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Background information to the National Annex to BS EN 1991-1-4 and additional guidance

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Foreword

Publishing information

This Published Document is published by BSI and came into effect on 31 December 2015. It has been prepared by Working Group 2 of BSI Subcommittee B/525/1, *Actions (loading) and basis of design*, under the authority of Technical Committee B/525, *Building and civil engineering structures*. A list of organizations represented on this committee can be obtained on request to its secretary.

Supersession

This Published Document supersedes PD 6688-1-4:2009, which is withdrawn.

Information about this document

The new edition of this Published Document introduces the following principal changes:

a) Annex B inserted; and

b) further reading updated.

Relationship with other publications

This Published Document gives non-contradictory complimentary information for use in the UK with BS EN 1991-1-4:2005 and its UK National Annex.

NOTE BS EN 1991-1-4 contains guidance applicable to all structures. Therefore, B/525/10, which is responsible for Eurocodes for the design of bridges, was consulted in the drafting of this Published Document.

Use of this document

This publication is not to be regarded as a British Standard.

As a guide, this Published Document takes the form of guidance and recommendations. It should not be quoted as if it were a specification and particular care should be taken to ensure that claims of compliance are not misleading.

Any user claiming compliance with this Published Document is expected to be able to justify any course of action that deviates from its recommendations.

Presentational conventions

The provisions in this Published Document are presented in roman (i.e. upright) type. Its recommendations are expressed in sentences in which the principal auxiliary verb is "should".

Commentary, explanation and general informative material is presented in smaller italic type, and does not constitute a normative element.

The word "should" is used to express recommendations of this Published Document. The word "may" is used in the text to express permissibility, e.g. as an alternative to the primary recommendation of the clause. The word "can" is used to express possibility, e.g. a consequence of an action or an event.

Notes and commentaries are provided throughout the text of this Published Document. Notes give references and additional information that are important but do not form part of the recommendations. Commentaries give background information.

This Published Document uses the decimal comma.

Contractual and legal considerations

This publication does not purport to include all necessary provisions of a contract. Users are responsible for its correct application.

This Published Document is not to be regarded as a British Standard

Introduction

When there is a need for guidance on a subject that is not covered by the Eurocode, a country can choose to publish documents that contain non-contradictory complimentary information that supports the Eurocode. This Published Document provides just such information and has been cited as a reference in the National Annex to BS EN 1991-1-4:2005.

1 Scope

This Published Document is a background paper that gives non-contradictory complementary information for use in the UK with BS EN 1991-1-4:2005 and its UK National Annex.

This Published Document gives:

a) background to the decisions made in the National Annexes for some of the Nationally Determined Parameters;

b) commentary on some specific subclauses from BS EN 1991-1-4:2005; and

c) additional data that can be used in conjunction with BS EN 1991-1-4:2005.

2 UK National Annex to BS EN 1991-1-4:2005

2.1 The fundamental value of the basic wind velocity $v_{b,0}$ [NA to BS EN 1991-1-4:2005, NA.2.4]

The fundamental value of basic wind velocity $v_{b,0}$ is defined as the 10-minute mean wind velocity with a 0,02 annual risk of being exceeded, irrespective of direction and season, at 10 m above ground level in terrain Category II, which is defined as open country with low vegetation and isolated obstacles with separations of at least 20 obstacle heights.

While the 10-minute averaging period is the meteorological standard for much of continental Europe, some individual countries use 1 hour, including the UK and Germany. Both these countries have adopted a factor of 1,06 to adjust the measured 1-hour average data to the 10-min period, based on empirical calibrations.

In the UK the basic wind velocity is obtained from: $v_{b,0} = v_{b,map} c_{alt}$

"Map" values, $v_{b,map}$ may be found in the UK wind map, which gives values that have been adjusted to sea level and to Category II roughness everywhere. The UK map is similar to the map in BS 6399-2:1997, except that the source data record has been increased from 11 years to 30 years and the original hourly-mean values have been factored up by 1,06 to represent 10-minute mean values. Thus the map in the National Annex is statistically more accurate.

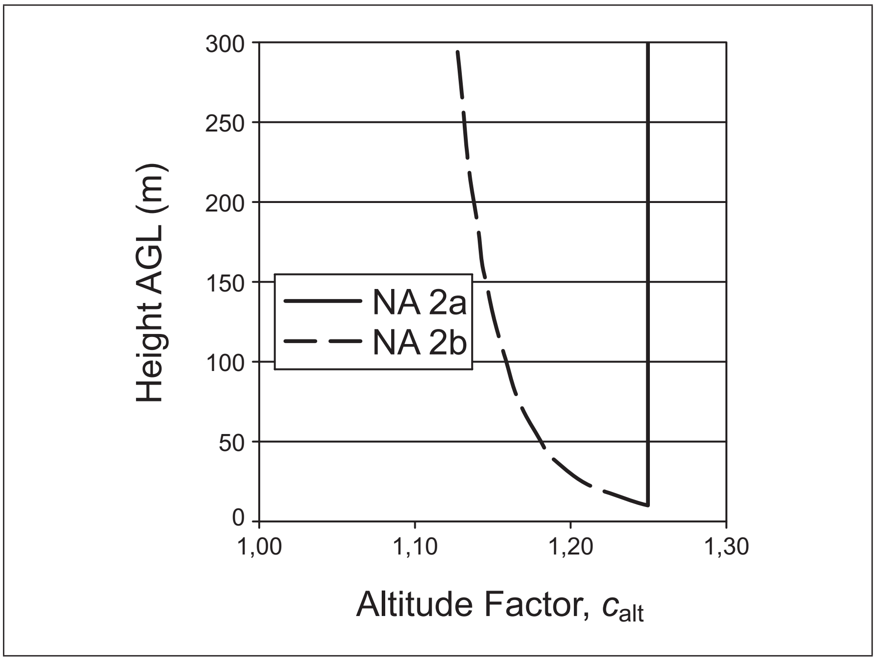
Altitude factor c_{alt} and corrections to account for changes of surface roughness are both National Choices. The former reduces the need to assess the effects of hills (orography) in many cases, while the latter allows conservatism to be reduced for sites further downwind of a coast or town boundary.

2.2 Procedure for determining the influence of altitude [NA to BS EN 1991-1-4:2005, NA.2.5]

In the current UK practice, the altitude factor is taken as constant with height above ground and its value depends only on the altitude of the site. The factor was calibrated empirically against measured data over sites of varying altitude (although generally limited to altitude values below about 200 m). Whilst the simple constant conservative value would be appropriate for structures that are less than 50 m in height and built on sites less than 100 m altitude, it becomes conservative for, say, a 300 m high guyed mast built on a 250 m high hill. Computational Wind Engineering analyses of several high altitude sites, calibrated against known terrain characteristics, confirm this to be the case. Clearly at large heights the altitude effect decreases so that, eventually, at the gradient wind speed height, the factor reduces to zero.

Accordingly, two formulae have been introduced in NA to BS EN 1991-1-4:2005. For the majority of building structures, the simple formula, NA to BS EN 1991-1-4:2005, Equation NA.2a) may be used, without undue conservatism. Figure 1 illustrates the comparison of the two formulae in the NA to BS EN 1991-1-4:2005 for heights up to 300 m above ground level for a site at 250 m above mean sea level. Altitude factor is a simplified substitute for the full orography assessment. The correction that varies with height [formula NA.2b)] removes a small double counting in BS 6399-2:1997; but makes the orography assessment more critical.

Figure 1 **An example of altitude correction factors**



2.3 Procedure for determining the roughness factor $c_r(z)$ [NA to BS EN 1991-1-4:2005, NA.2.11]

The roughness factor $c_r(z)$ accounts for the effect of the rough ground surface on the vertical profile of wind velocity. An approximate logarithmic profile is used in BS EN 1991-1-4:2005, which states that the expression given is valid when the upstream distance with uniform terrain roughness is sufficient to stabilize the profile sufficiently. It has been established that a "fetch" of over 100 km is required to achieve complete equilibrium. The coastline in the UK is such that equilibrium conditions do not generally occur in the UK. Therefore NA to BS EN 1991-1-4:2005 provides an alternative procedure to that indicated in BS EN 1991-1-4:2005. It defines the upstream distance as 100 km and provides a method that accounts for all intermediate values. The values of the roughness factor $c_r(z)$ are presented graphically for ease of use (NA to BS EN 1991-1-4:2005, Figure NA.3 and Figure NA.4).

It is recommended that all inland lakes extending more than 1 km in the direction of wind and closer than 1 km upwind of the site should be treated as "Sea" for the purposes of terrain classification.

2.4 Procedure for determining the orography factor c_0 [NA to BS EN 1991-1-4:2005, NA.2.13]

2.4.1 General

NA to BS EN 1991-1-4:2005, **NA 2.9** specifies that the recommended procedure given in BS EN 1991-1-4:2005, **A.3** should be used to determine the orography factor $c_0(z)$.

This procedure provides formulae (and graphs) to determine $c_0(z)$ for clearly defined cliffs and escarpments and hills and ridges. Unfortunately in the U.K. the majority of escarpments, hills and ridges occur in undulating terrain – only sea edge cliffs can clearly meet the configurations given in BS EN 1991-1-4:2005, **A.3**. In such cases, in the absence of reliable published documents or of verified computer methods, the procedure set out in **2.4.2** may be used. Alternatively, and conservatively, the site may be taken as an isolated hill, or escarpment, with a base level taken at the lowest level of the surrounding terrain within 8 km of the site, but with a slope taken as appropriate to the hill on which the structure is sited.

In all cases a check using the altitude factor alone, appropriate to the site altitude, should be undertaken and the resulting mean wind profile used if this is more onerous, (recognizing that both the altitude factor and orography factor vary with height above ground.)

2.4.2 Guidance

The procedure is shown in Figure 2. The procedure can be used for all wind directions and hence the downwind parameters can be determined in the same manner.

Figure 2 **Hill parameters in undulating terrain**

1. Plot a profile for 8 km in front of the site in the windward direction

2. Draw a typical slope Øe from the hill on which the structure is situated (through C). If the site is behind the brow of the hill, draw an average slope in the windward direction

3. Draw a horizontal through the minimum height level h_{min} within the 8 km distance

4. Find the average height h_{av} over the distance between the 8 km position (O) and the intersection of the slope line with the minimum height line (D)

5. Draw a horizontal at height ($h_{av} - h_{min}$) above the average height level

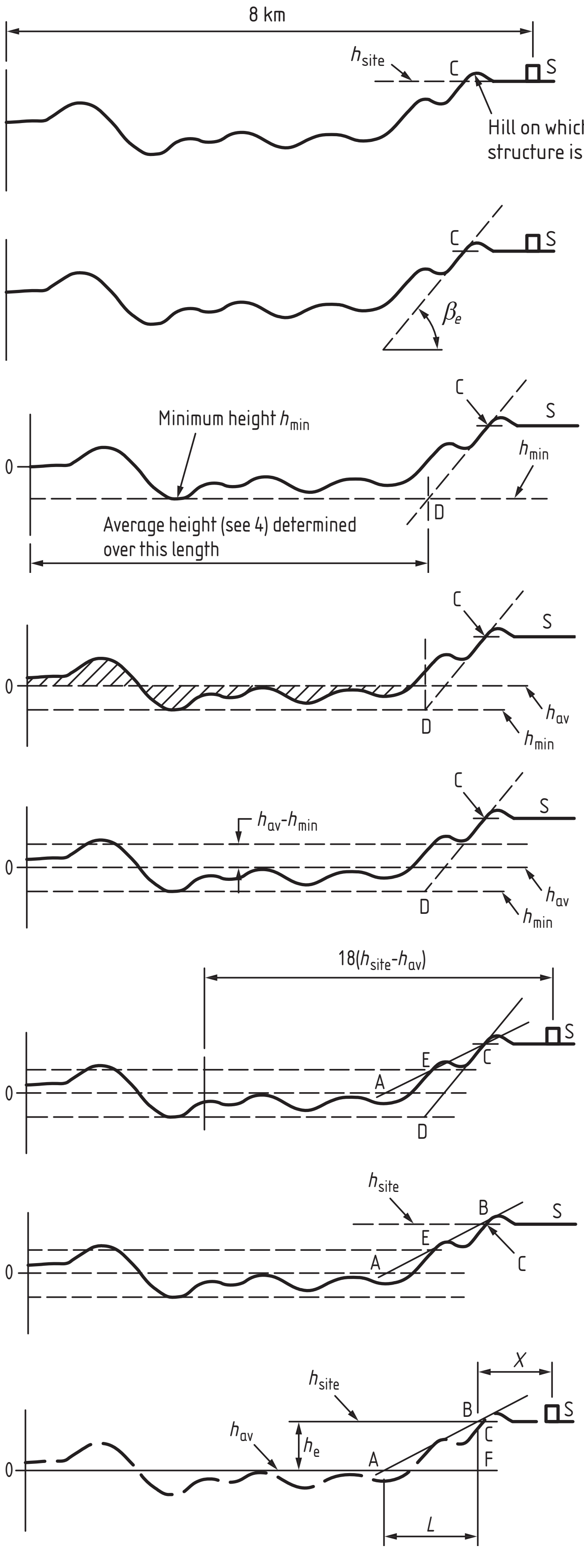
6. Where this line intersects the profile within a distance $18(h_{site} - h_{ave})$ from the site, draw a slope through the intersection point (E) to the profile of the hill on which the structure is situated. This meets the average height level at A. (If there is more than one intersection, use the point furthest from the structure within the prescribed distance)

7. Draw a horizontal through the site level to intersect this slope h_{site} at B

8. The hill is then defined as OABS, with the effective hill parameters, for the purposes of NA to BS EN 1991-1-4:2005, Figure NA.2 and BS EN 1991-1-4:2005, Annex A, taken as:

$A = h_{av}$, L_u = AF, $H_e = h_{site} - h_{av}$,
$\phi = H/L_u$, X = BS

where ϕ is the mean upwind hill slope.



X is the distance of the site downwind from the crest.

2.5 Terrain orography [BS EN 1991-1-4:2005, 4.3.3(2), Annex A.3 and NA to BS EN 1991-1-4:2005, Figure NA.2]

BS EN 1991-1-4:2005, **4.3.3**(2) states that the effects of orography may be neglected if the average slope of the upwind terrain is less than 3°. It then goes on to say: "The upwind terrain may be considered up to a distance of 10 times the height of the isolated orographic feature." In particular cases, the application of this sentence may be unclear and it should be interpreted as follows: this distance should be considered as the minimum distance of a site to the crest of the feature before the effects of the orographic feature can be discounted.

BS EN 1991-1-4:2005, **A.3** provides details of numerical calculations of orography. Paragraphs a) to d) of BS EN 1991-1-4:2005, **A.3**(3) define the location of sites when orography should be taken into account. The data cover upwind slope values of Φ (ratio of the height of feature to upwind length of the slope) in the range of 0.05 and 0.3. It should be noted that NA to BS EN 1991-1-4:2005, does not impose an upper limit for the upwind slope, as in BS 6399-2:1997. See NA to BS EN 1991-1-4:2005, Figure NA.2.

2.6 Determination of the turbulence factor k_I [NA to BS EN 1991-1-4:2005, NA.2.16]

The turbulence factor is required to determine the turbulence intensity $I_v(z)$. BS EN 1991-1-4:2005 adopts a simplified model for $I_v(z)$ whereby the intensity decreases with height in inverse proportion to the increase in mean wind velocity between the heights of z_{min} and z_{max} and a constant value below z_{min}. The model also assumes that the standard deviation of the turbulence is not affected by acceleration of the mean wind velocity over orography, so that the turbulence intensity decreases in inverse proportion to the increase in mean wind velocity over the orography. This model gives quite a poor representation of equilibrium turbulence conditions with height and entirely fails to predict the enhanced turbulence levels just after a change to rougher terrain. In order to avoid all issues of "compensating errors" NA to BS EN 1991-1-4:2005, adopts the full models in all cases, offsetting any complexity of these models by the use of design charts (NA to BS EN 1991-1-4:2005, Figure NA.5 and Figure NA.6). As the parameter required in design is $I_v(z) = k_I/(c_o(z)\ln(z/z_0))$ rather than k_I on its own, charts provide values of $k_I/\ln(z/z_0)$ directly for $c_o(z) = 1{,}0$, so that this can be applied in flat terrains. Where orography is significant, the values from NA to BS EN 1991-1-4:2005, Figure NA.5 should be divided by the relevant value of $c_o(z)$.

2.7 Determination of peak velocity pressure $q_p(z)$ [NA to BS EN 1991-1-4:2005, NA.2.17]

The term "peak velocity pressure" in BS EN 1991-1-4:2005 has the same meaning as "dynamic gust pressure" in BS 6399-2:1997. The gust speed is not explicitly derived in BS EN 1991-1-4:2005. The value of q_p is derived directly using *either*:

- the basic wind speed v_b and applying to it an exposure correction factor c_e; *or*
- the mean wind speed $v_m = c_r v_b$ and applying to it a peak factor (gust) model.

Generally the former is used in static structures and the latter in structures that are considered dynamic.

In BS EN 1991-1-4:2005 the peak factor model has been simplified as a linearized version of a quadratic expression, i.e. it ignores the second order term. This can significantly underestimate the turbulence effects particularly in urban terrain, where the turbulence is greatest. For this reason the NA to BS EN 1991-1-4:2005 uses the full model, i.e. $[1 + 3,0I_v(z)]^2$. The factor 3,0 in the expression represents the gust factor $g(t)$ for the shortest measured gust ($t \approx 1$ s). In BS 6399-2:1997 a value of 3,5 is used for $g(t)$; but that is in conjunction with one-hour averaging period. However BS EN 1991-1-4:2005 uses a 10-minute mean wind speed that is around 6% higher than the hourly mean value. These changes approximately correct for each other to produce similar gust pressures. If the higher value of $g(t)$ is used, the gust wind loads will be overestimated by about 12%.

Calculation of peak velocity pressure is undertaken through charts for c_e or for $I_v(z)$ and c_r.

If the gust speed corresponding to peak velocity pressure is required, this can be deduced by back calculation.

2.8 Value to be used for air density ρ [NA to BS EN 1991-1-4:2005, NA.2.18]

The value of ρ, which is an NDP, depends on the altitude, temperature and barometric pressure to be expected during wind storms. NA to BS EN 1991-1-4:2005 uses a value of 1,226 kg/m^3, which is appropriate for strong winds blowing off the Atlantic Ocean. The recommended value in BS EN 1991-1-4:2005 relates to low temperatures at low altitude.

2.9 Calculation procedure for the determination of wind actions [BS EN 1991-1-4:2005, 5.1]

The steps involved are summarized in BS EN 1991-1-4:2005, Table 5.1. It should be noted that within this general procedure a number of possibilities exist for the determination of the peak velocity pressure q_p in the four orthogonal directions for design purposes. In all cases a 45° sector on either side of the normal to each face is considered.

a) When the orientation of the structure with respect to due North is known, the maximum value of q_p obtained for each 30° quadrant using the appropriate values of direction factor, distance to shoreline and distance into town may be used to determine the maximum value for each orthogonal direction.

b) A more conservative value may be obtained by taking the direction factor as 1,0 and using the minimum distance to shore line and minimum distance into town within the 90° sector.

c) When the orientation of the structure is not known, q_p may be obtained by taking the direction factor as 1,0 and using the minimum distance to shore line in any direction and minimum distance into town in any direction.

Where the combination of the orthogonal loads is critical to the design, for example in deriving stresses in corner columns, the maximum stresses caused by wind in any component may be taken as 80% of the sum of the wind stresses resulting from each orthogonal pair of loads.

