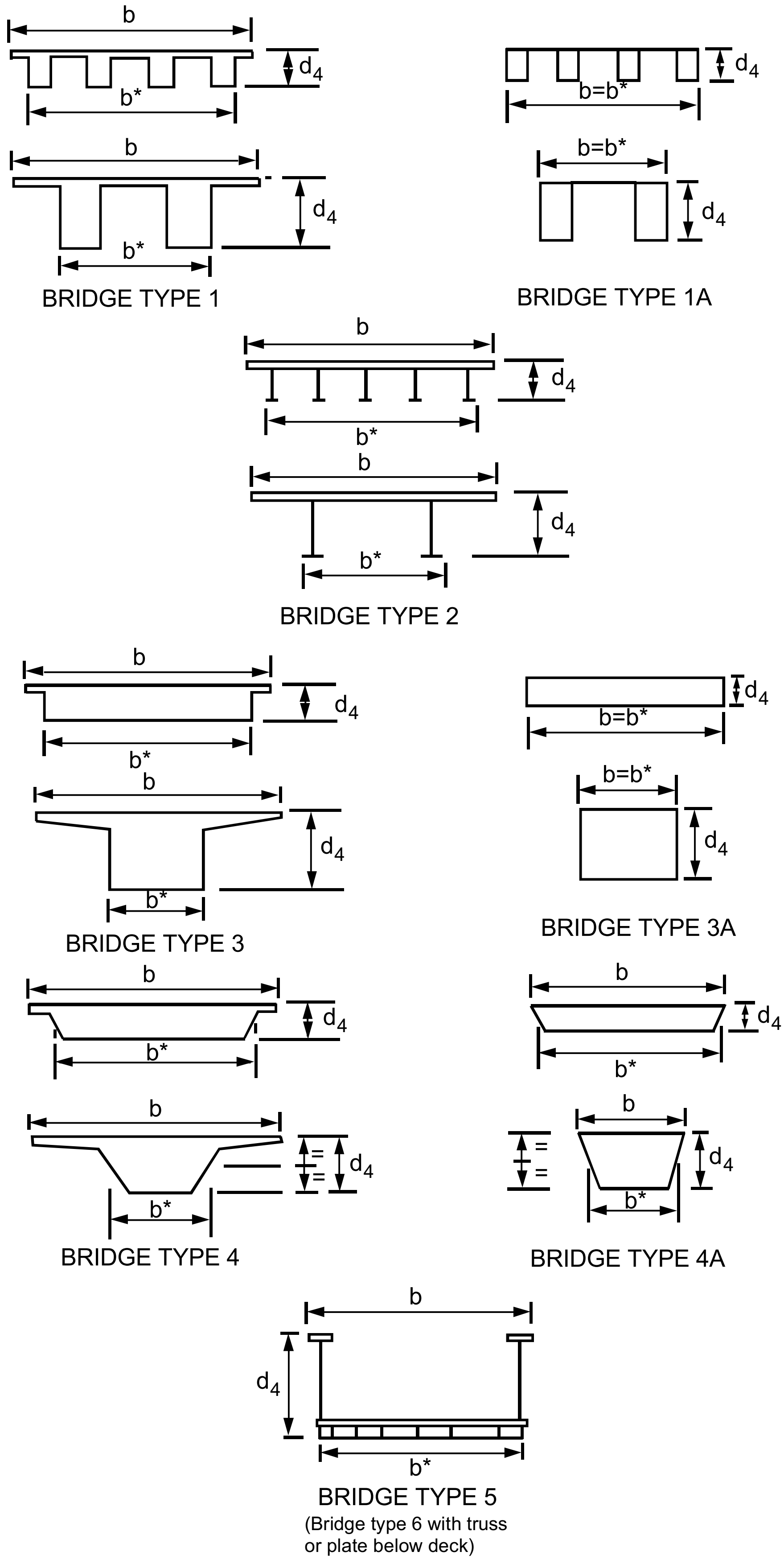

Fig.1 Bridge types and reference dimensions

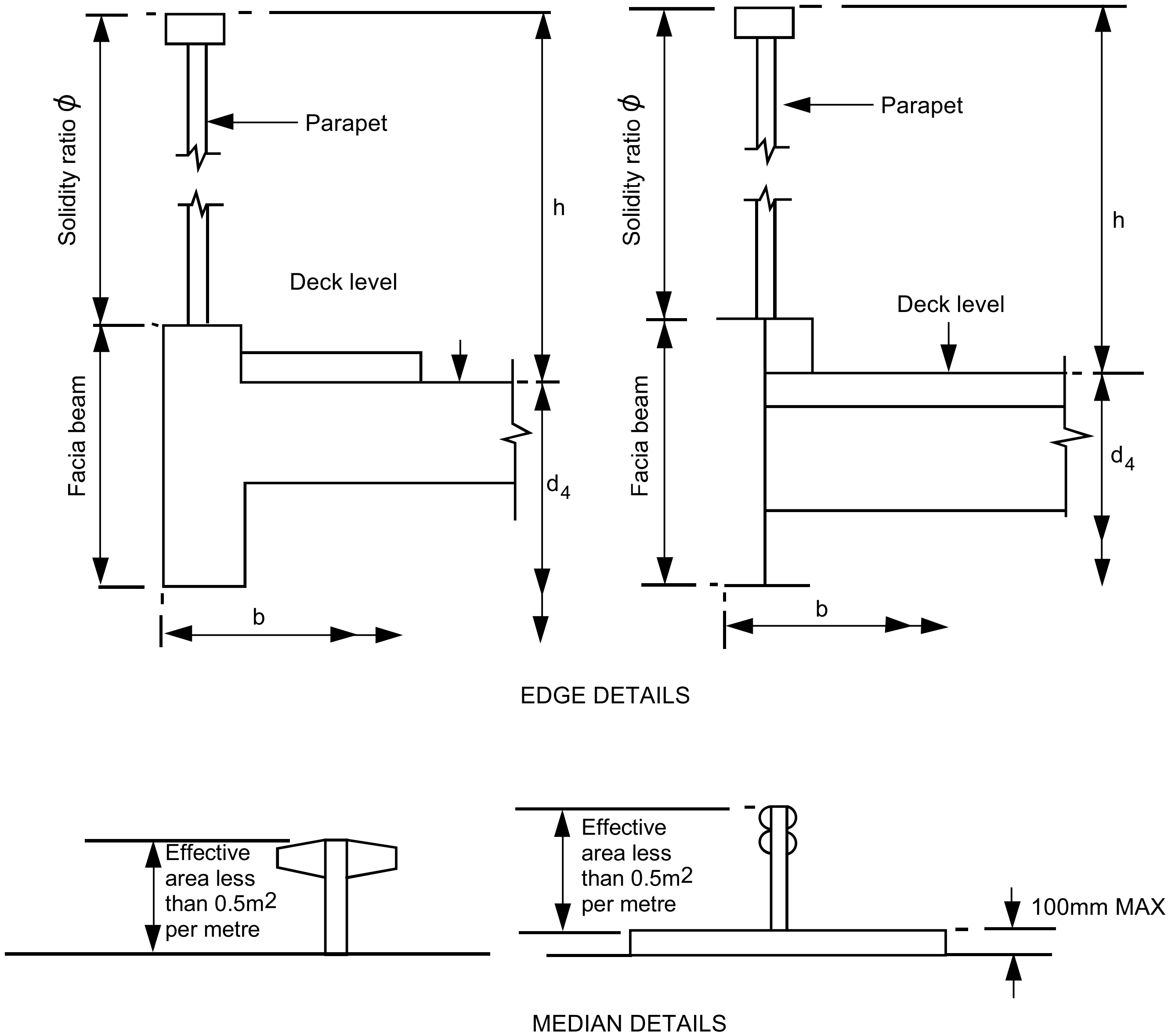

