Background to the development of BD 49/01: Design rules for aerodynamic effects on bridges

Prepared for Highways Agency

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Executive Summary

The design rules for bridge aerodynamics were first published by the Institution of Civil Engineers in the Proceedings of a conference on Bridge Aerodynamics in 1981. These rules were developed in response to the need of designers to have simplified methods of determining when aerodynamic problems might be encountered in bridges and what steps can be taken at the design stage to avoid such problems. The rules were further developed through a series of projects, which eventually resulted in the publication in 1993 of standard BD 49/93 by the Highways Agency.

When BD 49/93 was published, it was recognised that certain aspects required further investigation and there were number of clauses which contained caveats, particularly those associated with plate girder bridges. As experience in the application of the Rules was obtained, these aspects were identified and further studies were undertaken. In particular there was a need to confirm and extend the details in certain clauses where the results were sensitive to edge beam depth and overhang.

To address these problems a special programme of tests on plate girders was commissioned by the Highways Agency. Other desk studies were also carried out to clarify the clauses and extend the scope of the Rules. The standard was thus revised and re-published as BD 49/01.

This report sets down the background to these studies and describes the revisions made to the updated aerodynamic rules. In particular, it describes the background to the development of the aerodynamic susceptibility factor, which replaces the simple limiting span criteria of the earlier versions of the rules. This is a more sophisticated way of determining when the simplified rules are applicable and allows them to be used in a much wider range of structures. This report also presents the background to the other revisions including:

- improved considerations of edge details;
- amendments to critical wind speeds in light of further wind tunnel testing;
- improved accuracy of vortex shedding amplitudes;
- more accurate criteria for aerodynamic effects;
- initial guidance on proximity effects;
- revised guidelines for wind tunnel testing.

Notation

The notation given here is consistent with the notation used in BD 49.

a_{max}	Maximum amplitude of vibration due to vortex excitation
b	Overall width of bridge deck
b'	Overall width of neighbouring bridge deck for twin bridges
b^*	Effective width of bridge deck
c	Amplitude correction factor for decks without overhangs
C_L	Lift coefficient
C_s	Coefficient to take account of the extent of wind speed range over which oscillation may occur
d_4	Depth of bridge deck
d'	Depth of neighbouring bridge deck for twin bridges
d_s	Structural damping value
f	Frequency or natural frequency
f_B	Natural frequency in bending
f_T	Natural frequency in torsion
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
k	Height of fascia or solid up-stand
K_{1A}	Probability coefficient
K_D	Dynamic sensitivity factor
K_R	Reduction factor for twin decks
K_S	Scruton number given by $2m\delta_s/\rho d_4^2$
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
P_b	Aerodynamic susceptibility parameter
P_T	Turbulence sensitivity check parameter
S_c	Fetch factor, as defined in BS 6399: Part 2
S_t	Turbulence factor, as defined in BS 6399: Part 2
V_{cr}	Critical wind speed for vortex shedding
V_g	Critical wind speed for galloping and stall flutter
V_r	Hourly mean wind speed
V_s	Site hourly mean wind speed (10m above ground) as per BD 37
V_{VS}	Reference wind speed for vortex shedding
V_{WE}	Wind speed criteria for section model testing of divergent amplitude response
V_{WO}	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	Vertical component of wind speed
y_{max}	Maximum amplitude of vibration of the deck
α	Inclination of wind to horizontal due to local topography
δ_s	Structural damping expressed as logarithmic decrement
σ_{flm}	Peak stress per unit deflection in the first mode of vibration
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

1 Introduction

The design rules for bridge aerodynamics (hereinafter referred to as the Rules) were first published by the Institution of Civil Engineers in the Proceedings of a conference on Bridge Aerodynamics in 1981 (ICE 1981). These Proceedings also contain a paper by Smith and Wyatt, which gives the background to the development of the rules up to that time. The partial safety factors for use with the design rules and the requirements for wind tunnel tests were published in a later report (Flint and Neill Partnership, 1986). A further investigation (Flint and Neill Partnership, 1992) modified and clarified certain clauses in the Rules and it is this that formed the basis of the Rules contained in Highways Agency Standard BD 49/93.

When BD 49/93 was published, it was recognised that certain aspects required further investigation and there were number of clauses which contained caveats, particularly those associated with plate girder bridges. As experience in the application of the Rules was obtained, these aspects were identified and further studies were undertaken. In particular there was a need to confirm and extend the details in certain clauses where the results were sensitive to edge beam depth and overhang.

To address these problems a special programme of tests on plate girders was undertaken (Daly and Smith 2002). Other desk studies were also carried out to clarify the clauses and extend the scope of the Rules. This report sets down the background to these studies and describes the revisions made to the updated Rules as presented in BD 49/01 (Highways Agency, 2001).

2 Background

BD 49/93 was based in part on the results of wind tunnel tests on models, which attempted to represent the very wide range of possible bridge parameters affecting aerodynamic response. The original wind tunnel tests were limited in their scope and BD 49/93 had to be carefully drafted to ensure that their application to all forms of bridges would result in safe designs without being unduly conservative. Subsequent experience in the use of BD 49/93 publication revealed that a re-assessment of certain aspects of the Rules could result in their application being extended to cover a wider range of practical bridges and, if possible, to remove some of the conservatism. This re-assessment is the subject of this report and the results are embodied in the revised version of BD 49/93, now issued as BD 49/01, which also incorporates other corrections and clarifications that should improve application.

Notwithstanding the revisions made, it remains true to say that the aerodynamic behaviour of bridges is complex and depends upon a large number of variables which have been simplified so that BD 49/01 can be used as an aid to design. There are, and always will be, bridge types and configurations which will fall outside the scope of specified design rules for aerodynamics. In such cases it is still essential to seek advice and/or to undertake model wind tunnel tests to determine the likely response of the proposed bridge. BD 49/01 attempts to categorise those bridges for which further investigations or wind tunnel testing are required, as well as those where aerodynamic effects should be negligible. Further clarification and additions have been added to improve guidance on these aspects and produce more realistic bridge sections by use of a sensitivity parameter, rather than just a span basis.

3 Additional wind tunnel tests on plate girder bridges

3.1 The models

The programme of wind tunnel tests was performed using a series of modular section models of plate girder decks representative of practical bridges. This resulted in the different models shown in Figure 1, which formed a family of deck sections with variable overall width but with fixed overhang. Parapets of various height and solidity were provided, as shown in Figure 2, and made interchangeable between the various deck models.

Various fascia beam configurations were tested, as shown in Figure 3. These additional tests investigate a range of fascia beam depths and overhangs varying from 0.25 to 1.0 times the depth of the beams. In addition, two new edge details were introduced which represented thickening of the slab at the edge, as this was found to be typical of some existing bridges. Details of all the edge beams tested are shown in Figure 3.

3.2 Results

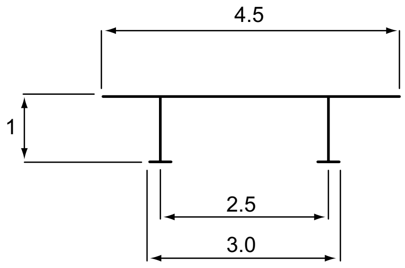
The wind tunnel testing typically included the measurement of vortex shedding response in both bending and torsion, and the determination of onset wind-speeds for divergent torsional instability for a range of damping values. The key results from the wind tunnel tests are given in Section 3.3 (review of results) and Sections 4.1 and 4.2 presents the implications in terms of revisions to the Rules. Full details of the testing, including the wind tunnels used, the section models, the wind tunnel test procedures, and the data obtained from the tests, are presented by Daly and Smith (2002).

3.3 Review of results

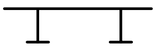
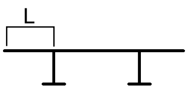
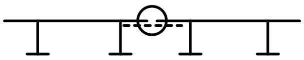
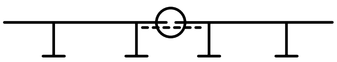
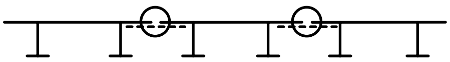
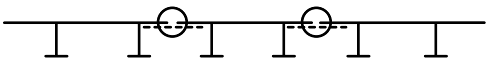
From a general review of the wind tunnel test results it was found that the amplitudes of vibration, surprisingly in the light of earlier studies, were more independent of the length of overhang than was previously thought. It was determined that an effective up-stand depth could be considered as:

$$k + \phi h$$

where	k	=	height of fascia beam or solid up-stand,
	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
and	h	=	height of bridge parapet or edge member above deck

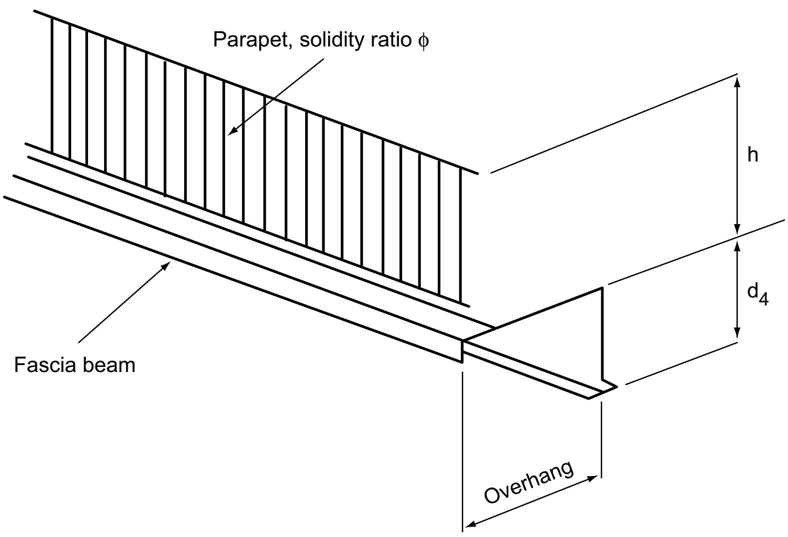


Basic model unit
dimensions are relative only

Model number	Configuration	Overhang $^{L}/_{d_4}$	Number of basic types	$^{b}/_{d_4}$	$^{b^*}/_{d_4}$
M1		1.0	1	4.5	3
M2		1.5	1	5.5	3
M3		1.0	2	9	7.5
M4		1.5	2	10	7.5
M5		1.0	3	13.5	12
M6		1.5	3	14.5	12