2.10 Separation of the structural factor $c_s c_d$ into size factor c_s and dynamic factor c_d [NA to BS EN 1991-1-4:2005, NA.2.20]

The simplified rules for $c_s c_d$ given in BS EN 1991-1-4:2005, **6.2** only give upper bound cases where $c_s c_d = 1$; for example, framed buildings less than 100 m tall. For shorter framed buildings which are not dynamically sensitive $c_s c_d$ is likely to be < 1. However, advantage of this reduction in $c_s c_d$ can only be realized by carrying out the full calculations given in BS EN 1991-1-4:2005, Annex B for the specific building. However, by separating the c_s and c_d factors it becomes relatively simple to determine the product for any building. For a large low-rise building the c_d factor will be approximately = 1, but the c_s factor could be 0.8 or less, giving a 20% reduction in load.

Whilst the Eurocode does not allow different formulations of c_s and c_d, through NA to BS EN 1991-1-4:2005, it is permitted to separate these factors as opposed to using them as a product.

There are a number of advantages in separating them including the following:

- specific allowance for loaded areas;
- specific allowance for non-correlated gusts on bridges [the equivalent to the reduced gust factor with longer loaded length in BS 5400-2 (BD/37)];
- facility to modify the dynamic factor where more accurate values of damping are available;
- values can be obtained directly from table/graphs without recourse to the complexity of using BS EN 1991-1-4:2005, **6.3.1** or Annex D with its in-built assumptions;
- c_s can now be easily applied to cladding panels and elements.

It should be noted that BS EN 1991-1-4:2005 does not provide a method for varying c_s from unity in the calculation of internal pressures to account for the response time of the internal volume. As there is no safe limiting value for internal pressure (e.g. smaller internal pressure might not result in safe wind load on the roof or wall), this change from the current UK practice is sometimes conservative and sometimes not.

The values for c_s and c_d factors given in NA to BS EN 1991-1-4:2005, Table NA.3 and in NA to BS EN 1991-1-4:2005, Figure NA.9 are calculated using the equations in BS EN 1991-1-4:2005, and are similar to the C_a factors and C_r factors given in BS 6399-2:1997. The c_d factor in BS EN 1991-1-4:2005 is based on analysis of structure in the fundamental mode of vibration and thus should be more accurate compared to the dynamic augmentation in BS 6399-2:1997, which deduces safe values based on empirical data for cantilever structures only.

2.11 Asymmetric and counteracting pressures and forces – representation of torsional effects [NA to BS EN 1991-1-4:2005, NA.2.23]

BS EN 1991-1-4:2005 requires consideration of the effects of possible asymmetry of wind loads, which fluctuate in time and position across a structure. Specific guidance is given for a) free standing canopies and sign boards; and b) torsional effects in rectangular structures. For other cases BS EN 1991-1-4:2005 recommends that an allowance for asymmetry should be made by completely removing the design wind action from those parts of the structure where its action will produce a beneficial effect. The guidance for torsional effects is an NDP and the UK considers the code guidance to be non-conservative compared to current UK and other Codes of Practice and to wind tunnel data. For this reason NA to BS EN 1991-1-4:2005 modifies the recommendations, while preserving the general format of the code method. The resulting torsional moment using NA to BS EN 1991-1-4:2005 will be close to that from BS 6399-2:1997, which requires displacement of the loads on each face by 10% of the face width from the centre of the face.

2.12 Procedure for determining the external pressure coefficient for loaded areas between 1 m² and 10 m² [NA to BS EN 1991-1-4:2005, NA.2.25]

The recommended procedure for the transition given in BS EN 1991-1-4:2005 has a logarithmic transition between loaded areas of 1 m^2 and 10 m^2. It has little scientific basis and is unnecessarily complicated; but may have become necessary because of neglect of the non-linear terms in the standard gust-pressure equation, which omission is corrected in NA to BS EN 1991-1-4:2005 (see **2.7**). If implemented it would mean that every cladding panel, window or element that fell within this range of areas would require an additional calculation to determine the c_{pe} value. On a large project this could amount to significant additional calculation effort and opportunity for error. The UK cladding and glazing industry were consulted and they felt that this rule should be simplified. Their preferred approach was to retain the $c_{pe,1}$ values for areas of 1 m^2 or less and use the $c_{pe,10}$ values for all areas > 1 m^2. This is the procedure given in NA to BS EN 1991-1-4:2005, and for areas > 1 m^2 this is the same as BS 6399-2:1997.

2.13 Values of external pressure coefficients for vertical walls of rectangular plan buildings [NA to BS EN 1991-1-4:2005, NA.2.27]

NA to BS EN 1991-1-4:2005, Table NA.4 provides net pressure coefficients for the overall along-wind loading effect (i.e. the combined effect resulting from pressures on front and rear faces). The net coefficients are smaller than the arithmetic sum of the coefficient values for front and rear walls because:

- the external pressure coefficients represent "worst-case" values over a ±45° range of wind directions; and
- they do not necessarily apply over the whole face at one time; the net pressure coefficients were obtained from peak pressure analysis of non-simultaneously measured pressures and therefore overestimate the net loads because peaks of pressure do not occur simultaneously on all surfaces.

In addition to the size factor correction, the factor given in BS EN 1991-1-4:2005 to account for lack of correlation between the pressures on the front and rear faces could also be applied to the net pressure coefficients for structures of significant size, such as buildings. The net pressure coefficients were obtained as the maximum value for any direction using directional coefficients. The lack of correlation arises because the measurements were non-simultaneous.

The external coefficients should be used for the design of cladding and for determining internal pressures when dominant openings are present on those faces. The net coefficients should be used for determining overall loadings.

2.14 Values of $c_{pe,1}$ and $c_{pe,10}$ for vaulted roofs and domes [NA to BS EN 1991-1-4:2005, NA.2.28]

The c_{pe} values given in BS EN 1991-1-4:2005 for cylindrical (barrel vault) roofs are NDPs and are of uncertain origin but are thought to be from smooth flow wind tunnel studies. Data from a recent wind tunnel study [1] have become available since the drafting of BS EN 1991-1-4:2005. The data given in NA to BS EN 1991-1-4:2005 are based on these more recent studies, which are considered to be properly simulated and are presented in a more user friendly format. The studies found that the roof suctions were significantly affected by (length *l*/width *d*) and the data reflect the *l*/*d* dependency.

2.15 Internal pressure – Effect of dominant openings [BS EN 1991-1-4:2005, 7.2.9(3)]

The wording of BS EN 1991-1-4:2005, **7.2.9**(3) dealing with the effect of dominant openings is rather obscure and it is open to different interpretations. It is also incomplete in that there is no advice on the effects at serviceability limit state.

The effects of dominant faces of buildings should be carried out on the basis of the following.

- For ultimate limit state verification, the relevant doors/windows may normally be assumed to be shut during severe wind storms where this is a reasonable assumption (i.e. the closures have sufficient strength and are likely to have been closed for the storm duration).
- A separate condition with these doors/windows open should be considered as an accidental design situation. In this case the wind action should be treated as the accidental action and it should be applied with a partial factor of 1,0. The relevant combination will normally reduce to $\Sigma G_k + A_d + \psi_{1,1} Q_{k,1}$, in which $Q_{k,1}$ represents the characteristic value of any other variable load acting simultaneously. This case will generally represent the critical uplift conditions on the roof.
- Alternative verifications may be appropriate in particular cases, for example, where emergency services might need to have access even during extreme winds. In such cases where relevant, the effect of dominant openings will need consideration in conjunction with extreme winds.

The designer should consider whether verification at serviceability limit state with the relevant door/window might be open (or broken) during less severe wind storms is appropriate. The wind load for this verification should be derived using a probability factor C_{prob} = 0,82. The resulting wind load at serviceability limit state will be approximately equal to the wind load at ultimate divided by $\gamma_{F,ult}$.

2.16 Internal pressure when there are no dominant openings [BS EN 1991-1-4:2005, 7.2.9(6)]

BS EN 1991-1-4:2005 provides a chart for internal pressure coefficients (c_{pi}) in terms of a parameter μ, which is the ratio of the area of openings where c_{pe} is negative or zero to the total area of all openings. This will allow internal pressure coefficients to be determined for buildings of any shape with any combination of permeable and impermeable walls and varying permeability. BS 6399-2:1997 provides data only for particular combinations and thus the limitations of BS 6399-2:1997 have been overcome by BS EN 1991-1-4:2005. NA to BS EN 1991-1-4:2005 provides some data on the porosity of typical construction. It should be noted that absolute porosities are very seldom needed; the expression for μ can be easily operated using relative porosities of the different faces.

2.17 Informative annexes

The informative annexes noted below have not been accepted in NA to BS EN 1991-1-4:2005. The reasons for not accepting them are briefly described.

Annex A.2: This deals with the transition between different roughness categories. NA to BS EN 1991-1-4:2005 implements the roughness-change model used in BS 6399-2, which is well established and verified by considerable research and experience in the UK wind climate. Procedure 1 in BS EN 1991-1-4:2005 would be conservative for considerable distance from the roughness change. Procedure 2 assumes that wind speed is instantly in equilibrium following each change in roughness. In reality the wind speed changes gradually with height above ground and distance from the roughness change. The procedure in NA to BS EN 1991-1-4:2005 implements these gradual changes more correctly.

Annex C: This is an alternative to BS EN 1991-1-4:2005, Annex B for determining the structural factor $c_s c_d$. It is new and untested and hence is not permitted in the UK at this stage.

Annex D: This provides values of $c_s c_d$ for “typical” structures. BS EN 1991-1-4:2005 allows separation of c_s and c_d at national level. NA to BS EN 1991-1-4:2005 implements this separation and as such BS EN 1991-1-4:2005, Annex D is not required.

Annex E: This deals with vortex shedding and aeroelastic instabilities. The main reason for not permitting it in the UK is that it contains no specific information for such responses for bridges. An alternative version that may be used in the UK is given in Annex A to this Published Document.

2.18 Analysis of freestanding lattice towers (see Annex B)

The wind model in BS EN 1991-1-4 adopts five discrete terrain categories and assumes that wind speed and turbulence intensity profiles at a particular site are in equilibrium with the surrounding terrain. The NA to BS EN 1991-1-4 substitutes a transitional model where the wind speed and turbulence intensity profiles at a particular site are a function of the length and roughness of the upwind fetch and the height above ground level.

The method of calculation for along-wind gust buffeting response of freestanding towers given in BS EN 1993-3-1:2006, Annex B is compatible with the wind model in BS EN 1991-1-4. However, it is incompatible with the wind model in the NA to BS EN 1991-1-4; a method of calculation to resolve this incompatibility is given in Annex B.

3 Data that can be used in conjunction with BS EN 1991-1-4:2005

3.1 External pressure coefficients for walls

The values in BS EN 1991-1-4:2005, Table 7.1 and NA to BS EN 1991-1-4:2005, Table NA.4 are also valid for non-vertical walls within ±15° of the vertical.

3.2 Buildings with re-entrant corners, recessed bays or internal wells

The following guidance is based on BS 6399-2:1997.

The external pressure coefficients given in of BS EN 1991-1-4:2005, Table 7.1 and NA to BS EN 1991-1-4:2005, Table NA.4 may also be used for the vertical walls of buildings containing re-entrant corners or recessed bays, as shown in Figure 3, subject to the following.

a) Where the re-entrant corner or recessed bay results in one or more upwind wings to the building, shown shaded in Figure 3a), Figure 3b) and Figure 3c), the zones on the side walls are defined using the crosswind breadth $B = B_1$ and B_3 and the height H of the wing.

b) The zones on the side walls of the remainder of the building are defined using the crosswind breadth $B = B_2$ and the height H of the building.

c) The side walls of re-entrant corners and recessed bays facing downwind, for example the downwind wing of Figure 3a), should be assumed to be part of the leeward (rear) face.

For internal wells and recessed bays in side faces [see Figure 3d)] the following apply, where the gap across the well or bay is smaller than $e/2$ where e = B or $2H$, whichever is less.

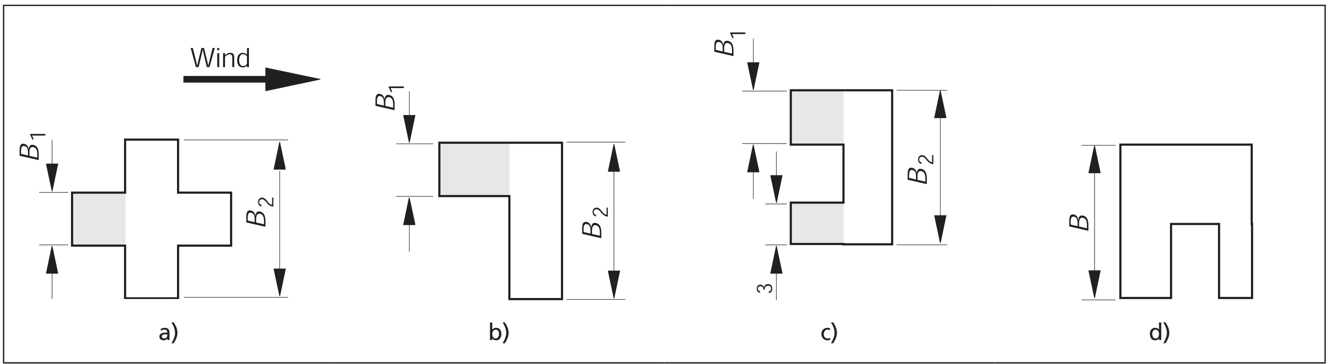
1) External pressure coefficient for the walls of a well is assumed to be equal to the roof coefficient at the location of the well.

2) External pressure coefficient for the walls of the bay is assumed to be equal to the side wall coefficient at the location of the bay.

Where the well or bay extends across more than one pressure zone, the area-average of the pressure coefficients should be taken.

If the gap across the well or bay is greater than $e/2$, the external pressure coefficients should be obtained from specialist literature.

Figure 3 **Typical examples of buildings with re-entrant corners and recessed bays**



3.3 Buildings with irregular or inset faces

3.3.1 Irregular flush faces

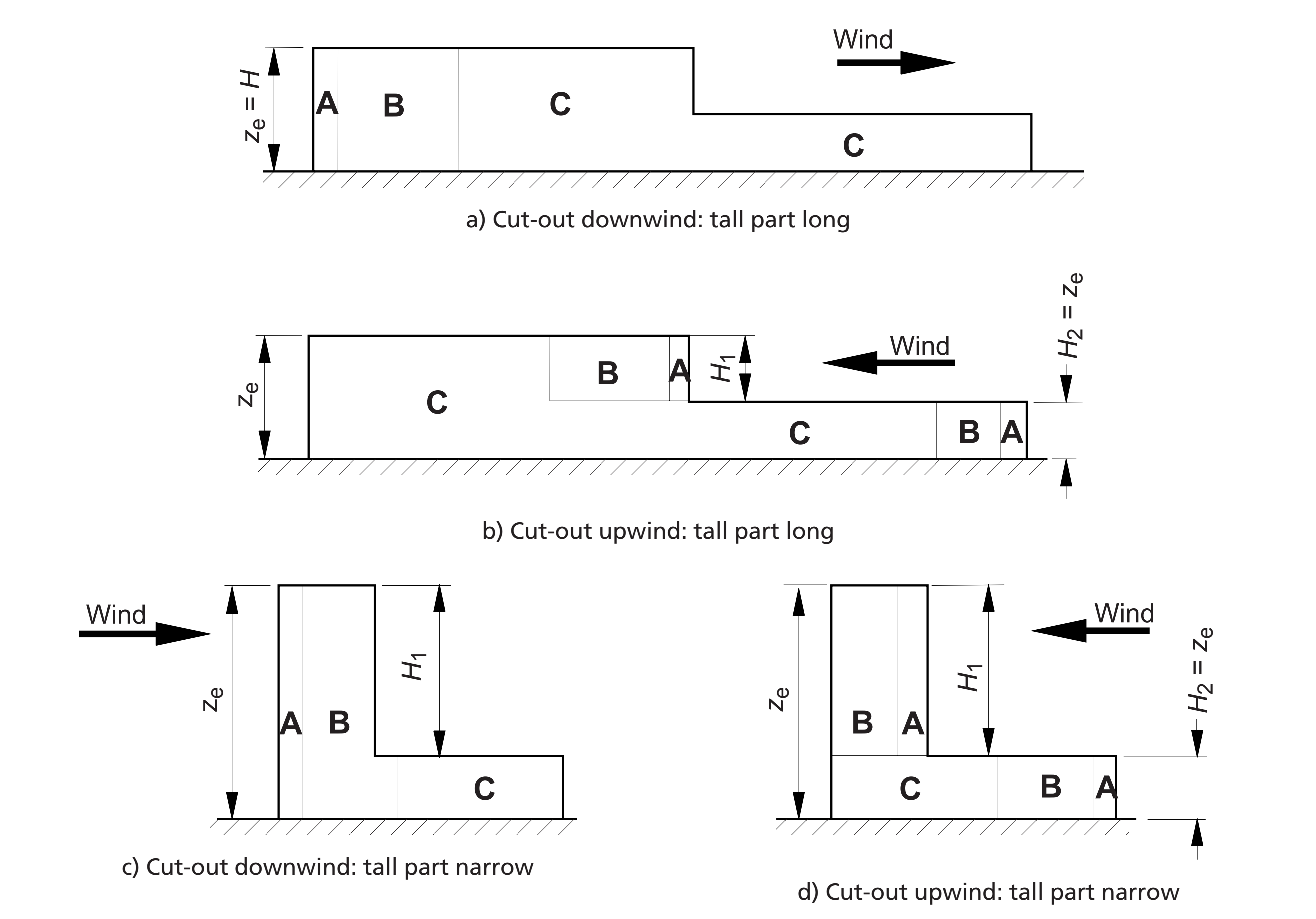
External pressure coefficients for the flush walls of buildings with corner cut-outs in elevation, as illustrated in Figure 4 which include, for example, buildings with a lower wing or extension built flush with the main building, should be derived as follows.

a) *Cut-out downwind*, as in Figure 4a) and Figure 4c). The loaded zones on the face should be divided into vertical strips from the upwind edge of the face with the dimensions shown in BS EN 1991-1-4:2005, Figure 7.5 in terms of the scaling length e, making no special allowance for the presence of the cutout. The scaling length e is determined from the height H and crosswind breadth B of the windward face.

b) *Cut-out upwind*, as in Figure 4b) and Figure 4d). The loaded zones on the face are divided into vertical strips immediately downwind of the upwind edges of the upper and lower part of the face formed by the cut-out. The scaling length e_1 for the zones of the upper part is determined from the height H_1 and crosswind breadth B_1 of the upper inset windward face. The scaling length e_2 for the zones of the lower part is determined from the height H_2 and crosswind breadth B_2 of the lower windward face. The reference height for the upper and lower part is the respective height above ground for the top of each part.

The pressure coefficients for zones A, B and C may then be obtained from BS EN 1991-1-4:2005, Table 7.1. Allowance for funnelling may be applied using the guidance in NA to BS EN 1991-1-4:2005, **NA.2.27**.