Fig.2 Bridge deck details

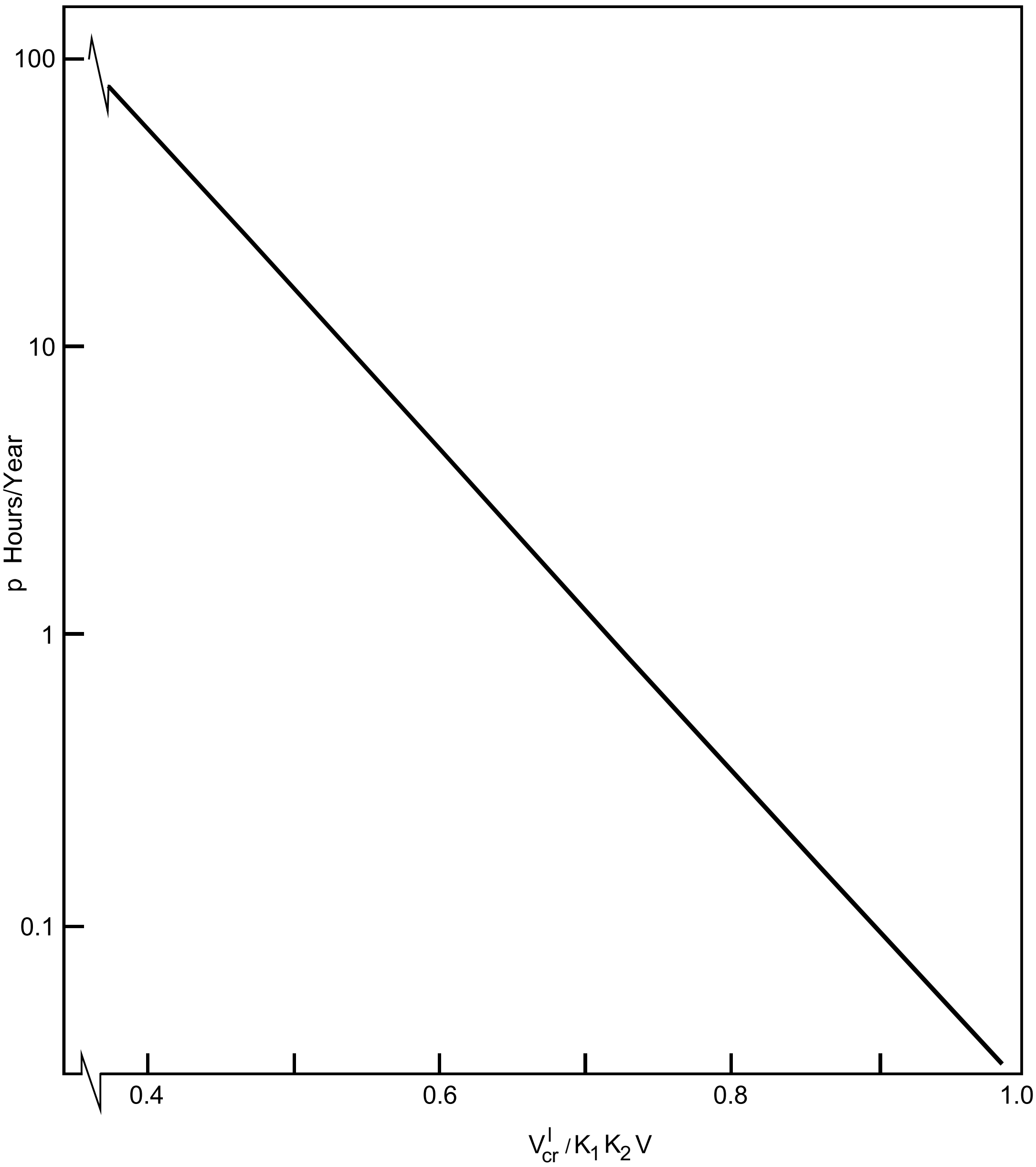

Fig.3 Expected frequency of occurrence of critical wind speed (Hours per annum of occurrence of speed within ± 2.5% of critical value)