Joint in model deck formed by screwing through to stiff backing plate in shadow zone

Figure 1 Model configurations



Parapet type	$^{h}/_{d_4}$
P1	0.3
P2	0.7
P3	1.1
P4	1.5

Suffix	ϕ
a	0.25
b	0.50
c	0.75
d	1.00

Figure 2 Parapet details

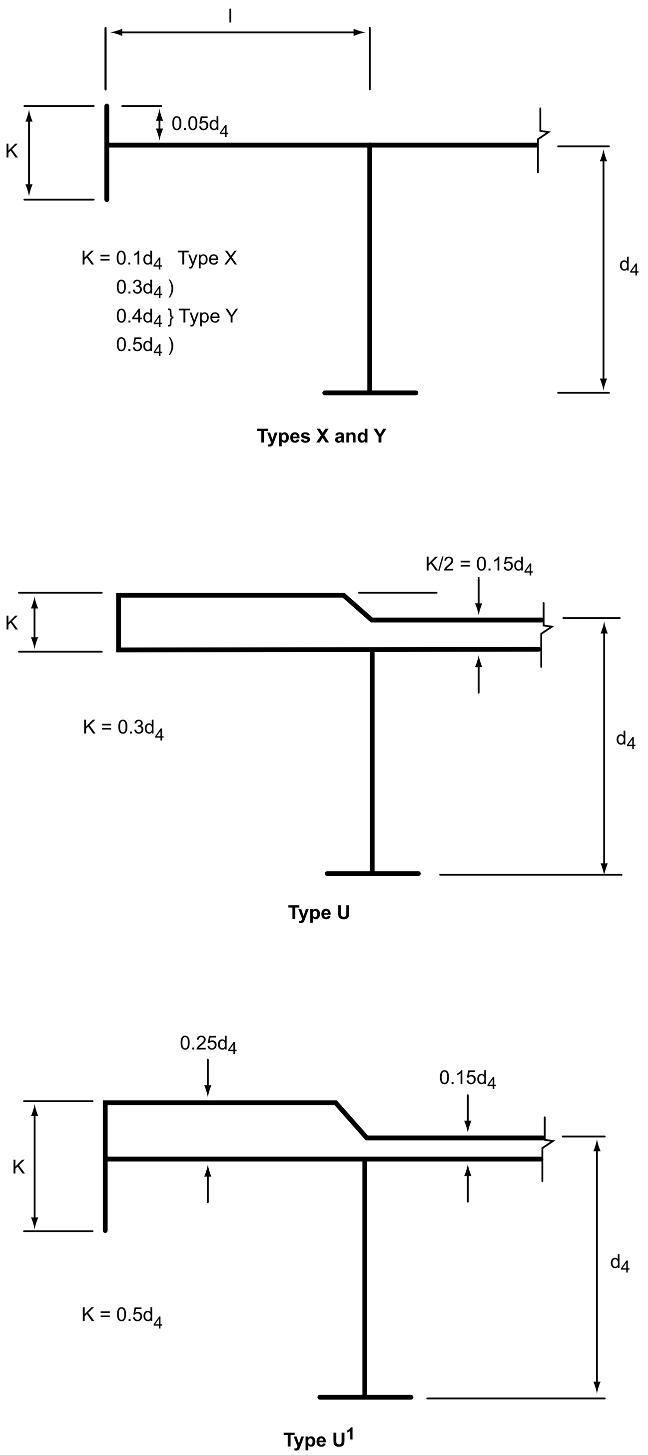


Figure 3 Leading edge details

The variation of effective up-stand depth to the depth of the bridge deck, d_4, tended to reflect the variation of amplitude from the tests and could be considered as a useful parameter for codification of maximum amplitudes.

All the results from the wind tunnel tests were tabulated and compared with the predictions from BD 49/93. These figures were then used to plot the more significant findings and to highlight where the comparisons were satisfactory and/or where BD 49/93 would need modification.

These results suggested that it may be prudent to decrease the prediction of the critical wind speed, V_{cr}, in bending and torsion for higher b^*/d_4 ratios for plate girders. This is discussed in Section 4.1.

The amplitude, y_{max}, from some tests exceeded the predicted amplitudes, particularly at the higher levels of damping. However, these results are generally for configurations that do not comply with the constraints set out in BD 49/93 for edge geometry. By omitting all those configurations that do not conform to these constraints, BD 49/93 applies more satisfactorily, particularly with the use of a factor reflecting the effective up-stand depth and improved criteria for edge geometry given in BD 49/01 (see Section 4.2).

Comparisons were also made with the earlier tests on box girder bridges which formed the basis of the BD 49/93. These showed that BD 49/01 generally predicted amplitudes between about 100% and 300% of the measured values in bending for those configurations meeting the modified geometric criteria and was also valid for torsion (see Section 4.2 below).

Some results for torsional amplitudes from certain test configurations showed values above those from BD 49/01. However, when those configurations that do not meet the criteria for edge geometry are omitted, BD 49/01 provides very conservative values of torsional amplitudes (see Section 4.2). Reference should be made to Daly and Smith (2002) for further particulars.

4 Amendments resulting from the additional tests

4.1 Vortex excitation - critical wind speeds

For plate girder bridges the critical wind speeds in Clause 2.1.1.2 of BD 49/01 are modified to:

$$V_{cr} = 6.5\ fd_4 \text{ for } b^*/d_4 < 5$$

$$V_{cr} = fd_4\ (0.7\ b^*/d_4 + 3.0) \text{ for } 5 \le b^*/d_4 < 10$$

$$V_{cr} = 10fd_4 \text{ for } b^*/d_4 \ge 10$$

where f is the natural frequency, either f_B, for bending, or f_T for torsion. These changes in V_{cr} are shown in Figures 4 and 5 which also include some test data for comparison.

It should be noted that these revised equations apply only to plate girder (and truss type) bridges. The original equations are retained for other bridge types.

When V_{cr} is low (less than 10 m/s), this tends to correlate to small amplitudes and low frequencies at which the assumed levels of damping may not be generated. Hence, for such cases, note 1 to Clause 3.1.2 of BD 49/01 factors down the structural damping value, δ_s, to ensure safe estimates are produced for amplitudes. The rule developed is arbitrary but safe and will often only apply to non-governing cases, ie, generally a higher mode with higher V_{cr} will govern.

4.2 Vortex excitation - amplitudes

The test results showed that BD 49/01 provides upper bound predictions of amplitude for configurations whose edge deck geometry complies with the constraints of

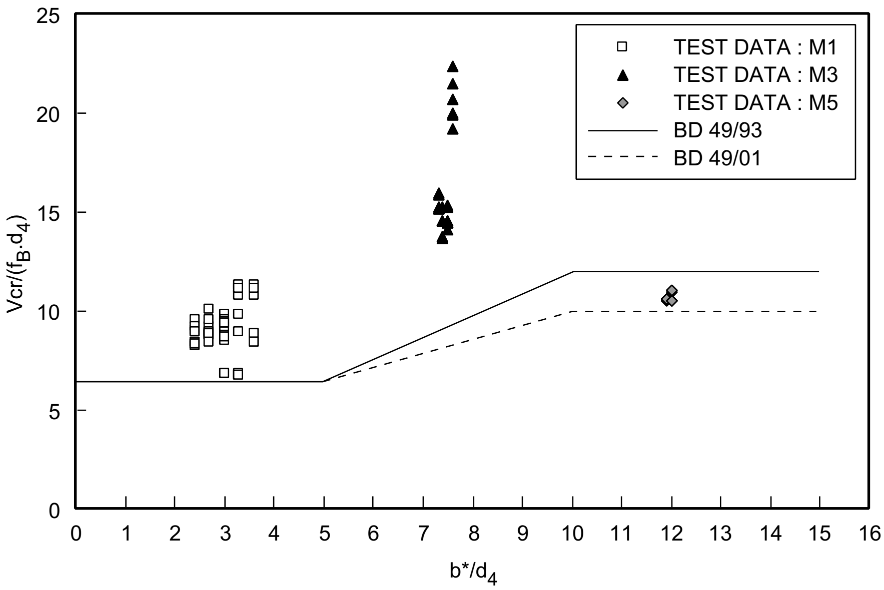


Figure 4a Critical wind speeds for vortex excitation (bending): Fascia beam X: (k/d_4 = .1)

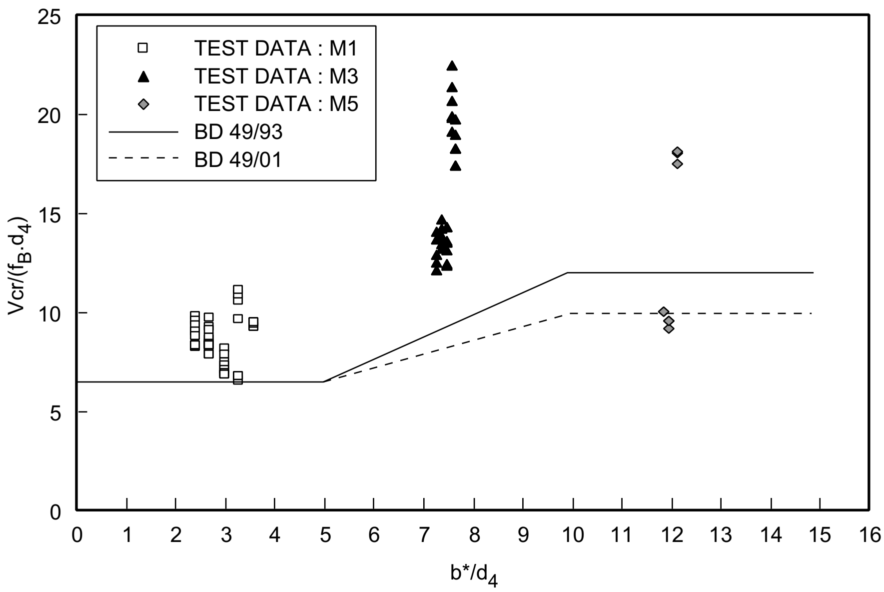


Figure 4b Critical wind speeds for vortex excitation (bending): Fascia beam Y: (k/d_4 = .2)