Figure 4 **Examples of flush irregular walls**



3.3.2 Walls of inset storeys

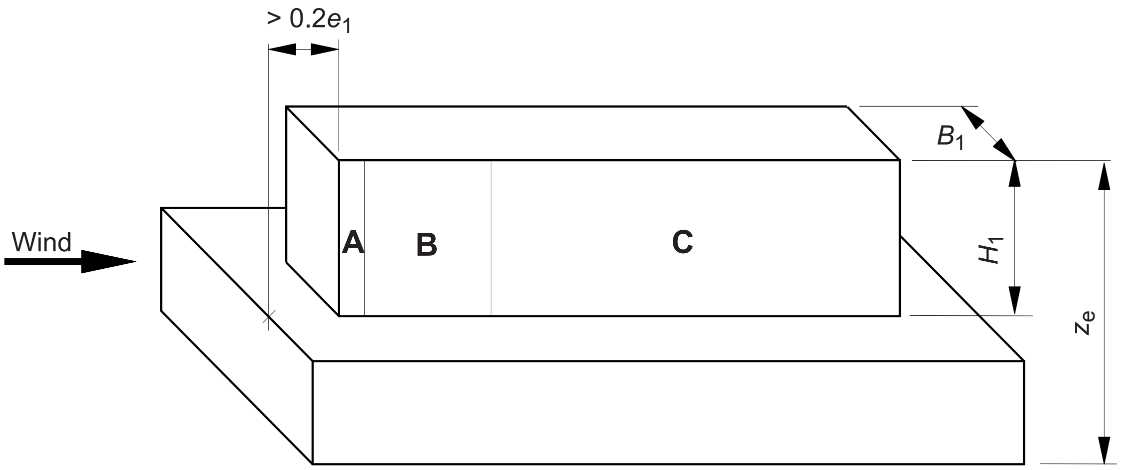
External pressure coefficients for the walls of inset storeys, as illustrated in Figure 5, should be derived as follows.

a) *Edge of face inset from edge of lower storey* [see Figure 5a)]. For the inset walls, provided that the upwind edge of the wall is inset a distance of at least $0{,}2e_1$ from the upwind edge of the lower storey (where e_1 is the scaling length for the upper storey), the loaded zones are defined from the proportions of the upper storey, assuming the lower roof to be the ground plane. However, the reference height z_e is taken as the actual height of the top of the wall above ground.

b) *Edge of face flush with edge of lower storey* [see Figure 5b)]. Where the upwind edge of the wall is flush, or inset a distance of less than $0{,}2e_1$ from the upwind edge of the lower storey, the procedure in a) should be followed, but an additional zone E should be included as defined in Figure 5b) with an external pressure coefficient of $C_{pe} = -2{,}0$. The reference height for zone E should be taken as the top of the lower storey The greater negative pressure (suction) determined for zone E or for the zone A in Figure 5a), should be used.

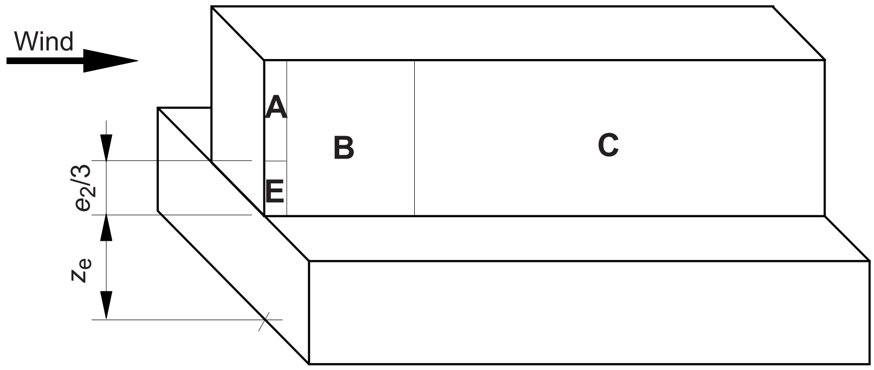
The pressure coefficients for zones A, B and C may then be obtained from BS EN 1991-1-4:2005, Table 7.1.

Figure 5 **Keys for walls of inset storey**



NOTE e_1 *is the scaling length of upper storey.*

a) Edge of face inset from edge of lower storey



NOTE e_2 *is the scaling length of upper storey.*

b) Edge of face flush with edge of lower storey

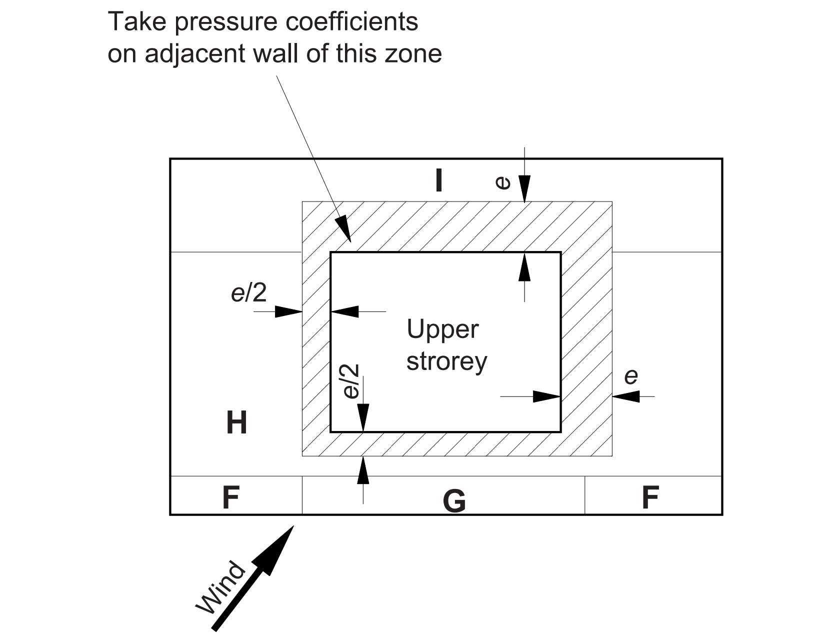
3.4 Flat roofs with inset storeys

For flat roofs with inset storeys, defined in Figure 6, external pressure coefficients for both the upper roofs and lower roofs should be derived as follows.

a) For the upper roof the appropriate procedure of BS EN 1991-1-4:2005, **7.2.3** should be used taking the reference height z_e as the actual height to the upper eaves. The scaling length *e* should be calculated using *H* = height from the upper eaves to lower roof level.

b) For the lower roof the appropriate procedure of BS EN 1991-1-4:2005, **7.2.3** should be used, where $z_e = H$ and is the actual height of the lower storey, ignoring the effect of the inset storeys. However, a further zone around the base of the inset storeys should be included, as shown in Figure 6, where *e* is the scaling parameter from BS EN 1991-1-4:2005, Figure 7.5 appropriate to the relevant walls of the inset storey. The pressure coefficient in this zone should be taken as that of the zone in the adjacent wall of the upper storey (as determined from BS EN 1991-1-4:2005, Table 7.1, or **3.1.4** of this Published Document).

Figure 6 **Key for inset storey**



NOTE e is the scaling length of upper storey.

3.5 Canopies attached to tall buildings

This guidance is based on [2].

For canopies attached below half-way up the building (*h*/*H* < 0.5 in Figure 7) the force coefficients in Table 1 may be used. The data are derived from tests on flat canopies, but are expected to be reasonable for pitched canopies. The reference dynamic pressure q_p should be calculated using reference height $z = H$. The coefficients for $\theta = 0°$ apply to canopy on windward wall and those for $\theta = 90°$ apply to canopies fixed to the side wall. Positive forces act downwards.

Figure 7 **Key to canopies attached to buildings**

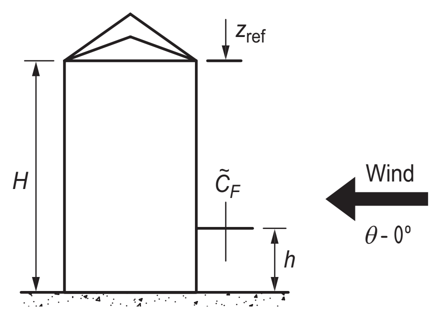


Table 1 **Global vertical force coefficients for canopies attached to tall buildings**

H/h	**2**	**3**	**6**	**12**	**18**	**24**	**30**	**36**
$\theta = 0°$	+0,3	+0,4	+0,69	+0,87	+0,92	+0,93	+0,93	+0,91
$\theta = 90°$	–0,24	–0,70	–0,95	–1,04	–1,16	–1,30	–1,29	–1,16

Canopies attached higher than half-way up the building should be assessed using the rules for free standing canopies fully blocked at one edge. See BS EN 1991-1-4:2005, **2.5.9**.

3.6 Open-sided buildings

Internal pressure coefficients c_{pi} for open-sided buildings are given in Table 2 according to the form of the building. The data have been taken from BS 6399-2:1997. In the table a wind direction of $\theta = 0°$ corresponds to wind normal and blowing into the open face, or the longer face in the case of two open faces, and normal to the wall in the case of three open faces.

For buildings with two opposite open faces, wind skewed at about $\theta = 45°$ to the axis of the building increases the overall side force. This load case should be allowed for by using a net pressure coefficient of 2,2, divided equally between each side wall. More details are given in BS 6399-2:1997.

Table 2 **Internal pressure coefficients c_{pi} for open-sided buildings**

Wind direction θ	One open face		Two adjacent open faces	Three open faces [A)]
	Shorter	Longer		
0°	+0,85	+0,80	+0,77	+0,60
90° [B)]	–0,60	–0,46	–0,57	–0,63
	+0,52	+0,67	+0,77	+0,40
180°	–0,39	–0,43	–0,60	–0,56

A) Values given should be applied to underside of roof only. For the single wall, use pressure coefficients for walls given in BS EN 1991-1-4:2005, Table 7.1.

B) Where two sets of values are given they should be treated as separate load cases.

3.7 Open-topped cylinders

The internal pressure coefficient for an open-topped vertical cylinder, such as a tank, silo or stack, is given in Table 3. The data have been taken from [2].

Table 3 **Internal pressure coefficients c_{pi} for open-topped vertical cylinders**

Proportion of cylinder	c_{pi}
Height/Diameter ⩾ 0,3	–0,8
Height/Diameter < 0,3	–0,5

3.8 Permanently unclad structures

There is limited guidance in BS EN 1991-1-4:2005. References [3] and [4] provide useful guidance and worked examples. Although these were written using BS 6399-2:1997 as the base code, the principles and general data contained in these references may also be used in conjunction with BS EN 1991-1-4:2005.

3.9 Directional method for assessment of wind loads

*NOTE BS 6399-2:1997, Clause **3** contains guidance on directional method. It provides methods for calculating overall loads, loads on walls and roofs for wind from any direction with respect to the axes of the building. Directional pressure coefficients are given. They form an internally consistent data set based on wind tunnel measurements and as such may be used in conjunction with BS EN 1991-1-4:2005. In practice directional wall pressures are commonly required and the relevant data for walls have been extracted and presented here.*

3.9.1 Wind direction

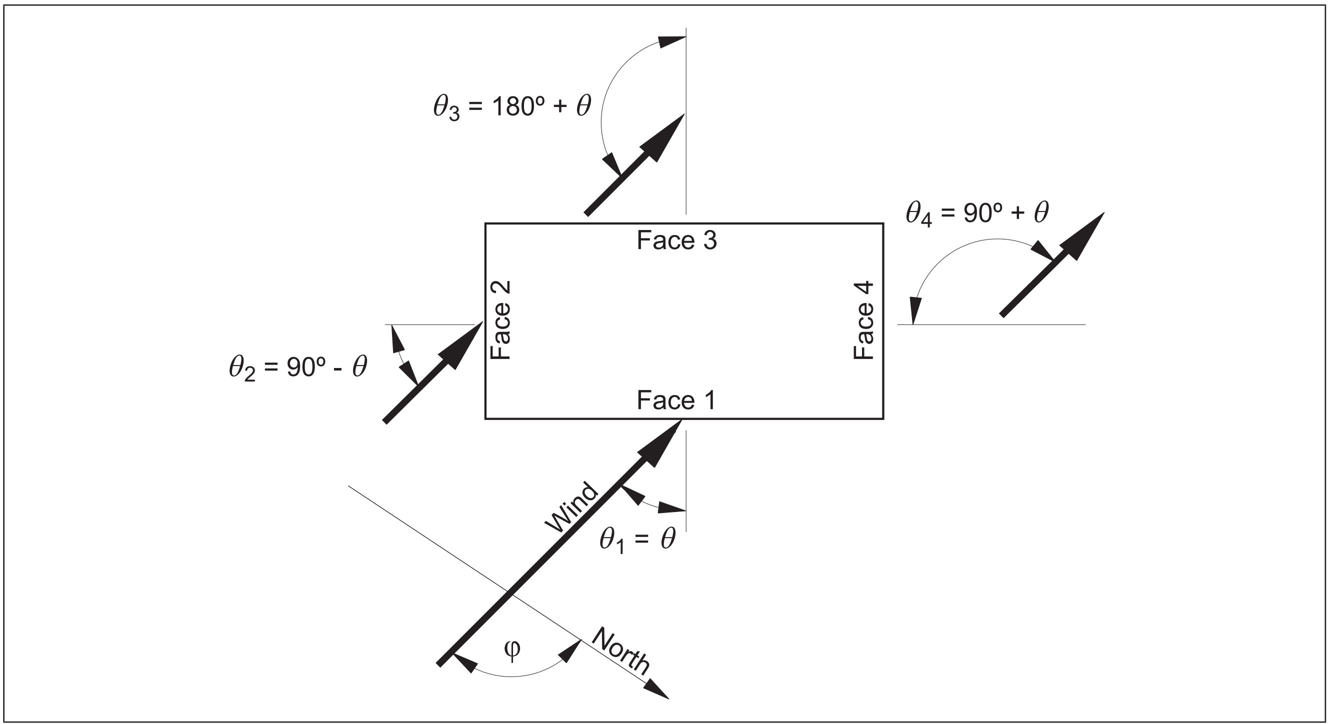
The directional wind load method requires knowledge of the wind direction in two forms:

a) in degrees east of north, represented by φ, used to determine wind speeds and dynamic pressure;

b) in degrees relative to normal to each building face (or around the periphery of a circular-plan building), represented by θ, used to determine the pressure coefficients.

NOTE In practice, it is usually most convenient to relate both and the various values of θ for each face, θ_1, θ_2, θ_3, etc., to a standard value of θ, corresponding to a principal axis or reference face of the building. This is illustrated in Figure 8 for the case of a rectangular-plan building.

Figure 8 **Wind directions for a rectangular plan building**



3.9.2 Directional wind speeds and peak velocity pressures

Wind speeds should be established for different directions (usually for twelve 30° segments from due north), using the procedures in the NA and accounting for the variations in terrain, distances from shore and distances in to town for each direction. The corresponding peak velocity pressures should be calculated for each direction.

3.9.3 Directional external pressure coefficients for walls of buildings

3.9.3.1 Vertical walls of rectangular-plan building

Pressure coefficients for walls of rectangular-plan buildings are given in Table 4 for the zones as defined in Figure 9. Zones A and B should be defined, measuring their width from the upwind edge of the wall. If zones A and B do not occupy the whole of the wall, zone D should be defined from the downwind edge of the wall. If zone D does not occupy the remainder of the face, zone C should then be defined as the remainder of the face between zones B and D.

The wind direction θ is defined as the angle of the wind from normal to the wall being considered (see **3.9.1**). The reference height z_e is the height above ground of the top of the wall, including any parapet, or the top of the part if the building has been divided into parts in accordance with BS EN 1991-1-4:2005, **7.2.2**. The crosswind breadth b and in wind depth d are defined in Figure 10. The scaling length e for defining the zones is given by $e = b$ or $e = 2h$, whichever is the smaller.

Where walls of two buildings face each other and the gap between them is less than e and greater than $e/4$ some funnelling of the flow will occur between the buildings. The maximum effect occurs at a spacing of $e/2$ and is maintained over a range of wind angles ±45° from normal to the axis of the gap. In this circumstance, the following apply.

a) Over the range of wind angle $-45° < \theta < +45°$ the windward-facing wall is sheltered by the leeward facing wall of the other building. The positive pressures in Table 4 apply where the wall is directly exposed to the wind but give conservative values for the whole wall.

b) Over the ranges of wind angle $-135° < \theta < -45°$ and $+45° < \theta < +135°$ funnelling occurs. Values for zone A at $\theta = \pm 90°$ should be multiplied by 1,2. Values for zones B at $\theta = \pm 90°$ should be multiplied by 1,1 and applied to all parts of zones B to D which face the other building over these ranges of wind angle. These “funnelling factors” give the maximum effect which corresponds to a gap width of $e/2$ and interpolation is permitted in the range of gap widths from $e/4$ to b (see NA to BS EN 1991-1-4:2005, **N.A.2.27**).

c) Over the ranges of wind angle $-180° < \theta < -135°$ and $+135° < \theta < +180°$ the values of pressure coefficient remain the same as given in Table 4.

d) Where the two buildings are sheltered by upwind buildings such that the effective height for the lower of the two buildings is $0,4h$, funnelling may be disregarded.

Table 4 **External pressure coefficients C_{pe} for vertical walls of rectangular-plan buildings**

Wind direction, θ	$d/H \leqslant 1$				$d/H \geqslant 4$			
	A	B	C	D	A	B	C	D
0°	+0.70	+0.83	+0.86	+0.83	+0.50	+0.59	+0.61	+0.59
± 15°	+0.77	+0.88	+0.80	+0.68	+0.55	+0.62	+0.57	+0.49
± 30°	+0.80	+0.80	+0.71	+0.49	+0.57	+0.57	+0.51	+0.35
± 45°	+0.79	+0.69	+0.54	+0.34	+0.56	+0.49	+0.38	+0.24
± 60°	+0.24	+0.51	+0.40	+0.26	± 0.20	+0.36	+0.29	± 0.20
± 75°	–1.10	–0.73	+0.23	± 0.20	–1.10	–0.73	+0.23	± 0.20
± 90°	–1.30	–0.80	–0.42	± 0.20	–1.30	–0.80	–0.42	± 0.20
± 105°	–0.80	–0.73	–0.48	–0.26	–0.80	–0.73	–0.48	–0.26
± 120°	–0.63	–0.63	–0.45	–0.29	–0.63	–0.63	–0.45	–0.29
± 135°	–0.50	–0.50	–0.40	–0.33	–0.50	–0.50	–0.40	–0.33
± 150°	–0.34	–0.34	–0.26	–0.32	–0.34	–0.34	–0.26	–0.32
± 165°	–0.30	–0.30	–0.23	–0.28	–0.20	–0.17	–0.15	–0.18
180°	–0.34	–0.24	–0.24	–0.24	–0.17	–0.15	–0.15	–0.15

NOTE 1 Interpolation may be used between given wind directions and for d/H in the range 1 < d/H < 4.

NOTE 2 When the result of interpolating between positive and negative values is in the range $-0.2 < C_{pe} < +0.2$, the coefficient should be taken as $C_{pe} = \pm 0.2$ and both possible values used.

Figure 9 **Key for vertical walls of buildings**

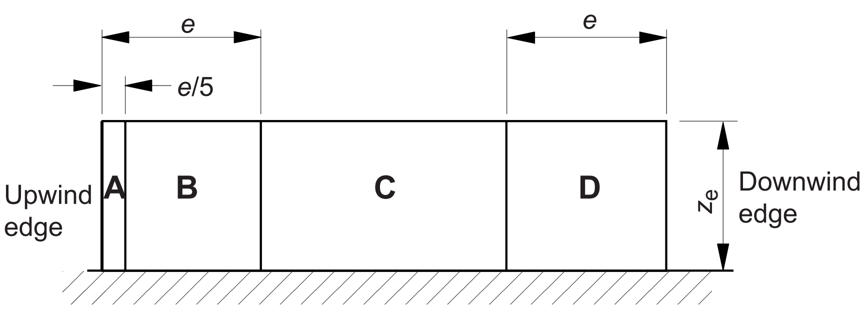
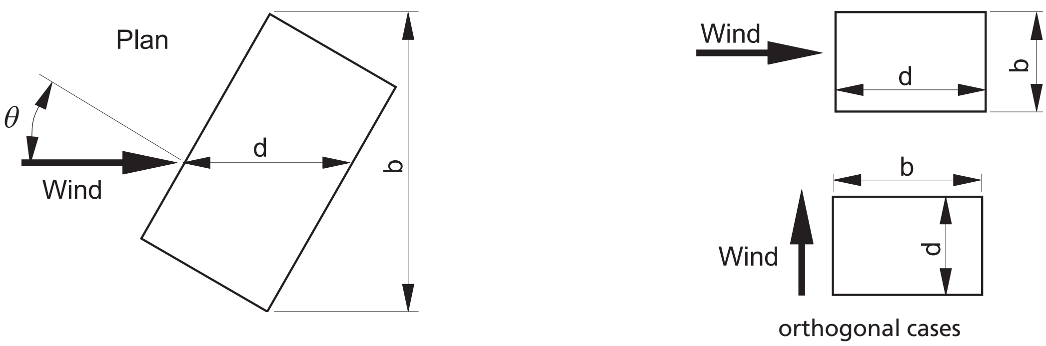


Figure 10 **Definitions of crosswind breadth and in wind depth**



3.9.3.2 Vertical walls of polygonal-plan buildings

The pressure coefficients given in Table 4 should also be used for the vertical walls of polygonal-plan buildings. In such cases there may be any number of faces (greater than or equal to three). The wind direction, principal dimensions and scaling length remain as defined in **3.9.3.1**.