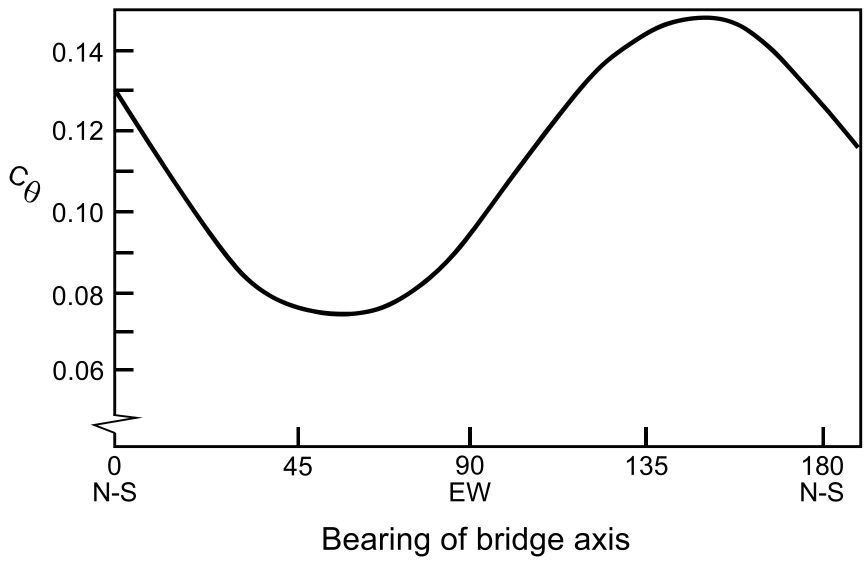

Fig. 4 Factor for orientation of bridge in plan

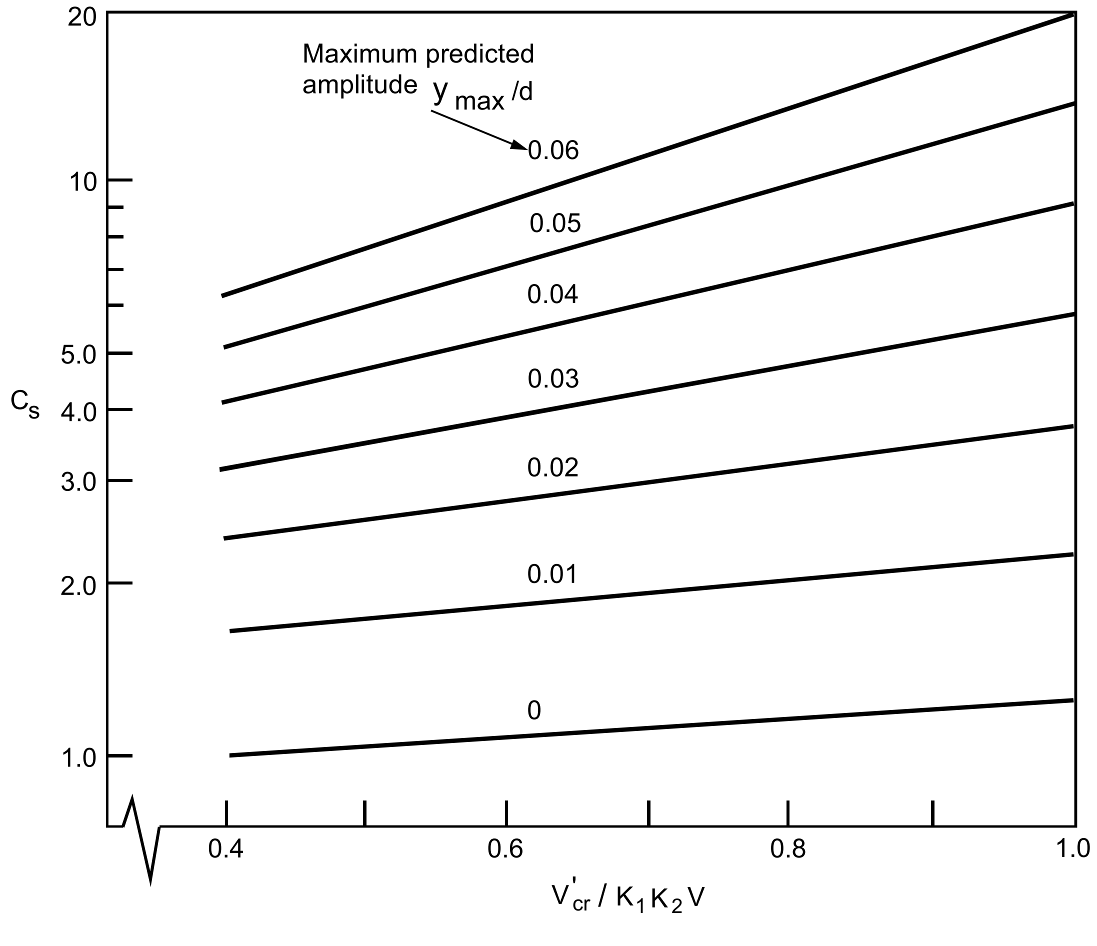

Fig.5 Speed range factor

FORMULAE FOR THE PREDICTION OF THE FUNDAMENTAL BENDING AND TORSIONAL FREQUENCIES OF BRIDGES

1. 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 shall 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.