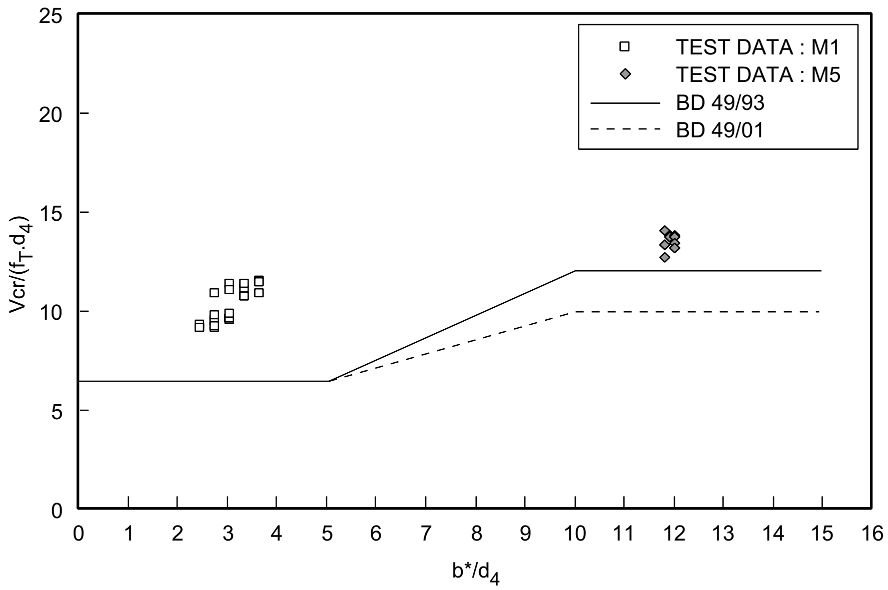


Figure 5a Critical wind speeds for vortex excitation (torsion): Fascia beam X: (k/d_4 = .1)

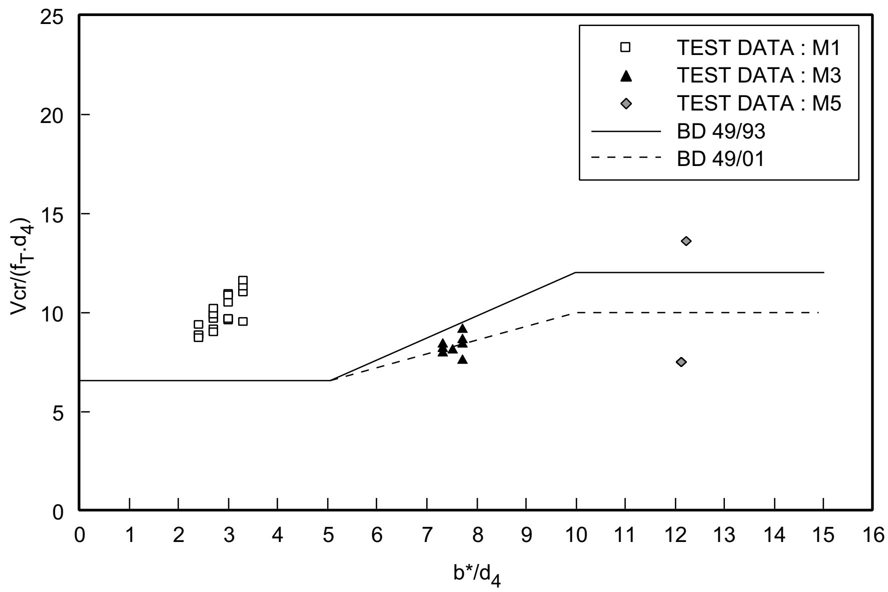


Figure 5b Critical wind speeds for vortex excitation (torsion): Fascia beam Y: (k/d_4 = .2)

BD 49/93. However it was not feasible to relax these restraints as large amplitudes were recorded for some configurations outside the constraints, although no consistent pattern emerged.

The only possible relaxation for plate girders was to increase the limit of the product $h\phi$ [original Clause 2.1.3.2(a)(ii)] from $0.25d_4$ to $0.35d_4$.

Use of BD 49/93 has also shown that the restriction of edge beam depth to less than $0.2d_4$ [original Clause 2.1.3.2(a)(i)] was leading to anomalies in design. For example, a 300mm deep slab on a 1m deep section was acceptable provided there was no up-stand at the edge. However if the slab was thickened at the edge, to say 320mm, then the design would not meet this criteria, having to be treated as an 'edge beam'. Hence, for the purposes of defining such members, edge stiffening of the slab to, say, half the slab depth could be ignored.

The amplitudes in BD 49/93 [Clause 3.1.2] need to be factored by 3.0 for decks without overhangs but no guidance is given concerning the minimum overhang which would qualify to escape this threefold multiplication of the response. For galloping behaviour the threshold for more severe behaviour was set when the overhang is less than $0.7d$ and the results of the tests were used to assess whether this limitation should be adopted for vortex excitation. However, the results of the tests with varying overhang and fascia beam depths, which showed the variation of amplitude with 'effective depth' as noted above, led to the consideration of a modification of BD 49/93, which would incorporate both this effective depth parameter, and the above factor of 3.0.

The result of this study is that in Clause 3.1.2 of BD 49/01 a new factor, c, can be applied to the predicted amplitudes for all sections, given by:

$$c = 3(k + h\phi) / d_4$$

where k = depth of fascia beam, or edge slab;

d_4 = reference depth of the bridge;

h = height of bridge parapet or other edge member above deck level;

ϕ = solidity ratio of parapet.

These parameters are defined in Figures 2 and 3, with h, k and d_4 in consistent units.

This eliminates the necessity of the fixed factor of 3.0 as was contained in BD 49/93, and allows dense and potentially even solid barriers to come within the scope of BD 49/01. However, as the tests did not comprehensively cover wind inclinations of up to ±5°, which on previous studies had been shown to be critical, solid barriers are still excluded.

The validity of this change is shown in Figure 6 for bending, and Figure 7 for torsion for all tests undertaken on both box girder and plate girder bridges which cover the range of damping considered appropriate for steel or composite structures at the time of the tests. More details on damping in the test arrangements are reported elsewhere (ICE 1981, Daly and Smith 2002).

In Figure 6 the measured amplitude is plotted against the predicted amplitude incorporating the new factor c. The rule is satisfied for all tests that lie below the 45° line shown on the graph. It can be seen that the rule is thus generally satisfied, with outliers being either sections with solid parapets (which are not covered by BD 49/01), or where the factor c less than 1 would safely ensure the rule was satisfied.

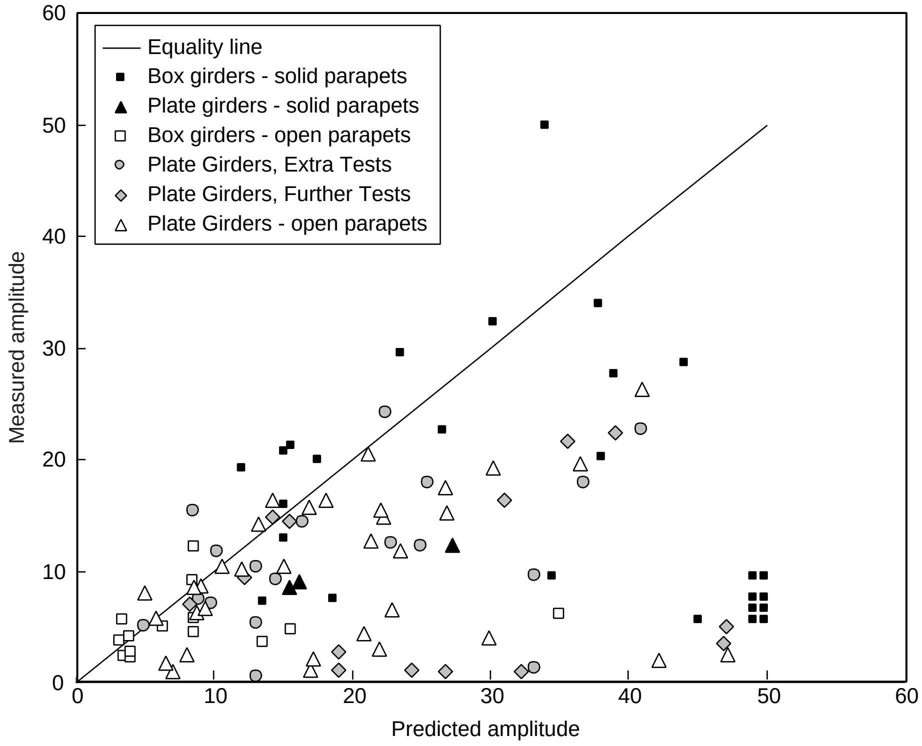


Figure 6 Use of c factor for bending

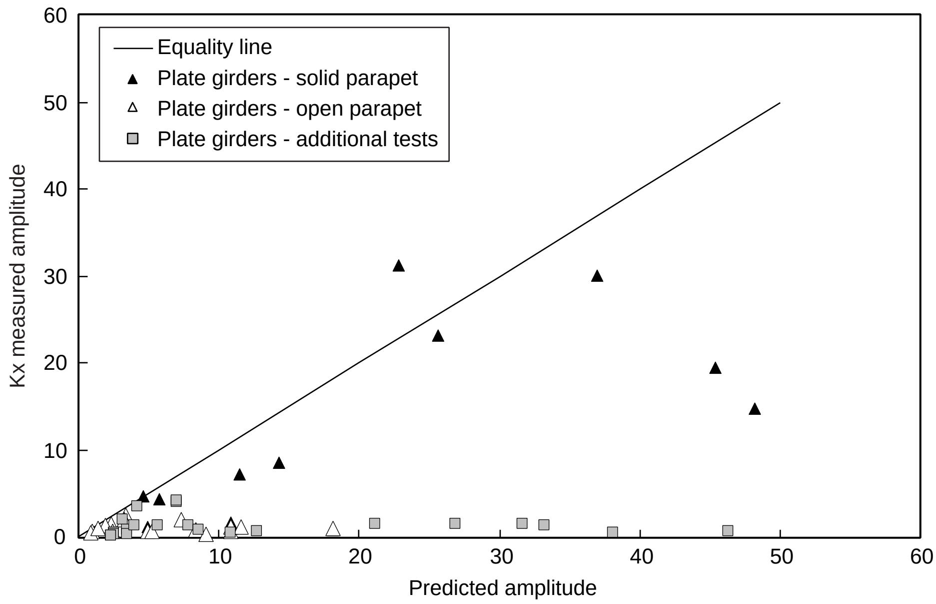


Figure 7 Use of *c* factor for torsion

Figure 8 shows the results for both bending and torsion. The response is given non-dimensionally (the ratio of the maximum measured value to the predicted value) for the damping range δ_s = 0.02 to 0.05 as a function of factor *c*. It can be seen that the use of the factor *c* provides a good envelope to the majority of the results.