NOTE 1 Instead of calculating the crosswind breadth b and in wind depth d for the complex building plan, these dimensions may be determined from the smallest rectangle or circle which encloses the plan shape of the building.

Provided the length of the adjacent upwind face is greater than $e/5$ the peak suction coefficients for zone A given in Table 4, for wind angle $60° < \theta < 120°$, can be reduced by multiplying them by the reduction factor appropriate to the adjacent corner angle β given in Table 5.

NOTE 2 A rectangular corner $\beta = 90°$ gives the highest local suction in zone A.

Table 5 **Reduction factors for zone A on vertical walls of polygonal-plan buildings**

Corner angle, β	Reduction factor
60°	0.7
90°	1.0
120°	0.6
150°	0.2

NOTE Interpolation is allowed in the range $60° < \beta < 150°$

Whenever the value of pressure coefficient for peak suction in zones B, C and D are more negative than the reduced pressure coefficient in zone A, the reduced zone A values should be applied to these zones also.

Annex A (informative) Vortex shedding and aeroelastic instabilities

NOTE The guidance in this annex is provided as replacement for BS EN 1991-1-4:2005, Annex E.

A.0 Introduction

This annex covers the aerodynamic response of structures, including bridges, to the effects of vortex shedding and other aerodynamic instabilities.

The notation used follows that of BS EN 1991-1-4:2005, the clauses of which are also referred to in the text.

Additional reliable published guidance, wind tunnel testing, or specialist advice, ought to be sought in cases where the methods of this annex reveal that aeroelastic effects are likely to be critical in design.

A.1 Vortex shedding

A.1.1 General

Vortex-shedding occurs when vortices are shed alternately from opposite sides of the structure. This gives rise to a fluctuating load perpendicular to the wind direction. Structural vibrations can occur if the frequency of vortex–shedding is the same as a natural frequency of the structure. This condition occurs when the wind velocity is equal to the critical wind velocity defined in **A.1.3.1**. Typically, the critical wind velocity is a frequent wind velocity indicating that fatigue, and thereby the number of load cycles, can become relevant.

The response induced by vortex shedding is composed of broad-banded response that occurs whether or not the structure is moving, and narrow-banded response originating from motion-induced wind load.

NOTE 1 Broad-banded response is normally most important for reinforced concrete structures and heavy steel structures.

NOTE 2 Narrow-banded response is normally most important for light steel structures.

There are two approaches given in this Annex to calculate the response to vortex excitation of chimneys. Both methods need to be treated with caution.

The first approach (in **A.1.5.2**) includes turbulence and roughness effects but it should be noted that this method of calculation, although of long history, to a large degree contradicts the understanding of vortex shedding provided by the more recent work of Professor BJ Vickery and co-workers and, in particular, fails to explain observed large amplitude aeroelastic instability behaviour of cylindrical structures. Its use in the UK should be limited to cases where there is supporting evidence of its applicability for structures of similar dynamic and aerodynamic properties. It should not be used in cases where **A.1.5.3** is applicable.

The second approach (in **A.1.5.3**) covers, typically, structures such as chimneys or masts. Responses in general are sensitive to conditions of turbulence, including intensity and length scale, which can differ at times due to meteorological conditions. For regions where low-turbulence conditions might occur, such as within a few kilometres of snow/ice-fields or large areas of water, approach **A.1.5.3** gives guidance. Appropriate values of the input parameters (such as K_a and turbulence intensity) are set out in the relevant subclauses. Parameters are not given for grouped or in-line arrangements, and for coupled cylinders and specialist literature or test data may be used to provide this. A more general method is given in various papers by BJ Vickery and co-workers [5] and [6], in GK Verboom and H. van Koten [7] and in Dyrbye and Hansen [8].

Existing guidance on tapered structures is limited mainly to circular shapes. Vickery and Clarke [9] or other papers by Vickery may be used.

Neither of these methods has been developed for building responses. For buildings which may be considered slender (see **A.1.2**), forcing due to vortex shedding may normally be assumed to be of the broadband type and is highly dependent on wind turbulence and surroundings, in addition to sensitivity to building shape. It can also become significant at wind speeds which are lower than the critical wind speed. Where this is likely to be important for design, especially building motions, this should be determined from wind tunnel measurements of the forcing spectra. Initial guidance may be obtained from NBCC [10].

A.1.2 Criteria for vortex shedding

The effect of vortex shedding should be investigated when the ratio of the largest to the smallest crosswind dimension of the structure or element under consideration exceeds six. Both dimensions are taken in the plane perpendicular to the wind.

The effect of vortex shedding need not be investigated when

$$v_{\mathrm{crit,i}} > 1{,}25 v_{\mathrm{m}} \qquad \text{A.1}$$

where

$v_{\mathrm{crit,i}}$ is the critical wind velocity for mode i, for both bending and torsion, as defined in **A.1.3.1**.

v_{m} is the characteristic 10-minute mean wind velocity specified in BS EN 1991-1-4:2005, **4.3.1**(1) at the cross-section where vortex shedding occurs (see Figure A.5). For bridge decks v_{m} is determined at the height of the bridge deck.

Broadband vortex shedding should be investigated up to a speed of $1{,}25v_{\mathrm{m}}$. Where significant vortex shedding responses occur at a windspeed v in excess of v_{m}, the forces for strength design may be scaled-down by dividing by $(v/v_{\mathrm{m}})^2$, where $v_{\mathrm{m}} < v < 1{,}25v_{\mathrm{m}}$.

A.1.3 Basic parameters for vortex shedding

A.1.3.1 Critical wind velocity $v_{\mathrm{crit,i}}$

The critical wind velocity for bending vibration mode i is defined as the wind velocity at which the frequency of vortex shedding equals a natural frequency of the structure or a structural element and is given in Equation A.2.

$$v_{\mathrm{crit,i}} = \frac{a n_{\mathrm{i}}}{St} \qquad \text{A.2}$$

where:

a) *for all structures other than bridges*, $a = b$ the reference width of the cross-section at which resonant vortex shedding occurs and where the modal deflection is maximum for the structure or structural part considered; for circular cylinders the reference width is the outer diameter;

NOTE 1 Although tapered cylinders are generally less prone than parallel forms to significant vortex shedding excitation, tapered cylinders can vibrate over a range of windspeeds depending on the location of the diameter where $v_m = v_{crit,i}$ and the mode excited. See ***A.1.1.7*** *for references.*

b) *for bridges, a* is the depth d_4 (see Figure A.3);

NOTE 2 The need to distinguish between bridges and other structures arises as the basic definitions for the directions of wind actions on bridges are different from those for other structures in BS EN 1991-1-4:2005 (See ***8.1*** *and Figure 8.2 of BS EN 1991-1-4:2005).*

n_i is the natural frequency of the considered flexural ($n_i = n_{i,y}$) or torsional ($n_i = n_{ti}$) mode i of cross-wind vibration calculated under characteristic/nominal permanent load; approximations for $n_{i,y}$ are given in BS EN 1991-1-4:2005, **F.2**.

St is the Strouhal number, values of which for common sections and bridge decks are given in **A.1.3.2**. Alternatively it may be determined from wind tunnel tests on suitable scale models.

Truss bridges with solidity $F < 0,5$ should be considered stable with regard to vortex excited vibrations, where F is the solidity ratio of the front face of the windward truss, defined as the ratio of the net total projected area of the truss components to the projected area encompassed by the outer boundaries of the truss (i.e. excluding the depth of the deck).

The critical wind velocity for ovalling vibration mode i of cylindrical shells is defined as the wind velocity at which two times the frequency of vortex shedding equals a natural frequency of the ovalling mode i of the cylindrical shell and is given in Equation A.3.

$$v_{crit,i} = \frac{b n_{i,0}}{2St} \qquad \text{A.3}$$

where:

b is the outer shell diameter;

St is the Strouhal number as defined in **A.1.3.2**;

$n_{i,0}$ is the natural frequency of the ovalling mode i of the shell.

NOTE 1 For shells without stiffening rings $n_{i,0}$ is given in BS EN 1991-1-4:2005, ***F.2****(3).*

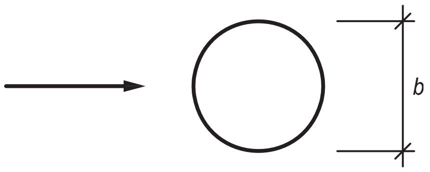
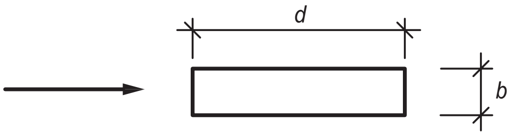
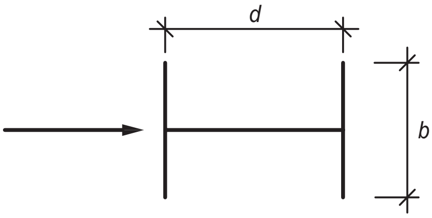
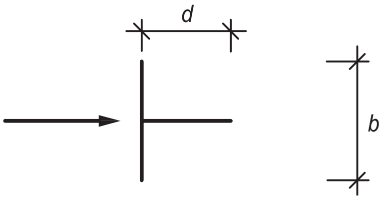
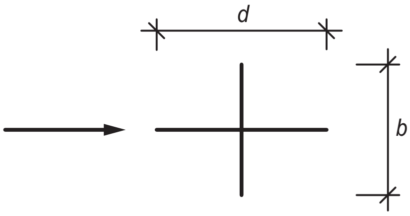
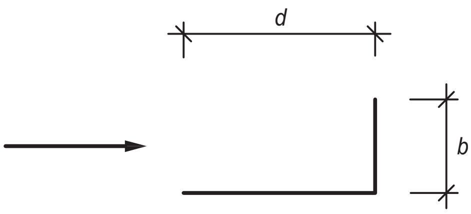
NOTE 2 Procedures to calculate ovalling vibrations are not covered in this Published Document.

NOTE 3 Procedures for calculating broadband vortex shedding responses at wind speeds away from critical are not given in this Published Document.

A.1.3.2 Strouhal number, *St*

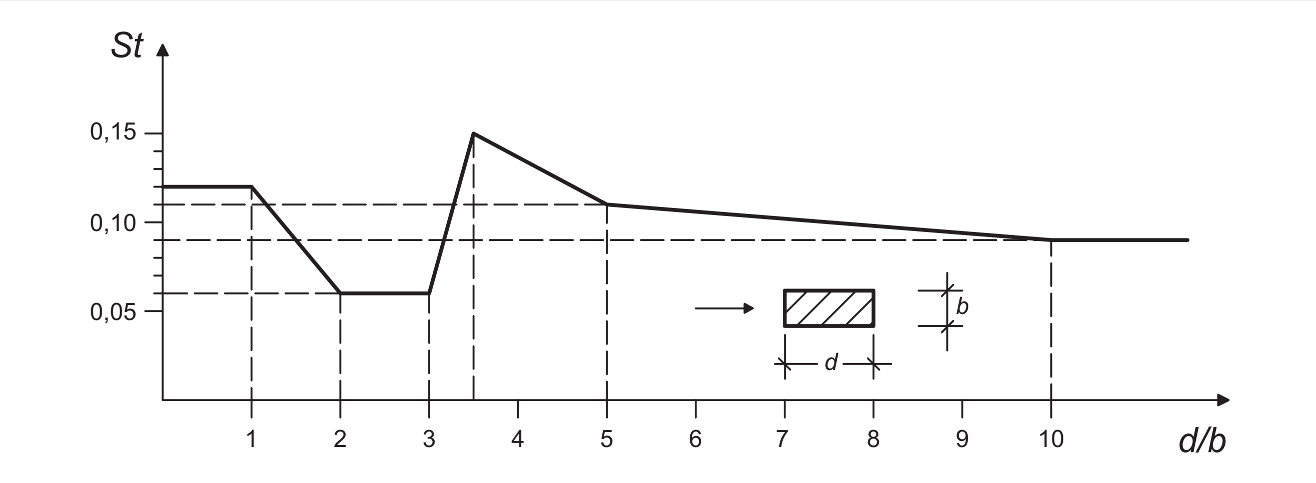
A.1.3.2.1 For cross-sections other than bridge decks the Strouhal number St may be taken from Table A.1.

Table A.1 **Strouhal numbers *St* for different cross-sections**

Cross-section		***St***
 for all *Re*-numbers		0,18
 $0{,}5 \leqslant d/b \leqslant 10$		From Figure A.1
 linear interpolation	$d/b = 1$ $d/b = 1{,}5$ $d/b = 2$	0,11 0,10 0,14
 linear interpolation	$d/b = 1$ $d/b = 2$	0,13 0,08
 linear interpolation	$d/b = 1$ $d/b = 2$	0,16 0,12
 linear interpolation	$d/b = 1{,}3$ $d/b = 2$	0,11 0,07
Bridge sections (see Figure A.3)		From Figure A.2

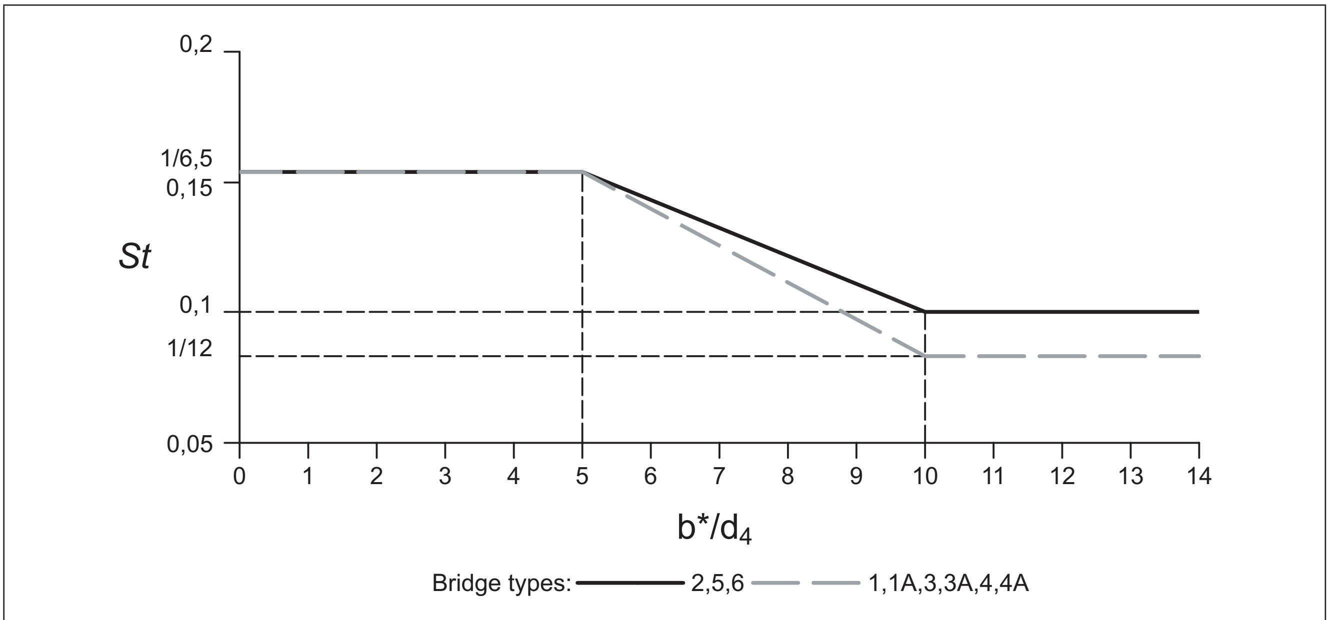
NOTE Extrapolations for Strouhal numbers as function of d/b are not allowed.

Figure A.1 **Strouhal number *St* for rectangular cross-sections with sharp corners**



A.1.3.2.2 For bridge decks the Strouhal number may be taken from Figure A.2.

Figure A.2 **Strouhal number *St* for bridge decks**



NOTE 1 Bridge types are labelled in Figure A.3, reproduced from BS EN 1991-1-4:2005, Figure 8.1.

NOTE 2 In Figure A.2:

- b^* *is the effective width of the bridge as defined in Figure A.3,*
- d_4 *is the depth of the bridge shown in Figure A.3 and Figure A.4. Where the depth is variable over the span,* d_4 *is the average value over the middle third of the longest span.*

A.1.3.3 Scruton number *Sc*

The susceptibility of vibrations depends on the structural damping and the ratio of structural mass to fluid mass. This is expressed by the Scruton number *Sc*, which is given in Equation A.4.

$$Sc = \frac{2\delta_s m_{i,e}}{\rho a^2} \tag{A.4}$$

where

δ_s is the structural damping expressed by the logarithmic decrement;

ρ is the air density under vortex shedding conditions;

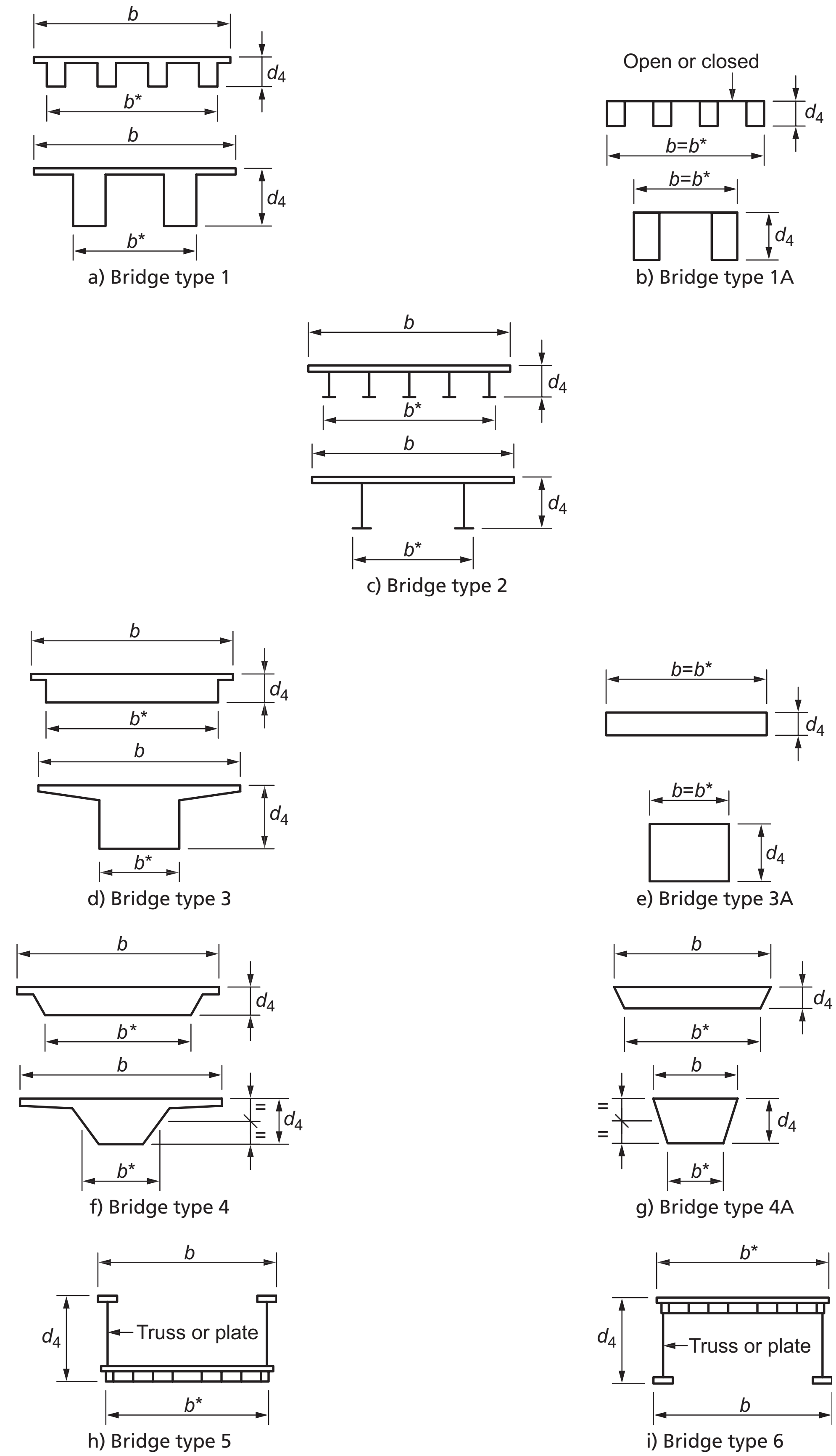
$m_{i,e}$ is the equivalent mass m_e per unit length for mode i as defined in BS EN 1991-1-4:2005, **F.4**(1);

a is the reference width of the cross-section at which resonant vortex shedding occurs, as defined in **A.1.3.1**.