2. 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,

w = weight per unit length of the full cross-section at mid-span (for dead and super load only).

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 A.1, 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 A.1 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 (ie bridges symmetrical about the central support):

K may be obtained from the curve for two-span bridges in Figure A.1 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 A.1 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	I_b	E	w	g
m	m^4	N/m^2	N/m	m/s^2
mm	mm^4	kN/mm^2	kN/mm	mm/s^2
ft	ft^4	lbf/ft^2	lbf/ft	ft/s^2

3. Torsional frequency

3.1 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 2. above, provided the average longitudinal bending inertia per unit width is not greater than 100 times the average transverse bending inertia per unit length.

3.2 Box girder bridges

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

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

where

$$P_1 = \frac{w\, b^2}{g\, I_p} \qquad P_2 = \frac{\sum r_j^2\, I_j}{b^2\, I_p}$$

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

f_B, w, I_b, L, g and K are as defined in 2. above,

b = total bridge width,

I_p = polar moment of inertia 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 area 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) $$I_p \approx \frac{w_D\, B^2}{12\, g} + \sum \left(I_{pj} + \frac{w_j\, r_j^2}{g} \right)$$

where

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

I_{pj} = polar moment of inertia 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) $$J_j = \frac{4\, A_j^2}{\oint ds/t} \quad \text{for a single closed cell}$$

where

A_j = enclosed cell area at mid-span,

$\oint ds/t$ = 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.