The range of the factor *c* for the configurations used in the plate girder tests, which reflects the full range that could be encountered, is shown in Table 1. It can be seen that the initial limiting value of *c* less than unity would be obtained for five configurations in the matrix of tests. A lower value of 0.5 still appeared to provide a satisfactory factor and this was finally used as a lower bound. Also the lower values of *c* encountered in deeper longer span highway bridges, where values of 0.75 to 1.5 may typically be encountered, will produce lower estimates of amplitudes for such bridges. Experience to date suggests that this may be more realistic, since the amplitude now derived would be reduced to 25% and 50% respectively of those from the fixed factor of 3.0 given in BD 49/93. Table 1 also shows that with 'dense' or relatively high parapets, in relation to the bridge or deck depth d_4 (such as may be encountered on footbridges) the factor can exceed 3.0, highlighting the problems of amplitude magnification that can occur with such bridges, as noted below.

The note in Clause 3.1.2 of BD 49/93 that advocated caution when the clause was used with plate girder bridges has been removed as earlier plate girder results have been re-reviewed and further plate girders tested. However, the equations for y_{max} have been generalised to cater for the

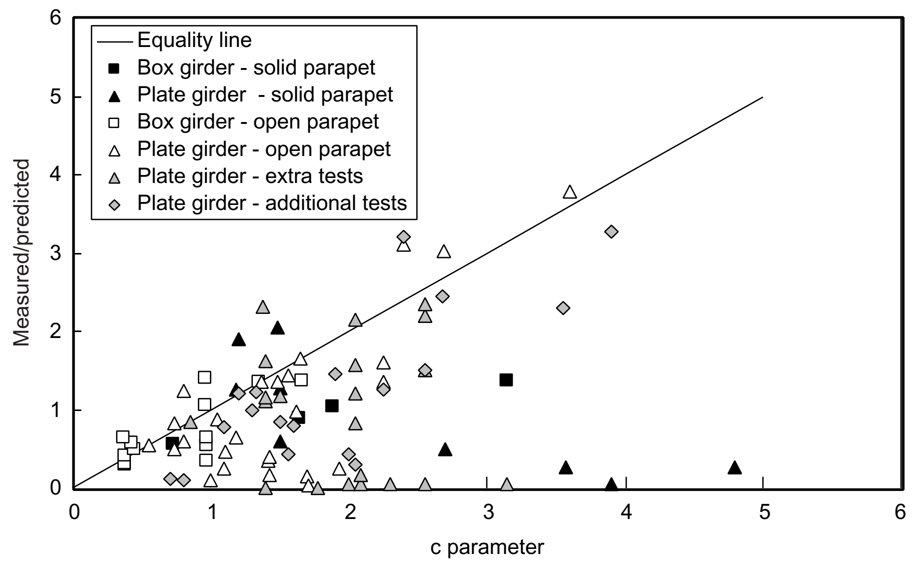


Figure 8 Non-dimensional plot of *c* factor

Table 1 Factor *c* for various configurations of edge details and barriers used in plate girder tests

	Barrier solidity	Parapet P-type	$h\phi/d_4$	k/d_4 [see Figure 3] 0.1 or 'X'	0.2 or 'Y'	U - type or 0.3 Y type	0.4 Y type	U' - type or 0.5 Y type
Any overall width of bridge	'Open' barriers $\phi \leq 1/2$	1a	0.075	0.525	0.825	1.125	1.425	1.725
		1b	0.150	0.750	1.050	1.350	1.650	1.950
		2a	0.175	0.825	1.125	1.425	1.725	2.025
		2b	0.350	1.350	1.650	1.950	2.250	2.550
		3a	0.275	1.125	1.425	1.725	2.025	2.325
		3b	0.550	1.950	2.250	2.550	2.850	3.150
		4a	0.375	1.425	1.725	2.025	2.325	2.625
		4b	0.750	2.550	2.850	3.150	3.450	3.750
Any overall width of bridge	'Dense' barriers $\phi > 1/2$	1c	0.225	0.975	1.275	1.575	1.875	2.175
		1d	0.300	1.200	1.500	1.800	2.100	2.400
		2c	0.525	1.875	2.175	2.475	2.775	3.075
		2d	0.700	2.400	2.700	3.000	3.300	3.600
		3c	0.825	2.775	3.075	3.375	3.675	3.975
		3d	1.100	3.600	3.900	4.200	4.500	4.800
		4c	1.125	3.675	3.975	4.275	4.575	4.875
		4d	1.500	4.800	5.100	5.400	5.700	6.000
See Figure 1		See Figure 2		Values of *c* factor				

$h\phi/d_4$ may be taken as $h\phi/d_4 + a\phi_b/b_4$ in cases where there are barriers of height a as well as parapets of height h.

Shows where factor is > 3.0 , where tests may be more appropriate, and may reap appreciable benefits when the factors are high. Note: the majority of these are with 'dense' barriers, which should be less common in practice.

mark-up of the '*c*' factor (previously 3.0) which applies to all bridge types and when high, enables the user to consider whether further studies (ie, wind tunnel tests) may be of possible benefit.

For torsion, BD 49/93 tends to provide a non-conservative prediction of the critical wind speed. Whilst it is likely that this is being partly catered for by the need to calculate the wind speed for classical flutter, Clause 2.1.3.2(b), V_g is modified as follows:

V_g = 3.3 $f_T b$, for plate girder bridges

V_g = 5.0 $f_T b$, for all other bridges

It was also significant that the torsional amplitudes for configurations that comply with the relevant geometry constraints were almost negligible and thus the torsional response due to vortex shedding for such plate girder bridges may be ignored: see Clause 3.1.2 of BD 49/01.

4.3 Edge geometry

The constraints on edge geometry, which are required for consideration of galloping and stall flutter (Clause 2.1.3.2) and for derivation of amplitudes under vortex excitation effects (Clause 3.1), have been moved to a separate section (Clause 2.3 of BD 49/01). This is to improve the readability of the document and to ease the cross-referencing from Clauses 2.1.3.2 (where the original limits were given) and 3.1. The use of a separate clause and the use of an introductory reference also helps to emphasise that there are still geometric constraints on the applicability of BD 49/01, although now slightly eased from those in BD 49/93 as discussed above in Section 4.2.

5 Other recent developments in bridge aerodynamics

5.1 General

Since the publication of BD 49/93, there has been considerable additional research in bridge aerodynamics as well as developments in bridge configurations themselves. Much of this has led to longer, lighter span bridges, both by the use of cable support configurations and advanced composite materials as well as in innovative designs, particularly for footbridges. Much of this work has been covered to a greater or lesser degree in various conferences and symposiums since 1993 including those at Deauville (AFPC, 1994), Hong Kong (HKIE, 1995), Boston (ASCE, 1995), Copenhagen (AA Balkema 1998), Malmo (IABSE, 1999), Hyderabad (IIBE, 1999) and the UK Wind Engineering Society Conference in Bristol (PF Consultants 1998).

Certain problems in the interpretation of BD 49/93 have also arisen, and it is likely that bridges have been designed beyond the original intended scope of BD 49/93. Similarly, strict interpretation of the Rules may have led to the need for additional wind tunnel tests that may have been unnecessary. Conversely it is known that there have been designs which were predicted to respond significantly and for which additional wind tunnel testing would have been advisable: the criteria for judging whether BD 49/93 is applicable or whether wind tunnel tests should be undertaken needed to be addressed.

These matters have been reviewed in this study in order to widen the scope of the Rules and provide a more general and less simplistic categorisation of the range of structures to which they are applicable. Some background is given in the following sections. Other aspects are also covered by Wyatt (1998).

5.2 Aerodynamic susceptibility parameter

5.2.1 Background to the susceptibility parameter

The current categorisation limits in Clause 2.1 of BD 49/93 are set down in terms of span length only. Whilst this has the benefit of simplicity, the aerodynamic response often depends as much on other parameters such as mass and stiffness as on span length.

To address this, the aerodynamic susceptibility parameter, P_b, is defined to replace the span limits in Clause 2.1 of BD 49/93:

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

where ρ = density of air

b = overall width of the bridge

m = mass per unit length of the bridge

V_r = hourly mean wind speed

L = length of the relevant maximum span of the bridge

and f_B = natural frequency in bending.

This parameter allows for the contribution of other parameters such as mass, frequency, continuity, bridge section and material as well as general wind speed criteria, rather than being dependent on span only. The parameter is intended to test the sensitivity of the bridge deck to potential problems such as vortex shedding, strong-excitation aerodynamic instabilities including galloping and flutter, and response to incident turbulence. The latter must consider the 'standard' free-stream natural wind, but may also require allowance for turbulence created by specific nearby objects (proximity effects). The criteria to be satisfied comprise subjective perception of motion (comfort) as well as safety in terms of both 'first passage' and fatigue strength assessments. The parameter P_b is therefore broadly drawn, and its validation relies significantly on calibration.