NOTE 1 The value of the air density ρ may be taken as 1,226 kg/m^3.

NOTE 2 Formulae for Scruton number (and specifically equivalent mass, $m_{i,e}$) in BS EN 1991-1-1-4:2005 are strictly valid only for parallel sided elements which are evenly exposed to the wind. For complex structures, more general formulae might be required. In this case refer to specialist publications.

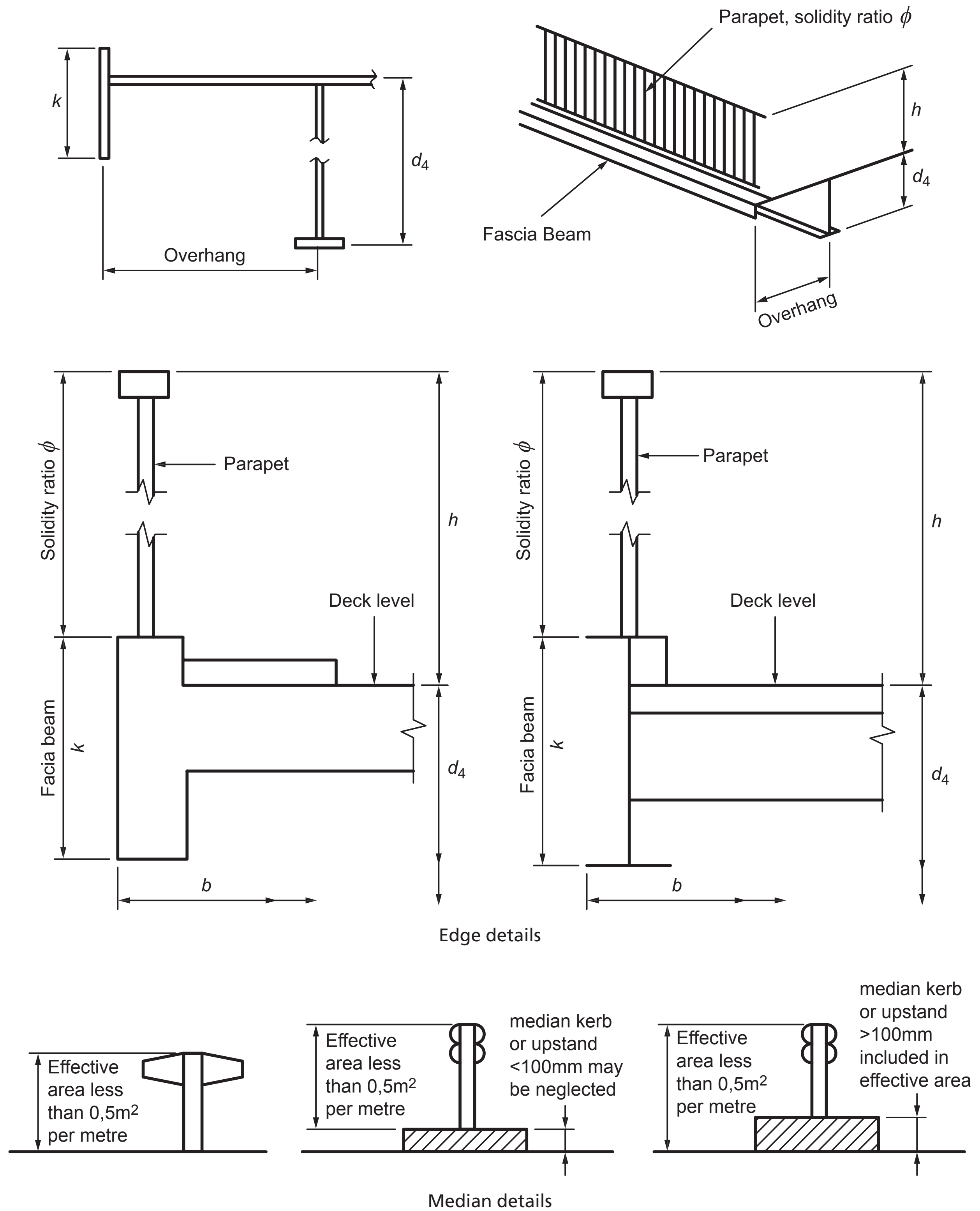
Figure A.3 **Bridge types and reference dimensions**



NOTE 1 For truss bridges of type 5 or 6, d_4 is taken as φd_4, where φ is the truss solidity.

NOTE 2 Trusses with $\varphi \geqslant 0.5$ may be treated conservatively as plate girders but taking the depth d_4 as φd_4.

Figure A.4 **Bridge deck details**



*NOTE For geometric constraints, see **A.1.5.4.2**.*

A.1.3.4 Reynolds number *Re*

The vortex shedding action on a circular cylinder and other bodies with rounded edges depends on the Reynolds number *Re* at the critical wind velocity $v_{crit,i}$. The Reynolds number is given in Equation A.5.

$$Re\left(v_{crit,i}\right)=\frac{bv_{crit,i}}{\nu} \qquad \text{A.5}$$

where

b is the outer diameter of the circular cylinder;

υ is the kinematic viscosity of the air ($\upsilon \approx 15 \times 10^{-6} m^2/s$);

$v_{crit,i}$ is the critical wind velocity, see **A.1.3.1**.

A.1.4 Vortex shedding action

The effect of vibrations induced by vortex shedding should be calculated from the effect of the inertia force per unit length $F_w(s)$, acting perpendicular to the wind direction at location *s* on the structure and given in Equation A.6.

$$F_W(s)=m(s)\left(2\pi n_{i,y}\right)^2 \Phi_{i,y}(s)\, y_{F,max} \qquad \text{A.6}$$

where

$m(s)$ is the vibrating mass of the structure per unit length (in kg/m);

$n_{i,y}$ is the natural frequency of the structure;

$\Phi_{i,y}(s)$ is the mode shape of the structure normalized to one at the point with the maximum displacement;

$y_{F,max}$ is the maximum displacement over time of the point with $\Phi_{i,y}(s) = 1$ (see **A.1.5**).

A.1.5 Calculation of the crosswind amplitude

A.1.5.1 General

A.1.5.1.1 Two different approaches for calculating the vortex excited crosswind amplitudes are given in **A.1.5.2** and **A.1.5.3**. A simplified approach for bridges is given in **A.1.5.4**.

*NOTE 1 See cautionary advice in **A.1.1** regarding the methods in **A.1.5.2** and **A.1.5.3**.*

*NOTE 2 Mixing of the approaches **A.1.5.2** and **A.1.5.3** is not allowed, except if it is specifically stated in the text.*

A.1.5.1.2 The approach given in **A.1.5.2** can be used for various kind of structures and mode shapes. It includes turbulence and roughness effects and it may be used for normal climatic conditions.

A.1.5.1.3 The approach given in **A.1.5.3** may be used to calculate the response for vibrations in the first mode of cantilevered structures with uniform crosswind dimensions from tip to mid-length of the structure. Typically structures covered are chimneys or masts. It cannot be applied for grouped or in-line arrangements and for coupled cylinders. This approach allows for the consideration of different turbulence intensities, which might differ due to meteorological conditions. For regions within 5 km of the coast, where it is likely that it might become very cold and stratified flow conditions might occur, the approach taken in **A.1.5.3** may be used, and appropriate values of the input parameters (such as K_a and turbulence intensity) which should be used in this approach, are set out in the relevant clauses.

A.1.5.1.4 For bridges in the UK, simplified rules for calculating vortex-excited crosswind and torsional amplitudes are provided in **A.1.5.4** which are deemed to satisfy the approaches of **A.1.5.2** and **A.1.5.3**. These rules have been based on a comprehensive parametric study of wind tunnel tests on differing bridge cross-sections.

A.1.5.2 Approach 1, for the calculation of the crosswind amplitudes of buildings and chimneys

A.1.5.2.1 Calculation of displacements

The largest displacement $y_{F,max}$ can be calculated using Equation A.7.

$$\frac{y_{F,max}}{b} = \frac{1}{St^2}\frac{1}{Sc}KK_W c_{lat} \qquad \text{A.7}$$

where:

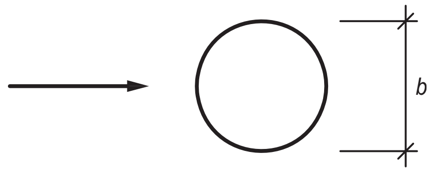
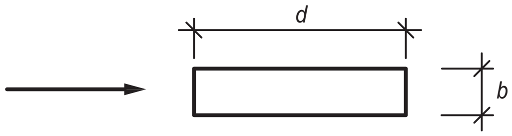
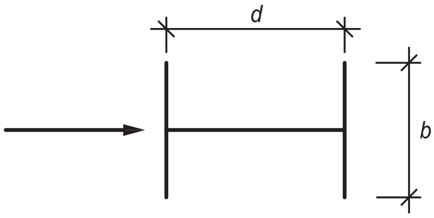
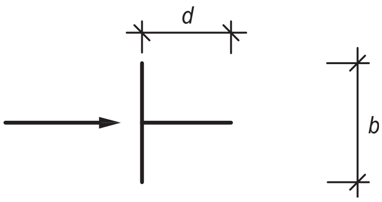
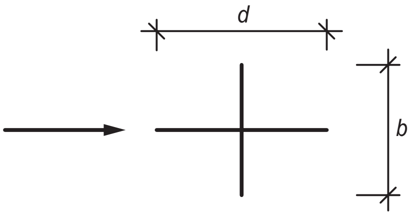
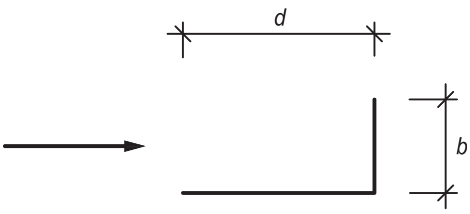
- St is the Strouhal number given in Table 1;
- Sc is the Scruton number given in **A.1.3.3**;
- K_W is the effective correlation length factor given in **A.1.5.2.4**;
- K is the mode shape factor given in **A.1.5.2.5**;
- c_{lat} is the lateral force coefficient given in Table A.2.

NOTE The aeroelastic forces are taken into account by the effective correlation length factor K_W.

A.1.5.2.2 Lateral force coefficient c_{lat}

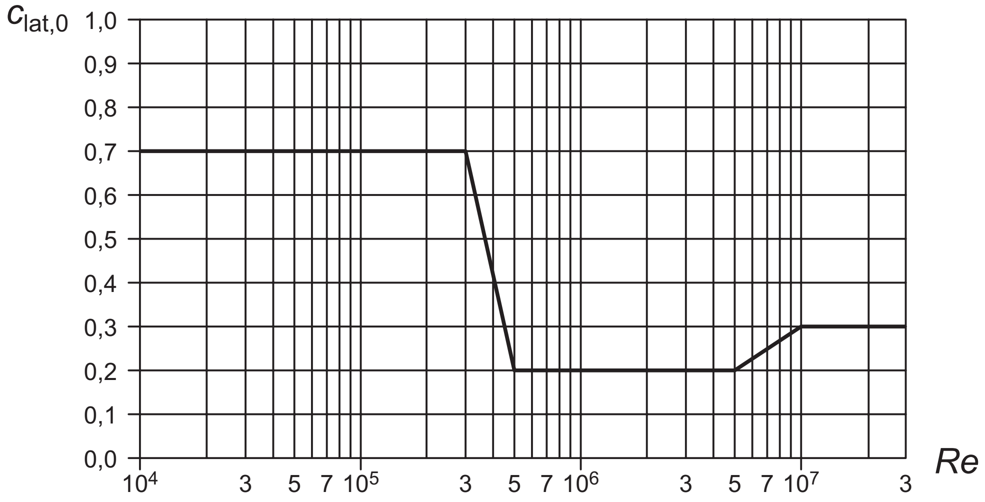
A.1.5.2.2.1 The basic value $c_{lat,0}$ of the lateral force coefficient is given in Table A.2.

Table A.2 **Basic value of the lateral force coefficient $c_{lat,0}$ for different cross-sections**

Cross-section		$c_{lat,0}$
 for all *Re*-numbers		From Figure A.5
 0,5 ⩽ *d*/*b* ⩽ 10		1,1
 linear interpolation	*d*/*b* = 1	0,8
	d/*b* = 1,5	1,2
	d/*b* = 2	0,3
 linear interpolation	*d*/*b* = 1	1,6
	d/*b* = 2	2,3
 linear interpolation	*d*/*b* = 1	1,4
	d/*b* = 2	1,1
 linear interpolation	*d*/*b* = 1,3	0,8
	d/*b* = 2	1,0

NOTE Extrapolations for lateral force coefficients as function of d/b are not allowed.

Figure A.5 **Basic value of the lateral force coefficient $c_{lat,0}$ versus Reynolds number $Re(v_{crit,i})$**



*NOTE For circular cylinders, see **A.1.3.4**.*

A.1.5.2.2.2 The lateral force coefficient c_{lat} is given in Table A.3.

Table A.3 **Lateral force coefficient c_{lat} versus critical wind velocity ratio $v_{crit,i}/v_{m,Lj}$**

Critical wind velocity ratio	c_{lat}
$\frac{v_{crit,i}}{v_{m,Lj}} \leqslant 0{,}83$	$c_{lat} = c_{lat,0}$
$0{,}83 \leqslant \frac{v_{crit,i}}{v_{m,Lj}} < 1{,}25$	$c_{lat} = \left(3 - 2{,}4\frac{v_{crit,i}}{v_{m,Lj}}\right)c_{lat,0}$
$1{,}25 \leqslant \frac{v_{crit,i}}{v_{m,Lj}}$	$c_{lat} = 0$

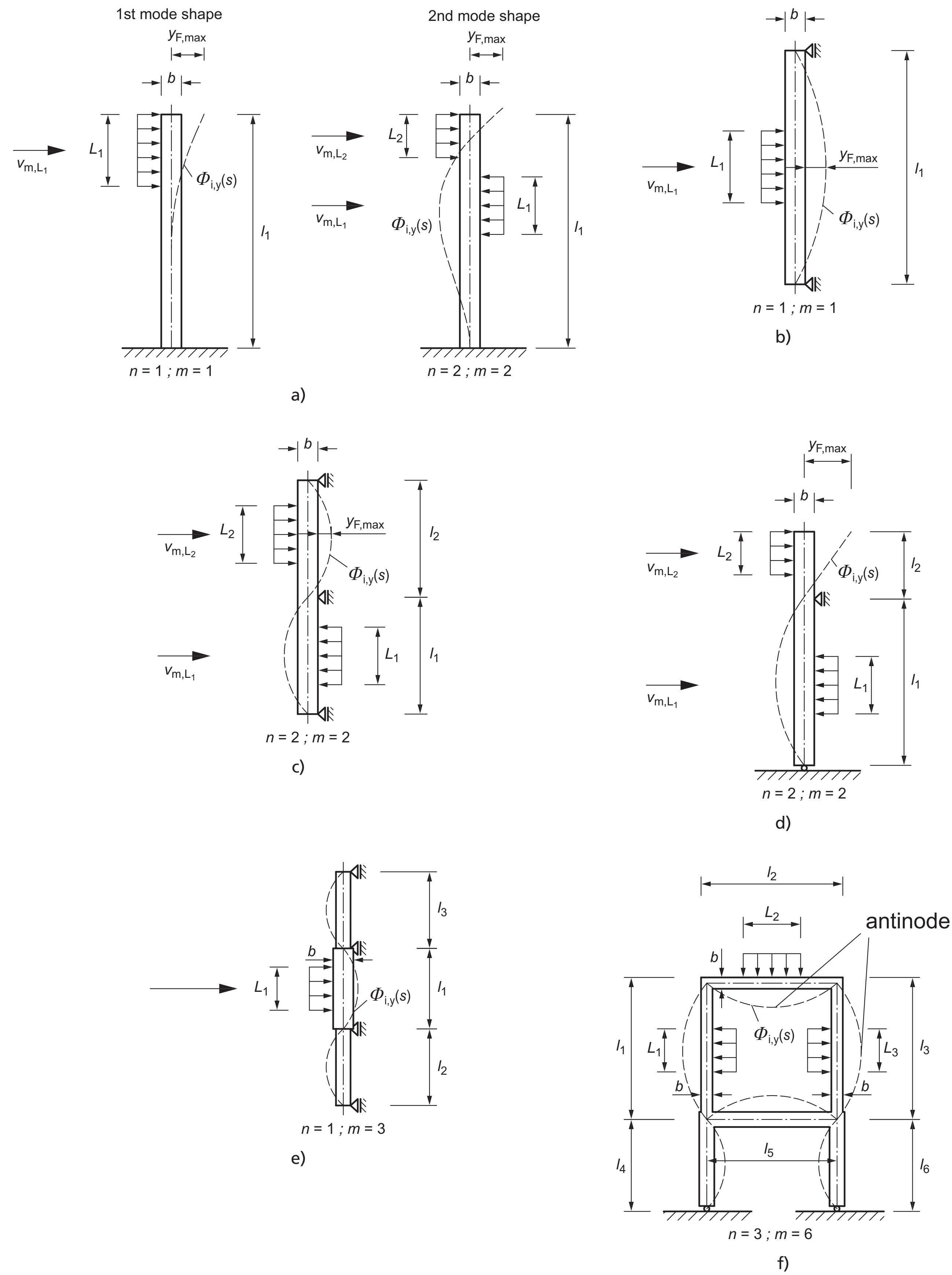
where:

- $c_{lat,0}$ is the basic value of c_{lat} as given in Table A.2 and, for circular cylinders, in Figure A.3;
- $v_{crit,i}$ is the critical wind velocity (see Equation A.1);
- $v_{m,Lj}$ is the mean wind velocity (see **4.2** of BS EN 1991-1-4:2005) in the centre of the effective correlation length as defined in Figure A.6.