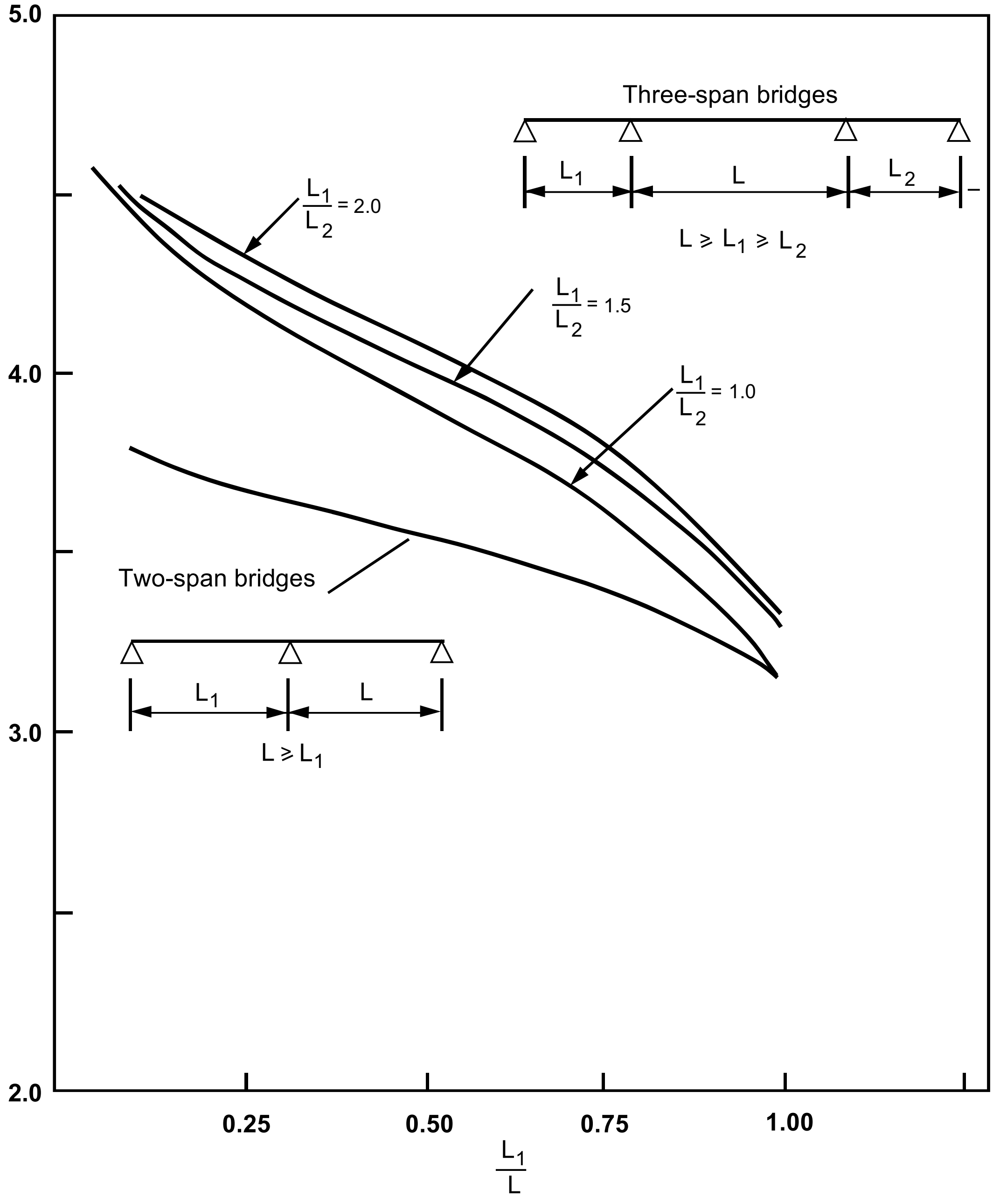

Fig.A.1 Factor K used for the derivation of fundamental bending frequency

REQUIREMENTS FOR WIND TUNNEL TESTING

1. 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. Many useful publications are available which give 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 two basic reasons for undertaking wind tunnel tests. The first is to obtain better information on the wind environment, covered in section 3 below and the second is to determine actual measurements of static and dynamic wind forces on models of the bridge, described in section 4 below.

2. Types of wind tunnel

Accurate estimates of the wind environment, required for the design of major bridges, can be obtained by topography in a wind tunnel capable of representatively simulating the characteristics of natural winds at the site. This includes the simulation of the salient properties of the "approach" wind. The influence of the immediate surroundings, including nearby building structures and significant topographic features, may need to be considered. 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.

Wind tunnel tests on bridges may be required to determine the time average drag, lift and twisting moments or aerodynamic coefficients of particular configurations which are not explicitly covered in this standard. For such tests 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 in the natural wind.

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

3. Use of boundary layer wind tunnels (BLWT)

3.1 Requirements

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 lateral and/or the vertical components of turbulence becomes necessary.

3.2 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 region of 1:2000, can be used in such situations to provide estimates of the subsequent modelling of the wind at a larger scale, suitable for studying particular wind effects on the bridge.

3.3 Local environment

Nearby buildings, structures, and topographic features of significant relative size influence the local wind flow and hence must 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 800 m 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.

4. 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 (see section 3 above). 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. For models consisting of circular section members which are Reynold's number (R_e) sensitive, adjustments based on full scale data and/or theoretical considerations may be necessary. Corrections are also needed for sharp-edge bodies if the local R_e value falls 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 $\pm 5^\circ$ are adequate.

5. Aeroelastic simulations of bridges

Ideally aeroelastic simulations, to provide information on the overall wind induced mean and/or dynamic loads and responses of bridges, would be 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. The scale required for such a model cannot normally be made compatible with the scale of turbulence achievable in the wind tunnel. Such tests, for whole models of bridges, are thus generally not feasible and recourse has to be made to section model tests.

As the modelling of dynamic properties requires the simulation of the inertial, 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.

6. 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 section 4, 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.

In addition to modelling the geometry in accordance with section 4, it is necessary to maintain a correct scaling of inertia forces, the time or 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.

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 must be exercised if divergent amplitudes are predicted, to ensure that these do not become so violent as to destroy the model.

7. 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 aeroelastic 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 experiment. 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.

8. Quality assurance

The reliability of all wind tunnel data must 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 cross-checks with the code requirements and previous experience.

9. 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.

10. 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