The form of P_b bears clear resemblance to the turbulence-sensitivity check parameter P_T (see Section 5.3 of this report and Clause 2.1.2 of BD 49/01). The latter, however, has sole focus on stress at maximum wind speed. The stress, σ_{flm}, defined as the flexural stress per unit deflection in the first mode of vibration, for a given structural form will be proportional to d_4/L^2. However, for vortex shedding and other instability problems, $f_B b/V_r$ is a normalisation relevant to phenomena primarily related to the cross-section, including most of the instability problems. The 'chordwise' admittance is also required for gust action and thus $f_B L/V_r$ is relevant to the correlation of any inputs along the length of the bridge, primarily for gust action but also for vortex shedding. Hence the equivalent of b^2 in the denominator of P_T is bL in P_b and is thus comparable to the norm for the slenderness d/L. Thus the parameter P_b has been presented in a form matching the conventional non-dimensional grouping of the input quantities, ie, $m/\rho b^2$ is a normalised mass, and $f_B b/V_r$ and $f_B L/V_r$ are normalisations of the vertical bending frequency discussed above. The parameter P_b takes the reciprocal of these quantities, so that an increased value corresponds to increased susceptibility to dynamic response. The numerical factor 16 is an arbitrary scaling to bring the upper threshold (ie, for applicability of the quantified check formulae) to unity.

The vertical bending frequency has been taken as the basis for frequency, on the presumption that the torsional frequency will be higher. Indeed, for a torsionally-stiff (box) structure, the torsional frequency will commonly be much higher. The resulting quadratic dependence on V_r makes this initial rating considerably more onerous for high level estuarial bridges than low-level bridges at inland locations. It may be desirable to review this sensitivity in the light of further calibration and experience, as noted in Sections 5.2.3, 5.2.4 and 8.1 of this report.

Although the turbulence response is dominated by a resonant effect, which (for random turbulence excitation) is inversely proportional to the square root of the total effective damping, the total damping includes a significant aerodynamic damping effect that further reduces the sensitivity. The torsional instability effects (including flutter) are also insensitive to damping. On the other hand, vortex shedding response, which may be more sensitive to damping, can be considered via the discomfort aspect, as discussed in Section 5.2.3. This aspect may not be so well fitted by P_b, and it remains to be confirmed that the expression includes sufficient conservatism in this respect. The fatigue aspect of vortex shedding has better protection, improved by the effective inclusion in P_b of the square of the ratio of the site reference wind speed V_r to the *critical* wind speed V_{cr}', since V_{cr}' is proportional to f_B. Thus when V_{cr}'/V_r is low, giving a high cycle count, the evaluated value of P_b will be high.

5.2.2 Bridges requiring check of aerodynamic stability

BD 49/01 includes checks for all the mechanisms of dynamic excitation by wind action that have been recognised as constituting a problem with respect to the global response of the structure. These require consideration of the fundamental (or at least low-order) mode(s) of vibration characterised by predominantly vertical or predominantly torsional motion of the structure.

It should be noted that this does not cover potential problems affecting individual elements such as cables or slender bracing elements in trusses. As the prescribed global checks are broadly conservative, there is no clear-cut reason to impose an upper limit of span for their applicability. The need to advance to more refined procedures (commonly involving wind tunnel testing) emerges from the difficulty of meeting the conservative default checks. It is, however, desirable to exclude structures having dynamic properties outside the range of current empirical experience.

The crucial dynamic properties are mass, natural frequencies and damping, together with the geometric dimensions. Since the rules have to be used as the delineator of applicability, guidance is provided in notes 1 to 3 to Clause 2.1 of BD 49/01 on the evaluation of these parameters. These are particularly relevant when required to determine where bridges may be considered as not being susceptible to aerodynamic effects. Hence, the scope of application of BD 49/01 will be more accurately reflected, resulting in an increase in the number of bridges being categorised as being not susceptible to aerodynamic effects. The revised document should also decrease the number of 'normal' bridges requiring wind tunnel tests: see also Sections 8 and 9 of this report.

The difficulty of the robust prediction of damping (in conjunction with differences in sensitivity of the various instability problems) is such that it has not been found possible to include it as a separate variable in the susceptibility check. As discussed in 5.2.1, this does not generally diminish the general use of P_b.

The likely range of the parameters is indicated in note 3 to Clause 2.1 of BD 49/01. Taking the unfavourable selection from these values, ie:

V_r = 40 m/s

m/b = 600 kg/m^2

and f_B = 50L$^{0.87}$

slightly reduces the upper limit of applicability in terms of span to L = 185m for this extreme case. This is only marginally more onerous than the blanket 200m upper limit of applicability in BD 49/93. However, V_r = 40 m/s is only applicable in the United Kingdom for extreme cases of high-level estuarial bridges. For example (using BD 37 terminology) with V_b = 24 m/s, S_p = 1.05 and S_d = 1.0, at height 50m, K_F = 1.0 and S'_c = 1.47, and presuming no significant topographic effect is likely in such a case, V_r = 37 m/s and the upper limit becomes of the order of 200m as given in BD 49/93. Furthermore, for such spans, such a low mass is not practicable except in the rare instances of orthotropic steel decks, in which case the natural frequency will be higher than the above. For example, the Cleddau Bridge (main span 213m) with orthotropic deck has m/b = 700 kg/m^2 and f_B = 56/L$^{0.87}$, and thus BD 49/01 would be applicable with P_b of 0.7. Indeed, with V_r = 37 m/s, BD 49/01 would be applicable to a bridge of this type up to the practical span limit for steel box girder construction, increasing the applicable span range considerably above the limit of 200m given in BD 49/93.

The same is more emphatically true for concrete and composite bridges. For all current structural forms in use in low-level inland United Kingdom locations, with m/b greater than 900 kg/m^2, f_B >50/L$^{0.87}$, V_r < 37 m/s, BD 49/01 would be applicable to spans up to 400m.

The height limit of 10m above ground level has been removed, as this has been found to exclude many bridges that could be encompassed in BD 49/01. However, although the applicability can be extended to virtually any height, a warning about abnormal wind effects from unusual terrain or topography has been added.

5.2.3 Bridges not requiring aerodynamic check

The parameter P_b is used to define a waiver, where the detailed checks are not required. This is applicable when P_b < 0.04. Discussion of potentially catastrophic oscillation ('divergent amplitude response' BD 49/93 Clause 2.1.3) in recent years has been directed towards long-span structures, although the classic early examples such as the Brighton Chain Pier should not be forgotten. Initial attention is therefore directed to vortex shedding. The lower range of applicability to be catered for by the susceptibility parameter needs to take account of the formulation for prediction of response amplitude. This is given in Clause 3.1.2 of BD 49/01 and is directly derived from the classic non-dimensional form for deterministic harmonic excitation, ie:

$$\frac{y_{\max}}{d_4} = \frac{C_L V_R^2}{4\pi K_S}$$

where C_L is the coefficient of fluctuating lift and V_R is the reduced velocity for resonance (V/fd), both being based on reference dimension d. The parameter K_S is the Scruton number given by $2m\delta_s/\rho d^2$, the normalisation of the structural damping. For Clause 3.1.2 of BD 49/01, C_L is presumed to be larger for more relatively slender cross-sections, in proportion to $(b/d)^{1/2}$. However, when this is converted to show response accelerations ($cf\,K_D$ in Clause 3.1.3 of BD 49/01, and noting that K_D is expressed in mm/s^2), the prediction is

$$K_D = 1.6\times10^6 c\left(\frac{d_4}{b}\right)^{0.5}\left(\frac{d_4}{L}\right)^2 \frac{C_f^2}{\left(\frac{m\delta_s}{\rho b}\right)}$$

in which C_f (dimensions m/s) is an arbitrary parameter chosen such that with

$$C_f = f_B\left(\frac{L}{80}\right)$$

The approximate lower-bound relationship suggested in note 3 to Clause 2.1 of BD 49/01 (ie, $f_B = 50/L^{0.87}$) corresponds closely to $C_f = 1.0$ over the span range where the 'waiver' is likely to be applicable ($C_f = 0.92$ at $L = 20m$ rising to $C_f = 1.14$ at $L = 100m$). Correspondingly, the upper limit $f_B = 100/L^{0.87}$ corresponds to $C_f = 1.85$ at $L = 20m$ rising to 2.27 at $L = 100m$. The above approximation to K_D shows that this does not generally diminish as the span is reduced, but emphasises the sensitivity to the ratio d/L . Decreasing span coupled with high values of d/L can give a K_D prediction that could be of concern in relation to the guidance in Table 1 of BD 49/01, due to the increase in natural frequency. However, the associated increase in the critical wind speed, V_{cr}, has been shown to increase such that at the chosen level of $P_b = 0.04$ it is above the reference wind speed V_{VS} and can be considered stable in accordance with Clause 2.1.1.3(a) or (b) of BD 49/01. The factor c in the K_D prediction above is typically not far from unity (specified minimum 0.5). Values of 0.75 to 1.5 may typically be encountered in the majority of the usual range of bridges for which the waiver to the rules should be applicable (although higher figures are possible in some cases). The discussion on amplitudes from vortex shedding in Clauses 4.2 and 5.7.1, as well as the discussion relating the factor c in Section 4.2, Table 1 and Figure 8 of this report are both also relevant here.

The waiver level has thus been adopted on the basis that at $P_b = 0.04$ detailed checks are not required since the resulting effects are either not significant, or the bridge can be deemed to be stable. These considerations have been confirmed by limited trials so far but further calibration is needed. The waiver is expected to apply to many relatively modest structures, and replaces the simple span threshold of $L = 50m$ (for highway bridges) in BD 49/93. The new waiver will generally be more generous (ie, spans greater than 50m being exempt) when V_r is low and m/b is high, but may be more onerous when V_r is high and m/b is low, not forgetting the influence of the other parameters.

5.2.4 Bridges where the waiver may not be applicable

The application of parameter P_b as a waiver should be further reviewed if any of the following apply:

i if V_r is outside the range of 20 to 40 m/s (ie, the range in note 3);

ii if m/b is outside the range of 600 to 1200 kg/m^2 (ie, the indicative range in note 3);

iii if the indicative range for f_B in note 3 in terms of span L does not apply, such as in some cases where the deck is cable or rod supported.