A.1.5.2.3 Correlation length L

The correlation length L_j, should be positioned in the range of antinodes. Examples are given in Figure A.6. The relationship between vibration amplitude and effective correlation length is given in Table A.4. For guyed masts and continuous multispan bridges special advice is necessary.

Figure A.6 **Examples for application of the correlation length L_j (j = 1, 2, 3)**



NOTE If more than one correlation length is shown, it is safe to use them simultaneously, and the highest value of c_{lat} is to be used.

Table A.4 **Effective correlation length L_j as a function of vibration amplitude $y_F(s_j)$**

$y_F(s_j)/b$	L_j/b
< 0,1	6
0,1 to 0,6	$4{,}8+12\dfrac{y_F(s_j)}{b}$
> 0,6	12

A.1.5.2.4 Effective correlation length factor K_W

A.1.5.2.4.1 The effective correlation length factor K_W, is given in Equation A.8.

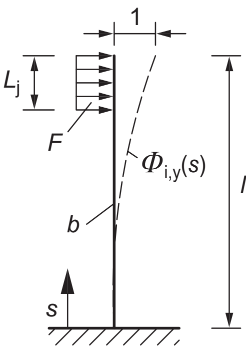
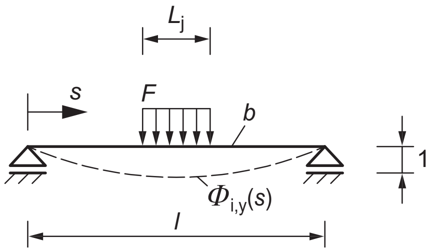
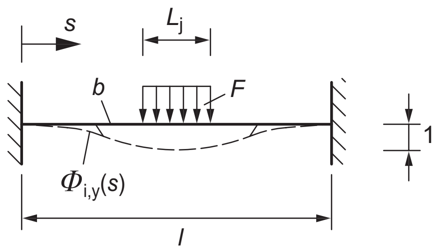
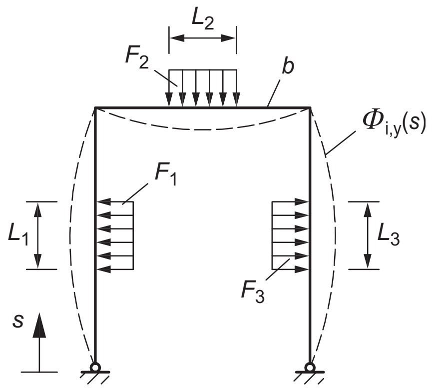
$$K_W=\frac{\sum_{j=1}^{n}\int_{L_j}\left|\Phi_{i,y}(s)\right|ds}{\sum_{j=1}^{m}\int_{\ell_j}\left|\Phi_{i,y}(s)\right|ds}\leq 0{,}6 \qquad \text{A.8}$$

where:

- $\Phi_{i,y}$ is the mode shape i (see **F.3** of BS EN 1991-1-4:2005);
- L_j is the correlation length;
- l_j is the length of the structure between two nodes (see Figure A.6); for cantilevered structures it is equal to the height of the structure;
- n is the number of regions where vortex excitation occurs at the same time (see Figure A.6);
- m is the number of antinodes of the vibrating structure in the considered mode shape $\Phi_{i,y}$;
- s is the co-ordinate defined in Table A.5.

A.1.5.2.4.2 For some simple structures vibrating in the fundamental cross-wind mode and with the exciting force indicated in Table A.5 the effective correlation length factor K_W can be approximated by the expressions given in Table A.5.

Table A.5 **Correlation length factor K_W and mode shape factor *K* for some simple structures**

Structure	**Mode shape, $\Phi_{i,y}(s)$**	**K_W**	***K***
	See **F.3** of BS EN 1991-1-4:2005 with ζ = 2,0 *n* = 1; *m* = 1	$3\frac{L_j/b}{\lambda}\left[1-\frac{L_j/b}{\lambda}+\frac{1}{3}\left(\frac{L_j/b}{\lambda}\right)^2\right]$	0,13
	See Table F.1 of BS EN 1991-1-4:2005 *n* = 1; *m* = 1	$\cos\left[\frac{\pi}{2}\left(1-\frac{L_j/b}{\lambda}\right)\right]$	0,10
	See Table F.1 of BS EN 1991-1-4:2005 *n* = 1; *m* = 1	$\frac{L_j/b}{\lambda}+\frac{1}{\pi}\sin\left[\pi\left(1-\frac{L_j/b}{\lambda}\right)\right]$	0,11
 *m* = 3	Modal analysis *n* = 3 *m* = 3	$\frac{\sum_{i=1}^{n}\int_{L_j}\left\|\Phi_{i,y}(s)\right\|ds}{\sum_{j=1}^{m}\int_{\ell_j}\left\|\Phi_{i,y}(s)\right\|ds}$	0,10

*NOTE 1 The mode shape, $\Phi_{i,y}(s)$, is taken from **F.3** of BS EN1991-1-4:2005. The parameters n and m are defined in Equation A.7 and in Figure A.5.*

NOTE 2 $\lambda = l/b$.

A.1.5.2.5 Mode shape factor

A.1.5.2.5.1 The mode shape factor K is given in Equation A.9.

$$K = \frac{\sum_{j=1}^{m} \int_{\ell_j} \left| \Phi_{i,y}(s) \right| ds}{4\pi \sum_{j=1}^{m} \int_{\ell_j} \Phi_{i,y}^2 (s)\, ds} \qquad \text{A.9}$$

where:

m is defined in **A.1.5.2.4.1**;

$\Phi_{i,y}(s)$ is the cross-wind mode shape i (see **F.3** of BS EN 1991-1-4:2005);

l_j is the length of the structure between two nodes (see Figure A.6).

A.1.5.2.5.2 For some simple structures vibrating in the fundamental cross-wind mode the mode shape factor is given in Table A.5.

A.1.5.2.6 Number of load cycles

The number of load cycles N caused by vortex excited oscillation is given by Equation A.10.

$$N = 2Tn_y\varepsilon_0 \left(\frac{v_{crit}}{v_0}\right)^2 \exp\left[-\left(\frac{v_{crit}}{v_0}\right)^2\right] \qquad \text{A.10}$$

where:

n_y is the natural frequency of cross-wind mode (in Hz). Approximations for n_y are given in Annex F of BS EN 1991-1-4:2005;

v_{crit} is the critical wind velocity (in m/s) given in **A.1.3.1**;

v_0 is $\sqrt{2}$ times the modal value of the Weibull probability distribution assumed for the wind velocity (in m/s), see Note 2;

T is the life time in seconds, which is equal to 3,2 × 10^7 multiplied by the expected lifetime in years;

ε_0 is the bandwidth factor describing the band of wind velocities with vortex-induced vibrations, see Note 3.

NOTE 1 The recommended minimum value of N is 10^4.

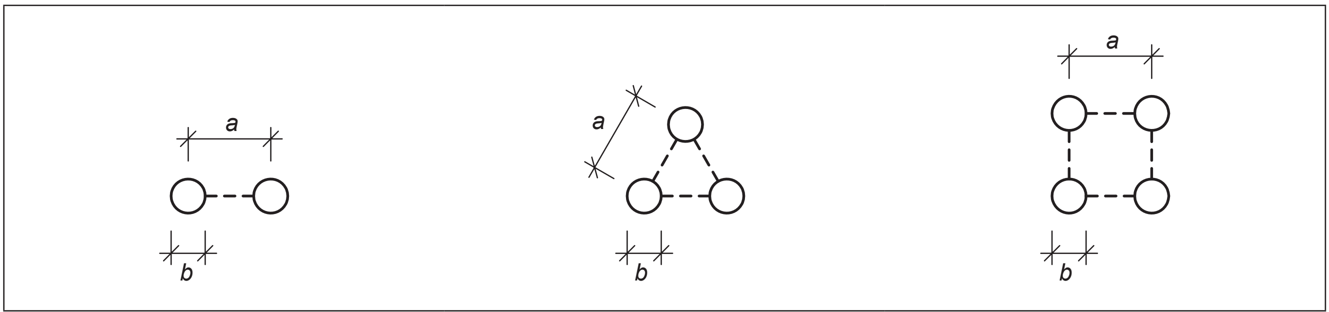
*NOTE 2 The value v_0 can be taken as 20% of the characteristic mean wind velocity as specified in **4.3.1**(1) of BS EN 1991-1-4:2005 at the height of the cross-section where vortex shedding occurs.*

NOTE 3 The bandwidth factor ε_0 is in the range 0,1 to 0,3. It may be taken as ε_0 = 0,3.

A.1.5.2.7 Vortex resonance of vertical cylinders in a row or grouped arrangement

A.1.5.2.7.1 For circular cylinders in a row or grouped arrangement with or without coupling (see Figure A.7) vortex excited vibrations can occur.

Figure A.7 **In-line and grouped arrangements of cylinders**



A.1.5.2.7.2 The maximum deflections of oscillation can be estimated by Equation A.6 and the calculation procedure given in **A.1.5.2** with the modifications given by Equation A.11 and Equation A.12.

For in-line, free standing circular cylinders without coupling:

$$c_{lat} = 1{,}5c_{lat(single)} \quad \text{for } 1 \leqslant \frac{a}{b} \leqslant 10$$

$$c_{lat} = c_{lat(single)} \quad \text{for } \frac{a}{b} \geqslant 15$$

$$\text{linear interpolation} \quad \text{for } 10 < \frac{a}{b} \leqslant 15 \qquad \text{A.11}$$

$$St = 0{,}1 + 0{,}085\log_{10}\left(\frac{a}{b}\right) \quad \text{for } 1 \leqslant \frac{a}{b} \leqslant 9$$

$$St = 0{,}18 \quad \text{for } \frac{a}{b} > 9$$

where:

$c_{lat(single)} = c_{lat}$ as given in Table A.3.

For coupled cylinders:

$$c_{lat} = K_{iv}c_{lat(single)} \quad \text{for } 1{,}0 \leqslant \frac{a}{b} \leqslant 3{,}0 \qquad \text{A.12}$$

where:

K_{iv} is the interference factor for vortex shedding (Table A.9);

St is the Strouhal number, given in Table A.9;

Sc is the Scruton number, given in Table A.9.

For coupled cylinders with $a/b > 3{,}0$ specialist advice is recommended.

NOTE The factor $1{,}5c_{lat}$ *for circular cylinders without coupling is a rough approximation. It is expected to be conservative.*

A.1.5.3 Approach 2, for the calculation of the cross wind amplitudes of buildings and chimneys

A.1.5.3.1 The characteristic maximum displacement at the point with the largest movement is given in Equation A.13.

$$y_{max} = \sigma_y k_p \qquad \text{A.13}$$

where:

s_y is the standard deviation of the displacement, see Equation A.2;

k_p is the peak factor, see Equation A.6.

A.1.5.3.2 The standard deviation σ_y of the displacement related to the width b at the point with the largest deflection ($\Phi = 1$) can be calculated by using Equation A.14.

$$\frac{\sigma_y}{b} = \frac{1}{St^2} \frac{C_c}{\sqrt{\frac{Sc}{4\rho} - K_a \left[1 - \left(\frac{\sigma_y}{b a_L}\right)^2\right]}} \sqrt{\frac{\rho b^2}{m_e}} \sqrt{\frac{b}{h}} \tag{A.14}$$

where:

- C_c is the aerodynamic constant dependent on the cross-sectional shape, and for circular cylinder also dependent on the Reynolds number *Re* as defined in **A.1.3.4**, given in Table A.6;
- K_a is the aerodynamic excitation parameter as given in **A.1.5.3.4**;
- a_L is the normalized limiting amplitude giving the deflection of structures with very low damping; given in Table A.6;
- St is the Strouhal number given in **A.1.3.2**;
- Sc is the Scruton number given in **A.1.3.3**;
- ρ is the air density under vortex shedding conditions, see Note 1;
- m_e is the effective mass per unit length; given in **F.4**(1) of BS EN 1991-1-4:2005;
- h,b is the height and width of structure. For structures with varying width, the width at the point with largest displacements is used.

NOTE 1 The value of the air density ρ may be taken as 1,226 kg/m^3.

NOTE 2 The aerodynamic constant C_c depends on the lift force acting on a non-moving structure.

NOTE 3 The motion-induced wind loads are taken into account by K_a and a_L.

A.1.5.3.3 The solution to Equation A.14 is given in Equation A.15.

$$\left(\frac{\sigma_y}{b}\right)^2 = c_1 + \sqrt{c_1^2 + c_2} \tag{A.15}$$

where the constants c_1 and c_2 are given by:

$$c_1 = \frac{a_L^2}{2}\left(1 - \frac{Sc}{4\pi K_a}\right); \qquad c_2 = \frac{\rho b^2}{m_e} \frac{a_L^2}{K_a} \frac{C_c^2}{St^4} \frac{b}{h} \tag{A.16}$$

A.1.5.3.4 The aerodynamic excitation parameter K_a decreases with increasing turbulence intensity. For a turbulence intensity of 0%, the aerodynamic excitation parameter may be taken as $K_a = K_{a,max}$, which is given in Table A.6.

NOTE Using $K_{a,max}$ for turbulence intensities larger than 0% might give conservative predictions of displacements.

A.1.5.3.5 For a circular cylinder and a square cross-section, the constants C_c, $K_{a,max}$ and a_L are given in Table A.6.

Table A.6 **Constants for determination of the effect of vortex shedding**

Constant	Circular cylinder $Re \leqslant 10^5$	Circular cylinder $Re = 5 \times 10^5$	Circular cylinder $Re \geqslant 10^6$	Square cross-section
C_c	0,02	0,005	0,01	0,04
$K_{a,max}$	2	0,5	1	6
a_L	0,4	0,4	0,4	0,4

NOTE For circular cylinders, the constants C_c and $K_{a,max}$ are assumed to vary linearly with the logarithm of the Reynolds number for $10^5 < Re < 5 \times 10^5$ and for $5 \times 10^5 < Re < 10^6$, respectively.

A.1.5.3.6 The peak factor k_p should be determined.

NOTE Equation A.17 gives the recommended value of the peak factor.

$$k_p = \sqrt{2}\left\{1+1,2\tan^{-1}\left[0,75\left(Sc/4\pi K_a\right)^4\right]\right\} \qquad \text{A.17}$$

A.1.5.3.7 The number of load cycles may be obtained from **A.1.5.2.6** using a bandwidth factor of $\varepsilon_0 = 0,15$.

A.1.5.4 Calculation of the cross wind amplitudes of bridges

*NOTE The following subclauses (**A.1.5.4.1** to **A.1.5.4.5**) relate to highway and railway bridges, for which they were derived. Application of these subclauses to footbridges is to be undertaken with caution, primarily because of the large depth of parapets in relation to the structural depth of the deck.*

A.1.5.4.1 General

The maximum amplitudes of flexural and torsional vibrations y_{max} of bridge decks should be obtained for each mode of vibration for each corresponding critical wind speed less than $1,25v_m$ as defined in **A.1.2**.

The amplitudes of vibration y_{max} 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 in **A.1.5.4.3** provided that the following conditions are satisfied.

a) For all bridge types, edge and centre details conform with the constraints given in **A.1.5.4.2**.

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

A.1.5.4.2 Geometric constraints

For applicability of the reduced velocities for divergent amplitude response (**A.2.4.1**) and the vortex shedding maximum amplitude derivation (**A.1.5.4.3**), the following constraints have to be satisfied.

a) Solid edge members, such as fascia beams and solid parapets have a total depth less than $0,2d_4$ unless positioned closer than $0,5d_4$ from the outer girder when they cannot protrude above the deck by more than $0,2d_4$ nor below the deck by more than $0,5d_4$. In defining such edge members, edge stiffening of the slab to a depth of 0,5 times the slab thickness may be ignored.

b) Other edge members such as parapets, barriers, etc., have a height above deck level, h, and a solidity ratio Φ_s such that Φ_s is less than 0,5 and the product $h\Phi_s$ for the effective edge member is less than $0,35d_4$. The value of Φ_s may exceed 0,5 over short lengths of parapet, provided that the total length projected onto the bridge centre-line of both the upwind and downwind portions of parapet whose solidity ratio exceeds 0,5 does not exceed 30% of the bridge span.

c) Any central median barrier should have a shadow area in elevation per metre length less than 0,5 m^2. Kerbs or upstands greater than 100 mm deep have to be considered as part of this constraint by treating as a solid bluff depth; where less than 100 mm the depth are to be neglected, see Figure A.4.

In the above, d_4 is the reference depth of the bridge deck (see Figure A.3 and Figure A.4). Where the depth is variable over the span, d_4 should be taken as the average value over the middle third of the longest span.

A.1.5.4.3 Approximate formulae

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

For vertical flexural vibrations:

$$y_{\max} = \frac{cb^{0.5}{d_4}^{2.5}\rho}{4m\delta_s} \qquad \text{A.18}$$

for bridge types 1 to 6 of Figure A.3.

For torsional vibrations:

$$y_{\max} = \frac{cb^{1.5}{d_4}^{3.5}\rho}{8mr^2\delta_s} \qquad \text{A.19}$$

for bridge types 1, 1A, 3, 3A, 4 and 4A at the deck edge.

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

In these equations:

$$c = \frac{3(k + h\Phi_s)}{d_4} \text{ but not less than 0.5}$$

where:

- b is the overall width of the bridge deck (see Figure A.3 and BS EN 1991-1-4:2005, Figure 8.2);
- m is the mass per unit length of the bridge (see Note 1);
- ρ is the density of air (see Note 2);
- r is the polar radius of gyration of the effective bridge cross section at the centre of the main span, i.e. (polar second moment of mass/mass)^{½};
- δ_s is the logarithmic decrement due to structural damping;
- h, d_4 and Φ are as defined in Figure A.3 and Figure A.4; and
- k is the depth of fascia beam or edge slab (see Figure A.4).

NOTE 1 Units should be applied consistently, particularly with respect to ρ and m; preferably ρ should be in kg/m^3, with other parameters all in consistent units. Where consistent kg, m units are used the resulting y_{max} is correspondingly given in metres.

NOTE 2 The value of the air density ρ may be taken as 1,226 kg/m^3.

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

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 **A.1.5.4.5** should be followed.

A.1.5.4.4 Damping

Values of δ_s should be obtained from BS EN 1991-1-4:2005, Table F.2 unless appropriate values have been obtained by measurements on bridges similar in construction to that under consideration and supported on bearings of the same type. If the bridge is cable supported the values given should be factored by 0,75.

*NOTE 1 Low wind speeds, where $v_{crit,i}$ is less than about 10 m/s, may need special study; an approximate way to cater for this is for δ_s to be factored by $(v_{crit,i}/1,25v_m)^{1/2}$ but $\leqslant$ 1,00, and with a limit of δ_s not less than 0,02, where $v_{crit,i}$ and v_m are as defined in **A.1.3.1** and **A.1.2.2** respectively.*

NOTE 2 The values for timber and plastic composites are indicative only; in cases where aerodynamic effects are found to be significant in the design, more exact figures should be obtained from specialist sources for the specific project.

A.1.5.4.5 Assessment of vortex excitation effects

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

$K_D = y_{max} n_{bi}^2$ for bending effects A.20a)

$K_D = y_{max} n_{ti}^2$ for torsional effects A.20b)

where:

y_{max} is the predicted bending or torsional amplitude (in mm) obtained from **A.1.5.4.3**;

n_{bi}, n_{ti} are the predicted frequencies (in Hz) in bending and torsion respectively.

Table A.7 then gives the equivalent static loading that should be used, if required, dependent on the values of K_D, to produce the load effects to be considered in conjunction with the wind loads appropriate to $v_{crit,i}$ for the mode of vibration for vortex excitation under consideration. In such cases the partial factor on the equivalent static loading should be taken as 1,2 for ULS and 1,0 for SLS.