Such occurrences should be rare and are the reasons for the general comments on b, L, f_B and V_r in Clause 2.1 of BD 49/01 distinguishing between normal and novel structures.

It should also be noted that the amplitude correction factor, c, given in Clause 3.1.2 of BD 49/01 can be readily evaluated for any configuration/edge detail. Any value in excess of 3.0 may also be indicative of non-applicability for the reasons discussed in Sections 4.2 and 5.2.3 above. As such high figures will generally only result from ‘dense’ parapets (ie, high solidity ϕ) and/or shallow depth (ie, low deck depth d_4). Early recognition of potentially high c factors in design is crucial.

5.3 Turbulence response

In keeping with the aerodynamic susceptibility parameter discussed in Section 5.2, a similar parameter has been introduced for turbulence. The general approach to both parameters is given in Section 5.2.1.

A study of turbulence response was made as part of the Aerodynamic Research Panel’s proceedings 1978-81, in conjunction with wind tunnel studies of aerodynamic admittance at NPL/NMI. The first phase of this work was reported by Walshe and Wyatt (1983). The later phases of the wind tunnel studies were reported in NMI Reports (Walshe 1984; Walshe and Elliott 1984): these expanded and fully supported the earlier work.

The available analytic models were reviewed and compared with the results of monitoring five major United Kingdom bridges in service carried out by TRL, notably on Wye and Erskine bridges in 1978 and 1980/81 respectively. This review was comprehensively reported by Hay (1992): the relatively simple formulation of dynamic response which has been taken as the basis of BD 49/93 was recommended as the best available for the purpose. Further monitoring on structures, such as on Humber bridge, has not led to a change of this view.

Some simplification has been achieved in BD 49/01 by curve-fitting to a representative range of examples of steel and composite bridges, including cable-stayed examples. In particular, the fitted result is proportional to the product of the square of the mean speed with the intensity of turbulence, as operative at deck level. This has been further approximated as $0.25V_s^2$, where V_s is the site hourly mean wind speed at 10m above ground level as defined in BD 37 (Highways Agency 1988). The numerical factor of 0.25 corresponds to $S_c^2S_t$ (where S_c is the fetch factor and S_t is the turbulence factor) taken from Table 22 in the detailed procedure of BS 6399: Part 2 (BSI 1997). This approximation is an excellent fit at most bridge levels (heights of 20m and above) only varying between 0.22 and 0.27 becoming conservative at lower levels. For example the BS 6399 values become 0.18 to 0.22 at 10m height, varying with distance from the coast.

It is a well recognised problem that the introduction of an allowance for quasi-resonant response into existing quasi-static design procedures and codes, inevitably causes an increase in the predicted peak response for all cases. In the simple procedure the dynamic effect is deemed to be covered by the load factor. One possibility is to specify an explicit reduction in the load factor to be applied to the prediction made by the full analysis. For the purposes of the waiver, it is necessary to establish a threshold below which the resulting increase can be deemed to be so covered. The threshold levels of P_T have been selected to limit the peak quasi-resonant flexural stress in steel components of deck girders to 25 N/mm^2, and correspondingly the axial stress in concrete forming a composite deck to 3 N/mm^2, and stay stress to 50 N/mm^2. These values do not directly represent actual increases in net stress level, as they combine with the fluctuating component of the quasi-static response by root-sum-square; clearly the actual net increase is considerably less.

The respective unit stress σ_{flm} should correspond to the stresses that will be checked in accordance with BS 5400: there may be cases where dynamic analysis packages incorporate a different modelling, for example, with respect to shear lag or stress concentrations. Where there is doubt, the proper procedure is to apply inertia loads (mass x mode shape function x square of circular frequency) and then use the relevant static analysis.

Two examples illustrate the likely impact of this procedure, the first for longitudinal bending stress and the second for stay tensions for two different long span bridges.

Example 1 (L = 213m) b = 20m, m = 14 t/m, f_B = 0.53 Hz, σ_{f1m} = 210 N/mm^2/m, V_s = 24 m/s, giving P_T = 1.2.
Conclusion - detailed study required.

Example 2 (L = 450m) b = 33m, m = 36 t/m, f_B = 0.31 Hz, σ_{f1m} = 120 N/mm^2/m, V_s = 21 m/s, giving P_T = 0.65.
Conclusion - waiver acceptable.

The question of dynamic response of bridges to gusts is fundamentally different from the case of buildings or towers. By their functional purpose, bridges commonly present a relatively large surface in plan, which is susceptible to excitation by vertical components of gust velocities. The provision in BD 49/01 discussed above is related solely to this point. Horizontal response, driven by the along-wind components of gust velocities, will also occur. This is actually a more conventional problem, for which a similar check based on the lateral-motion natural frequency could be established. The lateral frequencies of practical bridges have not been extensively surveyed, and are unlikely to be as consistent as the vertical frequencies, because of the influence of pier stiffness as well as that of the lateral girder stiffness. Very few cases have arisen where lateral response may be of serious concern. There may be some concern where lateral stiffness and/or pier stiffness is unusually low.

5.4 Divergent amplitude response

The limiting criteria in Clause 2.1.3.4 of BD 49/01 have been modified by amending the probability coefficient (1.3 in BD 49/93) in the light of the better procedures for predicting extreme winds to a reduced figure of 1.25. The factor of 1.3 in BD 49/93 and 1.25 in BD 49/01 reflect the need for achieving a very low probability of occurrence of winds that could induce these severe forms of oscillation for locations in the United Kingdom. Guidance is also now given in BD 49/01 on higher values appropriate for other climatic regions such as for tropical cyclone-prone locations in order to extend the use of the rules to sites outside the UK. However, it should be noted that the probability factor of 1.25 is widely applicable for locations with temperate climates, such as the majority of Europe.

In addition, the reduced critical wind speed expression in Clause 2.1.3.3 of BD 49/93 for flutter has been replaced by a modified expression in BD 49/01. This revision caters for the uncertainty in the margin on the bending to torsion frequency ratio and makes use of a more accurate approximation.

5.5 Wind tunnel testing

The BD 49/01 section on wind tunnel testing has been amplified to allow for the appropriate wind gust speed action integrated over the structure, coincident with horizontal inclination of the wind. This follows very recent work reported by Wyatt (1998) and Irwin (1998).

Although some thought was given to the possibility of inclusion of numerical target values in Annex C of BD 49/01 (such as for turbulence and length scales), it was decided that these may be too specific, and would have set a precedence for all parameters required. However, Annex C of BD 49/01 has been re-organised to improve the sequence of topics and produce a better emphasis and focus for the guidance and advice given. This has included the addition of some introductory paragraphs and other amplifications, in particular relating to considerations of turbulence and the different needs according to the reasons for undertaking the tests.

5.6 Proximity effects

BD 49/93 lacked any guidance on the proximity effects of twin-deck bridges, and thus a new Annex A has been added in BD 49/01 to provide guidelines on arrangements to be considered as twin-deck configurations. Annex A also gives guidance on the consequent evaluation of parameters (ie, to be based on the upwind deck) and a factor to allow for the vortex shedding forces in relation to the upstream and downstream parts of the twin-deck.

The basis for the twin decks is to limit this approach to cases where it can be assumed that the flow pattern is that determined by the overall profiles of the twins, ignoring any modification caused by flow through the gap. The vortex shedding response of a single structure is derived from an excitation expressed by the parameter $C_L V^2$ and a reduction factor K_R, deemed to allow for the favourable effect of real-wind turbulence by comparison with wind tunnel test conditions, which are functions of the overall slenderness of the deck. For twin structures, it can be conservatively assumed that this excitation is applied to one element only of the twin. Because the excitation is now applied to a structure of only one-half of the total mass, but benefits from more favourable K_R of the twin, the net effect is a response increased by a factor of $2^{½}$ rounded to 1.4.

The behaviour of a twin box girder bridge was extensively investigated at Loughborough University in an unpublished study for the case of Friarton Bridge. The study included modelling of potential misalignments or unequal frequency (‘tuning’) of the two components, as well as considering a single box in isolation. In this case, the response of the separate components placed together was not actually greater than that of the single box in isolation, confirming the conservative basis above. There is clearly a need for further investigations into this subject, as there is no other published information available.

Some warning may also be required about proximity effects in relation to turbulence. Based on limited experience such as on Elorn River bridge (Bietry *et al.*, 1994) the limiting value of P_T may need to 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 depth and breadth respectively of the neighbouring structure, including the potential effect of 'new' and 'old'. At Elorn, the separation was at least as large as the value G_2 above. Conversely, at bridges such as Tamar, where the separation is much smaller, no problem was indicated in the unpublished wind tunnel study for this bridge. Unpublished wind tunnel tests for the proposed suspension bridge for the Runcorn–Widnes crossing of the Mersey suggested that the turbulence created by the nearby deep low-porosity trussed railway bridge would cause excessive dynamic response of such a flexible structure. This led to the substitution of the trussed arch design, which has since been built.

5.7 Wind speed criteria and inclination

There is a clear distinction between criteria applicable to vortex shedding effects and those required for divergent amplitude effects. In the former, the consequence of occurrence of the envisaged event will not be a dramatic immediate failure, and the safety margins can justifiably be narrower than those applicable to the ULS check for conventional wind loading. The converse is true for divergent amplitude effects.

5.7.1 Wind speed limit for vortex shedding

At the wind speed level beyond which specific checks can be dispensed with (ie, V_{VS} in Clause 2.1.1.3 of BD 49/01), subjective comfort with respect to perception of motion is immaterial. The procedure for determining stresses associated with vortex shedding in Clause 4 of BD 49/01 is predicated on the likelihood of resonance at modest wind speeds with concurrent traffic. The stress patterns of vortex shedding response in the first mode are of a similar pattern to those of gravitational loading, and in general do not significantly stress members whose function is primarily resisting lateral wind loads. It is shown in the next paragraph that the resulting stress levels are typically much smaller than the gravitational stresses, and trials over a broad range have confirmed that the additional stress levels will not alone be sufficient to cause immediate ULS failure. Thus no specific probabilistic safety margin is required on the wind speed permitting the waiver of further consideration (ie, $V_{VS} = 1.25\ V_r$, as given in Clause 2.1.1.3 of BD 49/01). The factor 1.25 is an allowance for sustained gust speeds by comparison with the hourly mean. This factor was introduced when the first draft rules were published (ICE 1981) and has been found satisfactory in service and compatible with experience of vortex shedding responses of bridges reported since that time. It may be noted that at the low damping values discussed in BD 49/01, build up to the given maximum amplitudes requires resonance over many cycles. At $\delta_s = 0.03$ and sustained exact resonance, 23 cycles are required to build to one-half of the nominal eventual value.

The stresses likely to be generated in the event of resonance can be viewed by considering the maximum amplitude of vertical response as formulated in Clause 3.1.2 of BD 49/01. The maximum acceleration is then

$$a_{\max} = 4\pi^2 f_B^2 y_{\max} = 4\pi^2 f_B^2 \frac{cb^{0.5}d_4^{2.5}\rho}{4m\delta_s}$$

Writing $f_B = 80C_f/L$ defines a parameter C_f (units m/s) which will have values between 1.0 and 2.0 (see 5.2.3). This can be compared with the advice in Clause 2.1 (Note 3); the index 0.87 gives closer representation (provided suspension bridges are excluded) but is clearly less convenient for manipulation. Index unity in place of 0.87 only affects predictions by a factor of 1.23 over the span range 80m to 400m. Simple substitution then gives

$$\frac{a_{\max}}{g} = \frac{6400C_f^2 c}{\frac{m\delta_s}{\rho b}}\left(\frac{d_4}{b}\right)^{0.5}\left(\frac{d_4}{L}\right)^2$$

which, with for example

$$C_f^2 c = 2.3, \frac{m}{\rho b} = 500(m) \text{ and } \delta_s = 0.03 \text{ gives}$$

$$\frac{a_{max}}{g} = \left(\frac{d_4}{b}\right)^{0.5}\left(\frac{10d_4}{L}\right)^2$$

With $d_4/b = 1/4$ and $d_4/L = 1/20$ (say) gives $a_{max}/g = 1/8$. Thus the stresses in such cases are equivalent to inertia loads of 1/8 of the self-weight. There are clearly other cases, such as a continuous uniform beam, where the first-mode acceleration and the stress pattern is more onerous than the dead load pattern. However, it is clearly extremely unlikely that this offers any short exposure danger, due to the low order of a_{max}/g for d_4/b of the order given above.

5.7.2 Wind speed limit for divergent amplitudes

5.7.2.1 Probabilistic considerations

In contrast to shedding, the excitation mechanism relating to divergent amplitudes is potentially so strong that, in probabilistic assessment, occurrence must be equated to catastrophic failure. The safety margin required to give an acceptable notional probability of failure must be applied to the wind speed check value. This must allow for the physical uncertainty of occurrence of extreme wind speeds as a random process as well as uncertainly in both the selection of the wind climate parameters and in transfer from wind tunnel to full size. The further effect of uncertainty or bias on the resistance side of the reliability (failure) equation is negligible.

This probabilistic assessment is catered for by the use of a probability coefficient K_{1A}. The consensus model for extrapolation of extreme wind speeds to very low probabilities of occurrence is the type I extreme value (EV1) distribution fitted to the square of the wind speed. This leads to lower wind speeds for given very low probabilities than the values given by EV1 fitted to the wind speed itself, which was presumed for the original draft rules. Presuming the dispersion parameter for V^2 to be 0.2 times the mode of the annual extremes (following current UK codes), the factor 1.25 corresponds to a 1 in 250 probability of exceedance in a 120 year period. This is a significant improvement in reliability compared to the 1 in 50 target that had been mentioned in the papers supporting the launch of the original draft rules. For comparison, the factor of 1.3 on the earlier model with dispersion/mode ratio for *V* taken as 0.1 corresponded to 1 in 84 probability of exceedance within 120 years.

5.7.2.2 Gust speed correlation considerations

The limiting wind speed criterion V_{WO} (Clauses 2.1.3.4 and 6 of BD 49/01) against which the predictions of critical speed for divergent amplitude response in Clause 2.1.3 are assessed (or for use in section model wind tunnel testing in Clause 6) must relate to a sustained wind speed. In terms of the real conditions, this means a wind speed sustained over the time required for development of response (starting from the conventional gust-induced motions) to a dangerous level, as well as being sufficiently correlated over the length of the structure.

The established spectral analysis procedures for evaluating the effect of averaging, both in time and over the length of a bridge deck, have been applied to a number of examples covering a range of span, terrain and height above ground for two basic structural types. The first, identified as type A in the following discussion, is characterised by a slender and relatively streamlined deck structure. This is likely to show behaviour akin to classical flutter, in which the rate of growth of response amplitude will increase rapidly with excess of actual wind speed over the nominal critical speed. A substantial torsional stiffness, giving torsional natural frequency about twice the values of vertical frequency, is presumed for type A structures. Type B structures comprise less streamlined cross-sections, likely to show simple torsional instability for which a slower growth of response is likely for any given excess of wind speed over the nominal critical speed. For those sections, the torsional resistance of the stiffening girder (GJ) is presumed small. The growth rate factor was taken as 2.0 and 0.5 for A and B respectively, in accordance with Wyatt (1998).

The turbulence model is basically as described in the ESDU Data Items (ESDU 1999) for the strong-wind neutral-stability atmospheric boundary layer. The theoretical algebraic formulations based on homogenous isotropic turbulence have been used, with the superposition of a reduction factor applied to the length scale when evaluating to normalised separation parameter which is used for estimation of correlations along the span. This factor is based on the work of Irwin (1998), taken as $L_3 = 0.5\ L_1$ for high-level estuarial bridges, or $L_3 = 0.4\ L_1$ for the other cases. The span-wise correlation evaluation was weighted according to a typical first-mode shape for a main span and approach span contributions were neglected.

The time averaging is carried out for a range of averaging times, and the critical cases established by trial and error. For example, with a growth-rate factor of 0.5 (case B), excess of averaged wind speed over the steady wind critical speed would give a logarithmic growth of amplitude of 5% per cycle. This means 22 cycles are required for the amplitude to grow by a factor of three, which is considered to be a reasonable safety limit. If the torsional natural frequency were 1.0 Hz, the maximum 22-second space-and-time averaged speed could thus be safely discounted by 10%. Not surprisingly, case A is considerably more onerous, the effect of the higher growth per cycle being amplified by the shorter periodic time and BD 49/01 has been drawn to cover this case.

Typical illustrative cases, taking case A at 15m above ground level, are as follows:

Location	*Estuarial/Coastal*			*Inland/Country*		
Mean speed $V_f\ K_{1A}$ (m/s)	32	32	32	28	28	28
Span (m)	500	200	125	500	200	125
Required value V_{WO} (m/s)	35	38	40	32	35	37
Rule $V_{WO} = 1.10\ ({}^1/3V_r + {}^2/3V_d)\ K_{1A}$ (m/s)	40	41	42	35	36	37

Even if turbulence is introduced in the wind tunnel for section model testing, it will not adequately reproduce the correlations of the perturbations over the extent of the model, and assessment should be based on the mean speed with criterion V_{WO}. For full model testing, it is in principle possible to represent turbulence giving at least a substantial part of the correlated effect on the structure. The criterion V_{WE}, to be applied to the mean speed in the tunnel at deck level, represents partial acceptance of this.

5.7.2.3 Inclination of the wind caused by vertical gust components

BD 49/01 requires a separate assessment of the possibility of consistent inclination of the wind by topography or other major obstacles to the flow. This can be examined by topographic wind tunnel modelling (Annex C of BD 49/01). Experience in UK conditions relating to inclination of wind due to topography is limited and has not had a significant design impact. For example, a major investigation into the proposed second Forth Crossing into the potential effect of flow over the North Queensferry peninsula which might diverge from the peninsula at an angle of about 30º in plan showed only marginal vertical inclination of the wind.

The effects of vertical components of turbulence must always be taken into account, however. A major experimental effort was made to develop criteria for the original Severn Bridge by installing an anemometer array on the railway bridge further up the estuary but it proved very difficult to analyse the analogue charts. The provisional Severn envelope of wind tunnel test wind speed against angle of model inclination was linear from 80% of the value of wind speed for horizontal incidence at ±2º inclination, diminishing to 20% of the horizontal value at ±5º inclination (see Figure 9). This provisional Severn envelope, or generally similar curves, has since been applied in a rather arbitrary fashion, with little distinction according to span or location.