NOTE 1 The partial factors on permanent and live loads associated with the above are as required in BS EN 1990 and NA to BS EN 1990.

NOTE 2 It is considered that the combination of vortex excitation with pedestrian vibrations is so unlikely that it can be excluded as a load combination.

Approximate guidance to give an indication of the relative order of discomfort levels for pedestrians may be assessed from Table A.7 according to the derived values of the parameter K_D where:

$K_D = n^2 y_{max}$

where:

n is the natural frequency, in Hz; and

y_{max} is the maximum predicted amplitude, in mm.

The table then indicates where a full discomfort check may be required. In particular, 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, a specific assessment should be carried out based on wind tunnel tests or reference to relevant existing test data for the given cross-section shape. If K_D is still found to be greater than 30 mm/s^2, pedestrian discomfort might be experienced and the design should be modified as appropriate for the specific project.

Table A.7 **Assessment of vortex excitation effects**

K_D	**Effective loading due to vortex excitation in terms of α_D (see Note 1)**		**Motion discomfort only for v_{cr} < 20m/s (see Note 2)**
	A	B	
mm/s2 (See Note 1)	All bridges except those in B	Simply supported highway bridge and all concrete footbridges	All bridges
⩾ 100	When $K_D \geqslant 50$, α_D may be greater than 0,2: evaluate inertia loading using derived y_{max}	When $K_D \geqslant 50$, αD may be greater than 0,16: evaluate inertia loading using derived y_{max}	Pedestrian discomfort possible (see Note 2)
50			
30	When $K_D < 50$, evaluate inertia loading using derived y_{max} or for simplicity use upper bound load, $\alpha_D = 0{,}004K_D$	When $K_D < 50$, evaluate inertia loading using derived y_{max} or for simplicity use upper bound load, $\alpha_D = 0{,}0032K_D$	
20			Unpleasant
10			
5	When $K_D < 12.5$, α_D is less than 0.05 and may be neglected	When $K_D < 12.5$, α_D is less than 0,04 and may be neglected	Tolerable
3			
2			Acceptable
1			Only just perceptible

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

NOTE 2 Inertia loading is $\pm y_{max}\Phi m(2\pi n_{bi})^2$, in which m is the mass per unit length (including an allowance for live load). A corresponding procedure is applicable for torsional motion.

NOTE 3 When the critical wind speed for excitation in the relevant mode is greater than 20 m/s, motion discomfort is generally not experienced by any pedestrians still using the bridge due to the strength and buffeting effects of the associated gale force winds. For more information see [11] and[12].

A.1.6 Measures against vortex induced vibrations

Vortex-induced amplitudes may be reduced by means of aerodynamic devices such as helical strakes (only under special conditions, e.g. Scruton numbers larger than 8) or damping devices supplied to the structure. The drag coefficient c_f for a structure with circular cross-section and helical strakes based on the basic diameter b may increase up to a value of 1,4. Both applications require special advice.

A.2 Galloping

A.2.1 General

A.2.1.1 Galloping is a self-induced vibration of a flexible structure in crosswind bending mode. Non-circular cross-sections including L-, I-, U- and T-sections are prone to galloping. Ice can cause a stable cross-section to become unstable.

A.2.1.2 Galloping oscillation starts at a special onset wind velocity v_{CG} and normally the amplitudes increase rapidly with increasing wind velocity.

A.2.1.3 Procedures to estimate the onset wind velocity for individual members and the criteria to be satisfied are given in **A.2.2** and **A.2.3**. Procedures for bridge decks are given in **A.2.4**.

A.2.2 Galloping of individual members

A.2.2.1 The onset wind velocity of galloping, v_{CG}, is given in Equation A.21.

$$v_{CG} = \frac{2Sc}{a_G} n_{1,y} b \qquad \text{A.21}$$

where:

Sc is the Scruton number as defined in **A.1.3.3**;

$n_{1,y}$ is the crosswind fundamental frequency of the structure; approximations of $n_{1,y}$ are given in BS EN 1991-1-4:2005, **F.2**;

b is the width as defined in Table A.8;

a_G is the factor of galloping instability (Table A.8); if no factor of galloping instability is known, $a_G = 10$.

A.2.2.2 It should be ensured that:

$$v_{CG} > 1{,}25 v_m \qquad \text{A.22}$$

where:

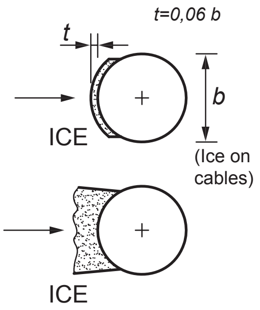
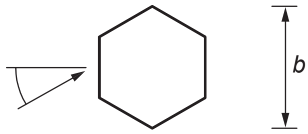
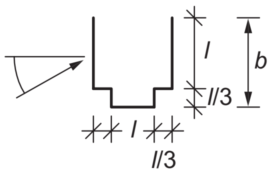
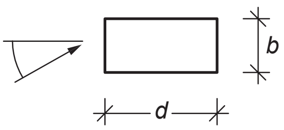
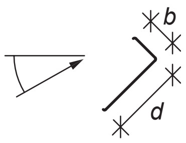
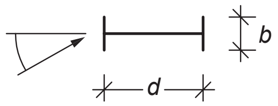
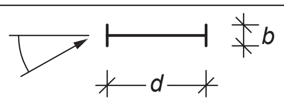
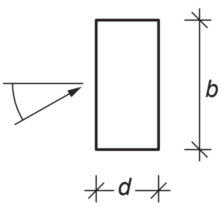
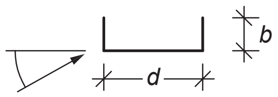
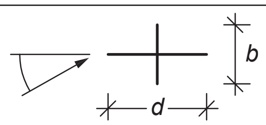
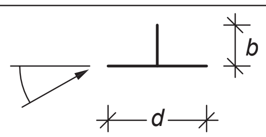
v_m is the mean wind velocity as defined in Equation 4.3 of BS EN 1991-1-4:2005 and calculated at the height, where galloping process is expected, likely to be the point of maximum amplitude of oscillation.

A.2.2.3 If the critical vortex shedding velocity v_{crit} is close to the onset wind velocity of galloping v_{CG}:

$$0{,}7 < \frac{v_{CG}}{v_{crit}} < 1{,}5 \qquad \text{A.23}$$

interaction effects between vortex shedding and galloping are likely to occur. In this case specialist advice is recommended.

Table A.8 **Factor of galloping instability a_G**

Cross-section		**Factor of galloping instability, a_G**	**Cross-section**		**Factor of galloping instability, a_G**
		1,0			1,0
					4
	$d/b = 2$	2		$d/b = 2$	0,7
	$d/b = 1{,}5$	1,7		$d/b = 2{,}7$	5
	$d/b = 1$	1,2		$d/b = 5$	7
	$d/b = 2/3$	1		$d/b = 3$	7,5
	$d/b = 1/2$	0,7		$d/b = 3/4$	3,2
linear interpolation	$d/b = 1/3$	4		$d/b = 2$	1

NOTE Extrapolations for the factor a_G as a function of a/b are not allowed.

A.2.3 Classical galloping of coupled cylinders

A.2.3.1 For coupled cylinders (Figure A.6) classical galloping can occur.

A.2.3.2 The onset velocity for classical galloping of coupled cylinders, v_{CG} may be estimated by Equation A.24

$$v_{CG} = \frac{2Sc}{a_G} n_{1,y} b \tag{A.24}$$

where Sc, a_G and b are given in Table A.9 and $n_{1,y}$ is the natural frequency of the bending mode (see F.2 of BS EN 1991-1-4:2005).

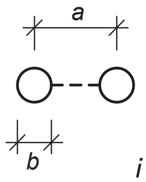
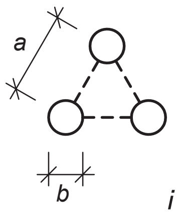
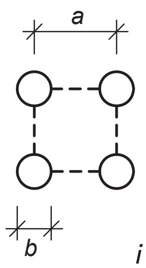
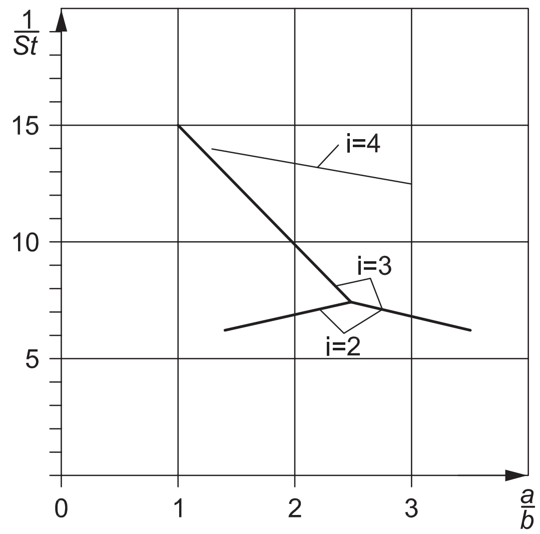
A.2.3.3 It should be ensured that:

$$v_{CG} > 1{,}25 v_m(z) \qquad \text{A.25}$$

where:

$v_m(z)$ is the mean wind velocity as defined in Equation 4.3 of BS EN 1991-1-4:2005, calculated at the height z, where the galloping excitation is expected, that is likely to be the point of maximum amplitude of oscillation.

Table A.9 **Data for the estimation of crosswind response of coupled cylinders at in-line and grouped arrangements**

	Scruton number $Sc = \frac{2\delta_s \sum m_{i,y}}{\rho b^2}$ (compare with Equation A.4)			
Coupled cylinders	$a/b = 1$	$a/b \geqslant 2$	$a/b \leqslant 1{,}5$	$a/b \geqslant 2{,}5$
 $i = 2$	$K_{iv} = 1{,}5$	$K_{iv} = 1{,}5$	$a_G = 1{,}5$	$a_G = 3{,}0$
 $i = 3$	$K_{iv} = 4{,}8$	$K_{iv} = 3{,}0$	$a_G = 6{,}0$	$a_G = 3{,}0$
 $i = 4$	$K_{iv} = 4{,}8$	$K_{iv} = 3{,}0$	$a_G = 1{,}0$	$a_G = 2{,}0$
Linear interpolation	 Reciprocal Strouhal numbers of coupled cylinders with in-line and grouped arrangements			

A.2.4 Galloping, and stall flutter, for bridge decks

A.2.4.1 Calculation of onset wind velocity

a) *Vertical motion*

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

Provided constraints a), b) and c) in **A.1.5.4.2** are satisfied v_g should be calculated from the reduced velocity v_{Rg} using Equation A.26:

$$v_g = v_{Rg} n_{b1} d_4 \qquad \text{A.26}$$

where:

$$v_{Rg} = \frac{C_g (m\delta_s)}{\rho d_4^{\,2}}$$

where:

n_{b1} is the natural fundamental frequency (in Hz) in bending as defined in **A.1.3.1**;

m and ρ are as defined in **A.1.5.4.3**;

C_g is 2,0 for bridges of type 3 and type 4 with side overhang greater than $0{,}7d_4$ or 1,0 for bridges of type 3, 3A, 4 and 4A with side overhang less than or equal to $0{,}7d_4$;

δ_s is the logarithmic decrement of damping, as specified in Annex F of BS EN 1991-1-4:2005;

d_4 is the reference depth of the bridge shown in Figure A.3, as defined in **A.1.3.2**.

Alternatively, wind tunnel tests should be undertaken to determine the value of v_g.

b) *Torsional motion*

Torsional motion should be considered for all bridge types. Provided the fascia beams and parapets conform to the constraints given in **A.1.5.4.2**, then v_g should be taken as:

$v_g = 3.3 n_{t1} b$ for bridge types 1, 1A, 2, 5 and 6; A.27

$v_g = 5 n_{t1} b$ for bridge types 3, 3A, 4 and 4A. A.28

For bridges of type 3, 3A, 4 and 4A (see Figure A.3) having $b < 4d_4$, v_g should be taken as the lesser of 12 $n_{t1} d_4$ or $5n_{t1}b$

where:

n_{t1} is the natural fundamental frequency in torsion in Hz as defined in **A.1.3.1**;

b is the total width of bridge;

d_4 is as given in Figure A.3.

A.2.4.2 Criteria to be satisfied

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

$$v_{WO} = K_{1U} K_{1A} v_m(z)\left(1 + 2I_v(z)\sqrt{B^2}\right) \qquad \text{A.29}$$

where:

K_{1U} is a factor to cover the uncertainty of prediction in this field; the default value of K_{1U} is 1,1;

K_{1A} is a coefficient selected to give an appropriate low probability of occurrence of these severe forms of oscillation; for locations in the UK, K_{1A} = 1,25.

NOTE A higher value is appropriate for other climatic regions, e.g. typically K_{1A} = 1,4 for a tropical cyclone-prone location.

$v_m(z)$ is the mean wind speed derived in accordance with BS EN 1991-1-4:2005, **4.3.1**;

$I_v(z)$ is the turbulence intensity obtained from of NA to BS EN 1991-1-4:2005, **NA.2.16**;

B^2 is the background factor defined in BS EN 1991-1-4:2005, **6.3.1**.

Where the values of v_g or v_f derived in accordance with **A.2.4.1** or **A.4.4** respectively are lower than v_{WO} further studies or wind-tunnel tests in accordance with **A.5** should be undertaken.

If the bridge cannot be assumed to be stable against galloping and stall flutter in accordance with the above criteria it should be demonstrated by means of a special investigation, or use of previous results, that the wind speed required to induce the onset of these instabilities is in excess of v_{WO}. It should be assumed that the structural damping available corresponds to the values of δ_s given in Annex F of BS EN 1991-1-4:2005.

A.3 Interference galloping of two or more free standing cylinders

A.3.1 Interference galloping is a self-excited oscillation which can occur if two or more cylinders are arranged close together without being connected with each other.

A.3.2 If the angle of wind attack is in the range of the critical wind direction β_k and if a/b < 3 (see Figure A.8), the critical wind velocity v_{CIG} may be estimated by:

$$v_{CIG} = 3,5 n_{1,y} b \sqrt{\frac{\frac{a}{b} Sc}{a_{IG}}} \qquad \text{A.30}$$

where:

Sc is the Scruton number as defined in **A.1.3.3**;

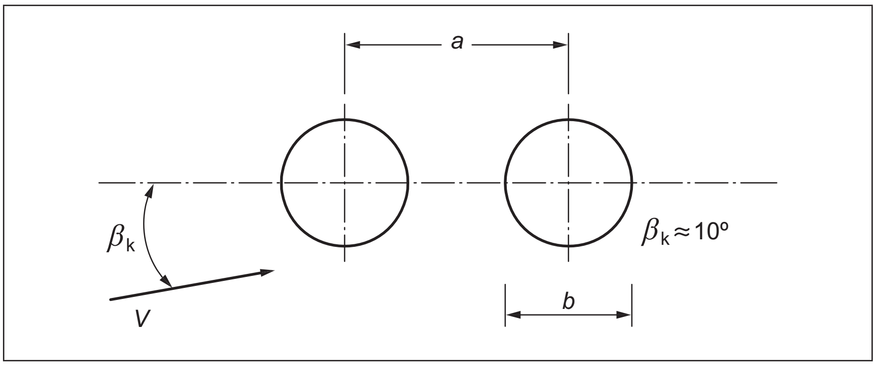
a_{IG} is the combined stability parameter a_{IG} = 3,0;

$n_{1,y}$ is the fundamental frequency of crosswind mode. Approximations are given in **F.2** of BS EN 1991-1-4:2005;

a is the spacing;

b is the diameter.

Figure A.8 **Geometric parameters for interference galloping**



A.3.3 Interference galloping may be avoided by coupling the free-standing cylinders. In that case classical galloping can occur (see **A.2.3**).

A.4 Divergence and Flutter

A.4.1 General

Divergence and flutter are instabilities that occur for flexible plate-like structures, such as signboards or suspension-bridge decks, above a certain threshold or critical wind velocity. The instability is caused by the deflection of the structure modifying the aerodynamics to alter the loading.

Divergence and flutter should be avoided.

The procedures given in this subclause provide a means of assessing the susceptibility of a structure in terms of simple structural criteria. If these criteria are not satisfied, specialist advice is recommended.

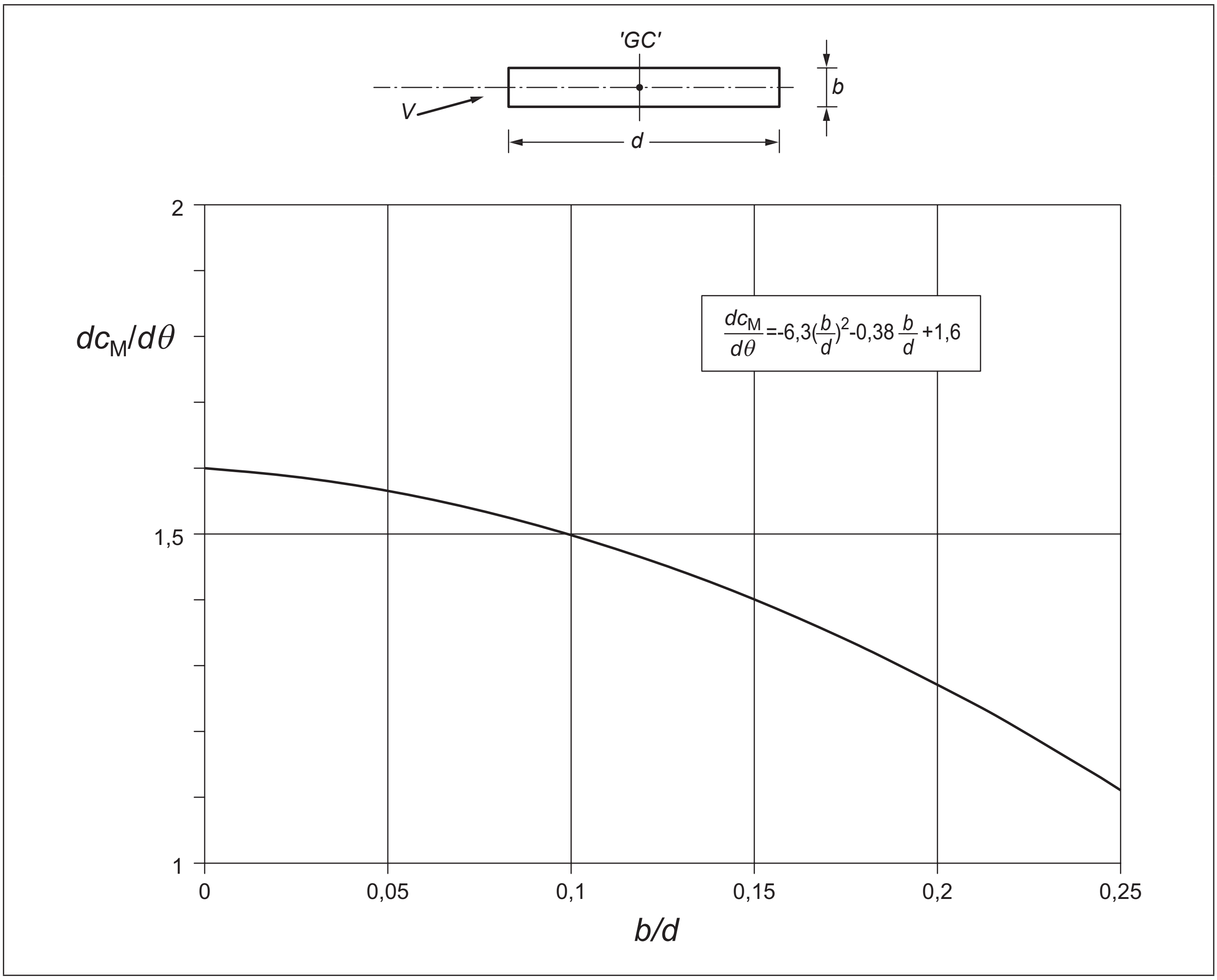
Subclause **A.4.2** provides criteria for plate-like structures and **A.4.3** a means of calculating the divergency velocity for such structures or elements. Subclause **A.4.4** provides criteria for bridge decks.