All the factors considered in the foregoing analysis of the along-wind gust action (Section 5.7.2.2) will similarly influence the effective inclination of the wind. Clause 6 of BD 49/01 therefore specifies an inclination proportional to the excess of the design horizontal gust speed (taking account of the span) over the concurrent mean value at the nominal mean hourly speed. The inclination is given as $a = 7\ (S_g/S_m - 1)$ where S_g and S_m can be derived from *Bd* 37

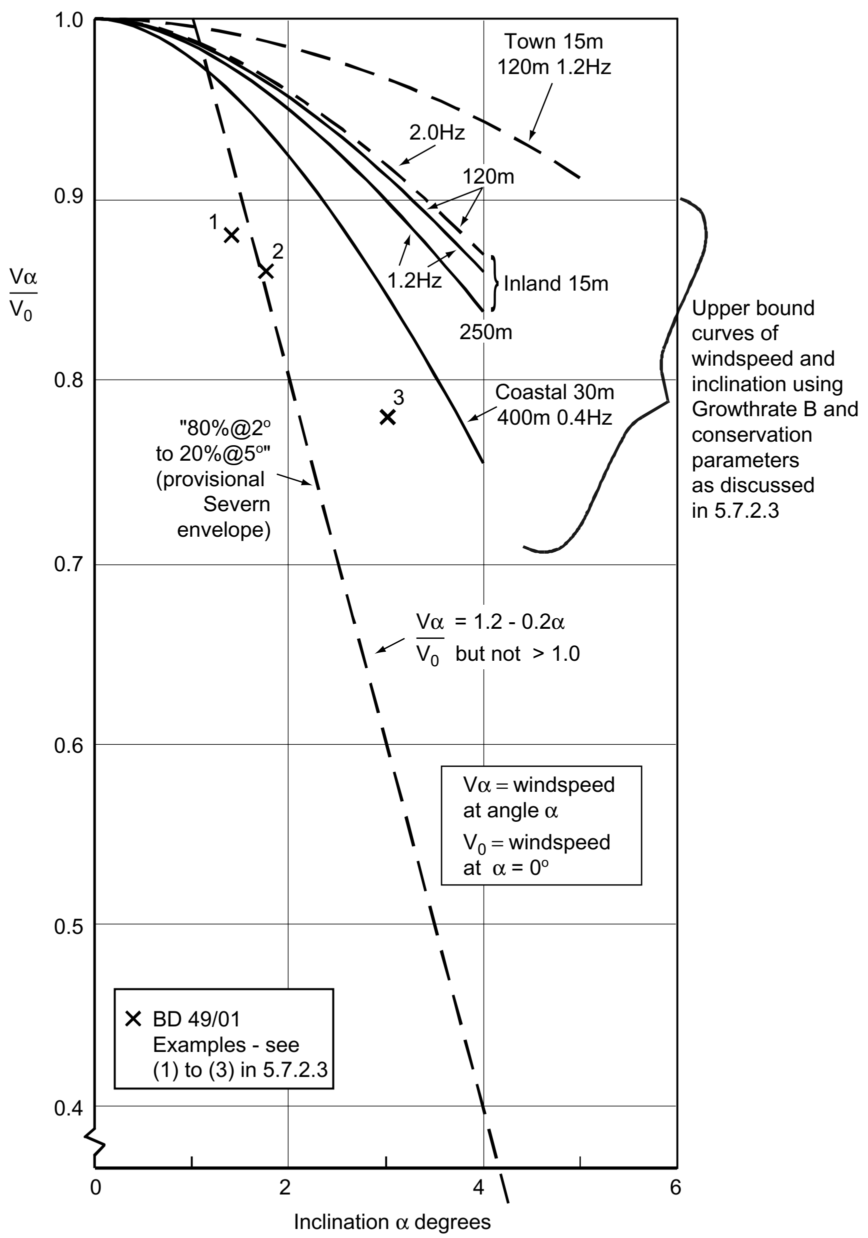


Figure 9 Inclination of the wind from vertical gust components

for a loaded length equal to the longest span. The nominal mean speed is given as $V_{Wa} = V_r\, K_{1A}$, where K_{1A} is the probability coefficient discussed in Section 5.4 above. This gives a modest increase of inclination for shorter spans and for locations (terrain and height above ground) that provide increased turbulence; there is no explicit allowance for natural frequency but clearly higher frequency is linked to shorter spans. The outcome is illustrated by considering three examples:

Example	*Location*	*Distance from sea (km)*	*Height (m)*	*Span (m)*	S_m	S_g	$V_{W\alpha}/V_{WO}$	*α (°)*
(1)	Coastal	0	30	400	1.39	1.66	0.88	1.4
(2)	Country	10	15	250	1.22	1.52	0.86	1.7
(3)	Town*	100	15	120	0.89	1.27	0.78	3.0

** 10km from country*

The first two examples are close to the aforementioned provisional Severn envelope, but the third is significantly more onerous (see Figure 9).

The analytic approach used for the horizontal wind speed analysis described above can be directly extended to consider the effects of the vertical component of turbulence (*w*, giving instantaneous inclination *w/V*) as for the along-wind components. Unfortunately the input data for the *w* component, especially concerning span-wise (horizontal crosswind separation) correlations, are relatively poor. The model of Irwin (1998) introduces two further turbulence-scale reduction factors to express the ground proximity, L_2/L_1 and L_4/L_1 in his notation (also used by Wyatt 1998) for the normalisation of the frequency abscissa for the vertical-component spectrum and the length component of the normalised separation parameter respectively. For the present study, these have been taken as 0.5 and 0.3 respectively (0.6 and 0.4 for the high-level estuarial location where the distortion is less severe). Evaluation of the maximum predicted values of the time-and-space average of component w, with allowance for growth rate of response as before, gives the value required for the angle of inclination. Combination with the foregoing results for the along-wind component gives an interaction curve of wind speed and inclination for each postulated trial value of averaging time, and the envelope of these curves gives the critical criterion as a continuous curve, flat-topped and symmetric about horizontal incidence (see Figure 9). The result of this study is considerably more onerous than the BD 49/01 Clause 6 expression, even for the more favourable low growth-rate postulate, and caution is therefore desirable if tests show marked sensitivity to inclination. However, this expression is generally far more onerous than the currently used Severn envelope and may need further review.

Further explanation and illustration to long spans is given by Wyatt (1998). It is generally found that the governing case in validation by section-model testing arises at a small angle of inclination, most often (for common structural profiles) at a small upwards inclination.

6 Design values

Clause 4 of BD 49/01 has been amplified from that in BD 49/93 to set out more fully the wind load combinations and partial factors to be used in the context of Table 1 of BD 37 in response to user feedback.

7 Footbridges

No separate changes have been made with respect to footbridges. However, certain aspects still need further review to cover various items relating specifically to footbridges, including:

a inherent differences of leading edge geometry;

b application to the newer/novel/esoteric designs not readily encompassed by the present rules;

c whether any modification is required to the discomfort criteria (such as forhorizontal movement) and how or whether this should be integrated with pedestrian activated movement;

d covered footbridges and their special problems;

e special requirements for wind tunnel testing;

It should be noted that, except possibly for b to d above, the rules in BD 49/01 can be used for any configurations that are within the general applicability criteria, which should still be a majority of all types of footbridges. As a cautionary note, the trends in footbridge design, which are heavily influenced by aesthetic considerations, result in lighter structures with unusual cross-sections that are beyond the bounds of codified procedures for aerodynamic behaviour, particularly as these are becoming more dynamically sensitive as a result of their aesthetic considerations. Care must therefore be exercised in extrapolating criteria given in BD 49/01 for all footbridges, particularly those of lighter or unusual cross sections, particularly where the parameters fall outside the indicative values as discussed in Section 5.2.4 above.

8 Further research work

8.1 Aerodynamic susceptibility factor

Some further calibration is required on the application of the sensitivity parameter, as well as with regard to guidance on assumption of typical values for f_B, V_r and $m/\rho b^2$ for preliminary design purposes, as discussed in Section 5.2 above. This should also help in deriving the most robust ranges of span only criteria for steel/composite/concrete bridges, when these bridges should have negligible effects. In the same context, application of the parameter to known examples of novel designs may help in deciding what BD 49/01 can actually cope with, as these depart further away from traditional designs, particularly with regard to footbridges. See also Section 7 above in relation to further studies required on footbridges and the caution needed with regard to novel designs or unusual footbridge configurations.

Further studies are required on cable-stayed and suspension bridges to check on possible inclusion in the use of the susceptibility parameter, probably by means of an effective span, as the susceptibility parameter can only readily cater for inter-related values of L, m, f_B, etc, in such a simplified formulation. However, the general requirements in BD 49/01 encompass cable supported bridges, as the aerodynamic behaviour is based on the shape of deck and leading edge, etc, and the derived frequencies, mode shapes, damping, etc, allow for the overall bridge, mass, geometry and support configurations. However, major long span cable-stayed and suspension bridges will still generally be beyond the scope of BD 49/01 and this should be allowed for in any further calibration studies.

8.2 Proximity effects

As twinning of bridges becomes more prevalent, though still rare, any wind tunnel data should be reviewed in the light of Annex A of BD 49/01, due to the present paucity of information, as discussed in Section 5.6 above. In addition, some further work on the use of slotted holes in decks and longitudinal openings are needed, as BD 49/01 does not encompass such cases except where covered by guidance in Annex A where applicable.

8.3 Wind speed criteria and inclination

Some further considerations and calibration is needed in relation to the inclination of wind caused by vertical gust components, as discussed in Section 5.7.2.3.

9 Summary of consequences of changes

The change in scope of BD 49/01 using the sensitivity parameter, P_b, should generally mean that more bridges can be shown to have negligible aerodynamic effects and less should need wind tunnel tests than the previous span only criteria. The span only criteria was certainly more inaccurate in the categorisation of various types of structures and, in hindsight, not very robust in catering for stiffness and mass variation even for normal bridges. Hence, BD 49/01 will be generally more economic and will be more robust with consequent relaxed requirements and decreased need for wind tunnel tests.

The use of the amplitude correction factor, *c*, will lead to an improved prediction of the likely maximum amplitude, y_{max}, with consequent increased accuracy, avoidance of over-design and a further decrease in the number of bridges potentially needing wind tunnel tests.

The relaxation of the geometric constraints for both edge members and parapet limitations, will also bring many more bridges within the applicability of BD 49/01 relating to galloping and stall flutter (Clause 2.1.3.2) as well as for the prediction of vortex excitation effects (Clause 3.1).

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Abstract

Highways Agency standard BD 49 sets out the design requirements for bridges with respect to aerodynamic effects including the provisions for wind tunnel testing. This standard has recently been revised in response to developments in the understanding of the behaviour of bridges in wind and in particular in light of recent wind tunnel tests commissioned by the Highways Agency. Revisions have also been found to be necessary as a result of experience in using the standard since it was first published in 1993. This included clarification of certain clauses, which had been open to misinterpretation.

This report presents the background to the revisions with a view to clarifying the scope of the revised document and the bounds within which the simplified rules are applicable. It also provides further references which can be consulted to assist in cases where the simplified rules are not applicable.

Related publications

TRL530 *Wind tunnel tests on plate girder bridges* by A F Daly and B W Smith. 2002 (*in production*)

CR256 *A re-appraisal of certain aspects of the design rules for bridge aerodynamics* by Flint and Neill Partnership. 1992 (price £25, code E)

CR36 *Partial safety factors for bridge aerodynamics rules and requirements for wind tunnel testing* by Flint and Neill Partnership. 1986 (price £35, code H)

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