A.4.2 Criteria for plate-like structures

To be prone to either divergence or flutter, the structure satisfies all of the three criteria given below. The criteria should be checked in the order given (easiest first) and if any one of the criteria is not met, the structure will not be prone to either divergence or flutter.

- The structure, or a substantial part of it, has an elongated cross-section (like a flat plate) with b/d less than 0,25 (see Figure A.9).
- The torsional axis is parallel to the plane of the plate and normal to the wind direction, and the centre of torsion is at least $d/4$ downwind of the windward edge of the plate, where b is the inwind depth of the plate measured normal to the torsional axis. This includes the common cases of torsional centre at geometrical centre, i.e. centrally supported signboard or canopy, and torsional centre at downwind edge, i.e. cantilevered canopy.
- The lowest natural frequency corresponds to a torsional mode, or else the lowest torsional natural frequency is less than 2 times the lowest translational natural frequency.

Figure A.9 **Rate of change of aerodynamic moment coefficient dc_M/dθ with respect to geometric centre "GC" for rectangular section**



A.4.3 Divergency velocity for plate-like structures

A.4.3.1 The critical wind velocity for divergence is given in Equation A.31.

$$v_{\mathrm{div}} = \left[\frac{2k_\theta}{\rho d^2 \dfrac{\mathrm{d}c_M}{\mathrm{d}\theta}} \right] \qquad \text{A.31}$$

where:

k_θ is the torsional stiffness;

c_M is the aerodynamic moment coefficient, given in Equation A.32:

$$c_M = \frac{M}{\frac{1}{2}\rho v^2 d^2} \qquad \text{A.32}$$

dc_M/dθ is the rate of change of aerodynamic moment coefficient with respect to rotation about the torsional centre, θ expressed in radians;

ρ is the density of air (see **A.1.5.4.3**);

d is the in wind depth (chord) of the structure (see Figure A.7);

b width as defined in Figure A.9.

A.4.3.2 Values of $dc_M/d\theta$ measured about the geometric centre of various rectangular sections are given in Figure A.9.

A.4.3.3 It should be ensured that:

$$v_{d/v} > 2v_m(z_e) \qquad \text{A.34}$$

where:

$v_m(z_e)$ is the mean wind velocity as defined in Equation 4.3 of BS EN 1991-1-4:2005 at height z_e (defined in Figure 6.1 of BS EN 1991-1-4:2005).

A.4.4 Flutter of bridge decks

A.4.4.1 Calculation of onset velocity

The critical wind speed for classical flutter v_f should be calculated from the reduced critical wind speed:

$$v_{Rf} = \frac{v_f}{n_{t1}b} \qquad \text{A.34}$$

i.e. $v_f = v_{Rf} n_{t1} b$, where:

$$v_{Rf} = 1{,}8\left[1 - 1{,}1\left(\frac{n_{b1}}{n_{t1}}\right)^2\right]^{1/2}\left(\frac{mr}{\rho b^3}\right)^{1/2} \text{ but not less than } 2{,}5;$$

n_{t1}, n_{b1} are the predicted fundamental frequencies in torsion and bending (in Hz);

m, ρ and b are defined in **A.1.5.4.3**;

r is as defined in **A.1.5.4.3**.

Alternatively the value v_f may be determined by wind tunnel tests (see **A.5**).

A.4.4.2 Criteria to be satisfied

The bridge should be shown to be stable with respect to flutter up to wind speed v_{WO} given by Equation A.29.

If the bridge cannot be assumed to be stable against classical flutter in accordance with Equation A.34 it should be demonstrated by appropriate wind tunnel tests on suitable scaled models (see **A.5**) (or use of previous results), that the critical wind speed v_f for classical flutter is greater than v_{WO} (see **A.2.4.2**).

A.5 Wind tunnel testing of bridges

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

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

Where stability with respect to divergent amplitude response is established by section-model testing stability should be demonstrated up to the wind speed criterion v_{WO} (see **A.2.4.2**) given by:

$$v_{WO} = K_{1U}K_{1A}v_m(z)\left(1+2I_v(z)\sqrt{B^2}\right) \qquad \text{A.35}$$

where:

$I_v(z)$ is the turbulence intensity obtained from **NA.2.16** of NA to BS EN 1991-1-4:2005;

B^2 is the background factor defined in BS EN 1991-1-4:2005, **6.3.1**.

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

$$v_{w\alpha} = K_{1U}K_{1A}v_m(z) \qquad \text{A.36}$$

where:

$\alpha = \bar{\alpha} \pm 25I_v(z)\sqrt{B^2}$;

K_{1U}, K_{1A} are given in **A.2.4.2**.

For full-model testing, the criterion should be wind speed v_{WE} given by:

$$v_{WE} = K_{1U}K_{1A}v_m(z)\left(1+I_v(z)\sqrt{B^2}\right) \qquad \text{A.37}$$

Further guidance on wind tunnel testing is in preparation.

Annex B (informative) Along-wind response of lattice towers

B.1 General

This annex covers the along-wind gust buffeting response of freestanding lattice towers to enable BS EN 1993-3-1:2006, Annex B to be used with the UK National Annex (NA) to BS EN 1991-1-4.

The wind model in BS EN 1991-1-4 adopts five discrete terrain categories and assumes that wind speed and turbulence intensity profiles at a particular site are in equilibrium with the surrounding terrain. The NA to BS EN 1991-1-4 substitutes a transitional model where the wind speed and turbulence intensity profiles at a particular site are a function of the length and roughness of the upwind fetch and the height above ground level.

The method of calculation for along-wind gust buffeting response of freestanding towers given in BS EN 1993-3-1:2006, Annex B is compatible with the wind model in BS EN 1991-1-4. However, it is incompatible with the wind model in the NA to BS EN 1991-1-4; this annex provides a method of calculation to resolve this incompatibility.

B.2 Codification

The following substitutions for codification are made from BS EN 1993-3-1:

In BS EN 1993-3-1:2006, equation B14a):

$$\mathrm{F_{m,W}(z) = q_m} \sum c_f A_{ref} \tag{B.1}$$

where:

$$\mathrm{q_m} = 0.613 v_m^2 \tag{B.2}$$

and:

$$\mathrm{v_m} = c_\mathrm{r}(z) c_\mathrm{r,T}(z) c_\mathrm{o}(z) v_\mathrm{b} \tag{B.3}$$

with $c_r(z)$ obtained from the NA to BS EN 1991-1-4:2005, Figure NA.3 and $c_{r,T}(z)$ obtained from the NA to BS EN 1991-1-4:2005, Figure NA.4; $c_o(z)$ is the orography factor.

Thus, the proposed method in this Annex B uses the wind speed and turbulence profiles presented in the NA to BS EN 1991-1-4.

In BS EN 1993-3-1:2006, equations B.14b), B.15 and B.17:

$$\{[1+7I_\mathrm{v}(z_\mathrm{e})]c_\mathrm{s}c_\mathrm{d}-1\} = \mathrm{G_{EN}} \tag{B.4}$$

This annex gives the following modified equations:

$$\mathrm{F_{T,W}(z) = F_{m,W}(z)} \left\{1+\left[1+0,2\left(\frac{z_\mathrm{m}}{h}\right)^2\right]\mathrm{G_{EN}}\right\} \tag{B.5}$$

[BS EN 1993-3-1:2006, equation B.14b)]

$$\mathrm{S_{max}(z) = S_{m,W}(z)} \left\{1+\left[1+0,2\left(\frac{z_\mathrm{m}}{h}\right)^2\right]\mathrm{G_{EN}}\right\} \tag{B.6}$$

(BS EN 1993-3-1:2006, equation B.15)

$$\mathrm{S_{1,TW}(z) = S_{m,TW}(z)} \left[1+0,2\left(\frac{z_\mathrm{m}}{h}\right)^2\right]\mathrm{G_{EN}} \tag{B.7}$$

(BS EN 1993-3-1:2006, equation B.17)

NOTE These modified equations also remove the orography factor [$c_o(z_m)$] from the definition of G_{EN}. This is because orography accelerates the mean flow but not the turbulent part. This is accounted for correctly in BS EN 1991-1-4 and its NA, so further adjustment is not required within the definition of G_{EN}.

B.3 Methodology

The gust factor G_{EN} is defined as follows:

$$G_{EN} = (gf)(2J_a J_p I_{v,0}) \tag{B.8}$$

J_a and J_p are aerodynamic admittances, defined here for a particular height above ground level and tower of total height (H):

$$J_a^2 = \left(\frac{\int_z^H \gamma_z dz}{\int_z^H \gamma \frac{I_{v,ref}}{I_v(z)} dz} \right)^2 \tag{B.9}$$

and:

$$J_p^2 = \frac{\int_z^H \int_z^H \gamma_z \gamma_{z'} C(z - z')dzdz'}{(\int_z^H \gamma_z dz)^2} \tag{B.10}$$

where:

I_v is the turbulence intensity

z, z′ are the distance above ground of any two points

γ_z is the non-dimensional coefficient, calculated as:

$$\gamma_z = \frac{\overline{V}_z b_z \beta(z) \sigma_v(z)}{\overline{V}_o b_o \beta_o \sigma_o} \tag{B.11}$$

where:

$\overline{V}_Z$ is the 10 min mean wind speed at height z, in m/s;

b_z is the wind resistance at height z, in m^2/m;

β_z is the structural influence factor at height z;

σ_z is the turbulence at height z, in m/s.

Coefficients with a subscript of zero should be taken at the same arbitrary reference height.

The term C(z-z′) is defined as follows:

$$C(z - z') = \exp\left(\frac{-|z - z'|}{^zL_u} \right) \tag{B.12}$$

^zL_u is the length scale for the along-wind gust component (u) for vertical separation (z). It is shown in Table B.1 to Table B.7. Table B.1 should be used for sites in country terrain; Table B.2 to Table B.7 should be used for sites in town terrain. Intermediate values may be obtained by interpolation.

Values of length scale have been calculated using ESDU Data Item 86035 [13]. Values have been calculated for a basic hourly mean wind speed at 10 m above ground level in country terrain (z_0 = 0,03 m) of 23 m/s and a Coriolis factor of $11,8 \times 10^{-5}$, which are typical values for the UK.

Table B.1 **Length scale $^{z}L_{u}$ for a single roughness change from sea to country terrain, for an upwind fetch from site to sea of x(km)**

z (m)	0,1 km	0,3 km	1 km	3 km	10 km	30 km	>600 km
10	16	14	13	13	14	15	17
15	25	22	20	20	21	22	25
20	33	29	27	26	27	28	32
30	44	38	36	36	36	38	43
40	50	45	42	43	44	45	51
50	53	50	46	48	50	52	57
60	56	54	50	52	56	57	63
70	59	58	53	55	61	64	69
80	62	62	57	59	66	70	75
90	66	66	61	62	70	77	81
100	69	69	65	65	75	83	87
120	77	77	74	72	84	96	100
140	85	85	83	80	92	109	114
160	93	93	92	87	100	121	127
180	101	101	100	94	107	133	141
200	108	108	108	102	114	143	154
250	126	126	126	122	127	165	185
300	141	141	141	139	138	179	213

Values of $^{z}L_{u}$ should be calculated at an elevation midway between the top of the tower (H) and the height of interest (z).

Figure B.1 **Gust peak factor (Davenport's g)**

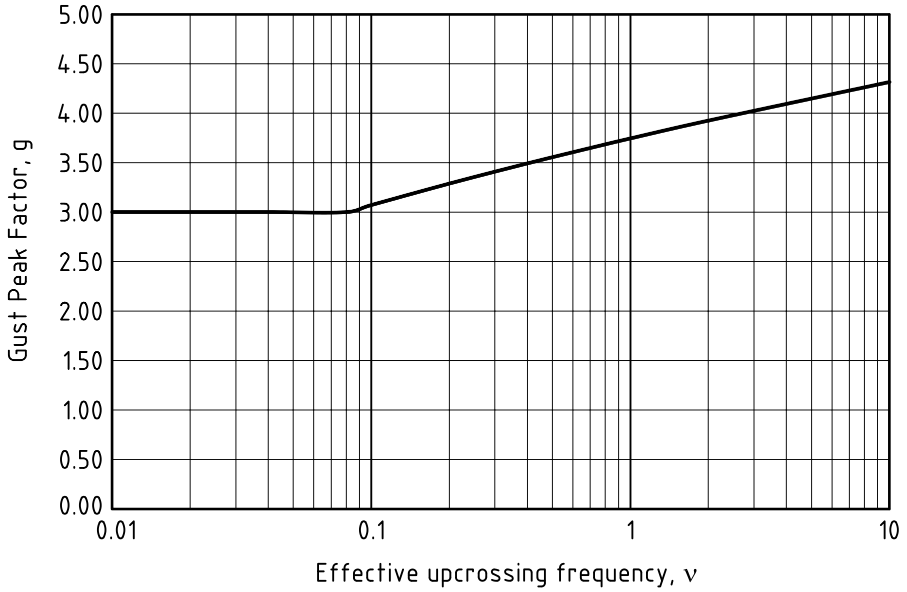


Figure B.1 shows the gust peak factor plotted for a 600 s averaging period. Conventional lattice towers have an effective up-crossing frequency in the range 0,1 to 1,0 Hz and a dynamic augmentation factor (f) for base moment and shear of about 5%. On this basis, (gf) becomes 3,6. Therefore, $G_{EN} = 3.6(2J_a J_p I_{v,0})$

The methodology proposed in this annex removes the need for a modal analysis to determine the fundamental natural frequency of the freestanding lattice tower. An allowance for dynamic augmentation is included in the calculation of G_{EN}, and this is expected to be sufficient provided the criteria given in BS EN 1993-3-1:2006, **B.3.1** are met and the guidance given in BS EN 1993-3-1:2006, **B.3.1**(2) is followed.

The integrals given in **B.3** can be rewritten as summations if the tower is subdivided into N discrete panels. Each panel of the tower has a wind resistance, in m^2, equal to $b_z \Delta z$ 1), where Δz is the vertical dimension of the panel under consideration, with a mid-height above ground level of z.

Thus:

$$J_a^2 = \left(\frac{\sum_n^N \gamma_z}{\sum_n^N \gamma_z \dfrac{I_{v,ref}}{I_v(z)}} \right)^2 \qquad \text{B.13}$$

and:

$$J_p^2 = \frac{\sum_n^N \sum_n^N \gamma_z \gamma_{z'} C(z - z')}{\left(\sum_n^N \gamma_z\right)^2} \qquad \text{B.14}$$

where:

$$\gamma_z = \frac{\overline{V}_z (b_z \Delta z) \beta(z) \sigma_v(z)}{\overline{V}_o (b_o \Delta z) \beta_o \sigma_o} \qquad \text{B.15}$$

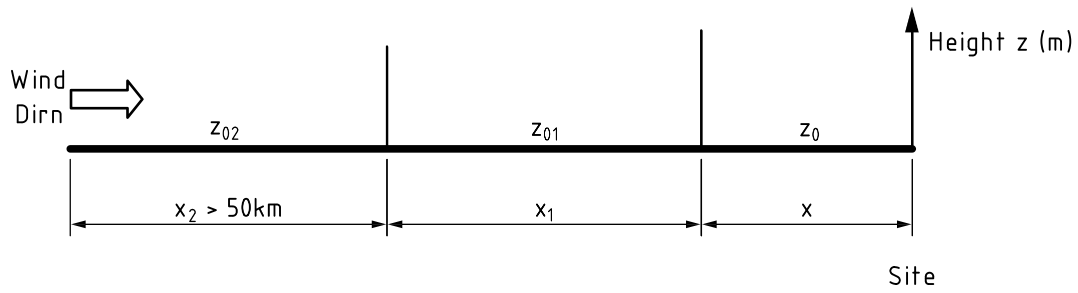
Panels are numbered sequentially from n = 1 at the base of the tower to N at the top. The summations are evaluated for the panels above the height (z) at which load effects are required.

Figure B.2 defines the parameters associated with two changes of ground roughness needed to select the appropriate length scale.

NOTE The definition of upwind fetch follows the convention in the source ESDU [13] data item which differs from the definition in the NA to BS EN 1991-1-4.

1) This is directly analogous to $\sum c_f A_{ref}$.

Figure B.2 **Definition of fetch for two roughness changes**



NOTE z_{02} = 0,003 m for sea. z_{01} = 0,03 m for country. z_0 = 0,3 m for town.

Table B.2 **Length scale for ^zL_u for two roughness changes where x_1 = 0,1 km for an upwind fetch of x km**

z (m)	0,1 km	0,3 km	1 km	3 km	10 km	30 km	>600 km
10	4	4	5	6	7	8	10
15	7	6	7	9	11	13	17
20	11	9	10	12	15	18	23
30	20	15	16	18	22	27	35
40	29	21	21	24	29	35	46
50	37	27	26	29	36	43	56
60	44	32	30	34	42	51	66
70	50	36	33	38	47	58	75
80	56	41	36	42	53	65	84
90	61	45	39	46	58	71	93
100	65	49	42	49	63	78	102
120	73	59	48	55	74	92	121
140	80	68	54	60	84	107	140
160	87	78	60	65	93	122	159
180	93	88	67	71	102	138	179
200	100	96	74	76	111	153	200
250	115	115	94	89	127	188	251
300	130	130	115	101	139	218	299

Table B.3 **Length scale for $^{z}L_u$ for two roughness changes where x_1 = 0,3 km for an upwind fetch of x km**

z (m)	0,1 km	0,3 km	1 km	3 km	10 km	30 km	>600 km
10	4	4	5	6	7	8	10
15	7	6	7	9	11	13	17
20	11	10	10	12	15	18	23
30	21	16	16	18	22	27	35
40	30	22	21	24	29	35	46
50	38	27	26	30	36	43	56
60	45	32	30	34	42	51	66
70	51	37	33	39	47	58	75
80	56	41	36	42	53	65	84
90	61	45	39	46	58	71	93
100	65	50	42	49	64	78	102
120	73	59	48	55	74	93	121
140	80	68	54	61	84	107	140
160	87	78	61	66	94	122	159
180	93	88	67	71	103	138	179
200	100	96	75	76	111	153	200
250	115	115	95	89	128	188	251
300	130	130	115	101	139	218	299

Table B.4 **Length scale for $^{z}L_u$ for two roughness changes where x_1 = 1 km for an upwind fetch of x km**

z (m)	0,1 km	0,3 km	1 km	3 km	10 km	30 km	>600 km
10	4	4	5	6	7	8	10
15	8	7	8	9	11	13	17
20	12	10	11	12	15	18	23
30	22	17	16	19	23	27	35
40	31	23	22	24	29	35	46
50	39	28	27	30	36	43	56
60	46	33	30	35	42	51	66
70	52	38	34	39	47	58	75
80	57	42	37	43	53	65	84
90	61	46	40	46	58	71	93
100	66	50	43	50	64	78	102
120	74	60	49	56	74	93	121
140	80	69	55	61	84	107	140
160	87	78	61	66	94	123	159
180	93	88	68	71	103	138	179
200	100	96	75	76	111	153	200
250	115	115	95	89	128	189	251
300	130	130	115	101	139	219	299

Table B.5 **Length scale for $^{z}L_{u}$ for two roughness changes where x_1 = 3 km for an upwind fetch of x km**

z (m)	0,1 km	0,3 km	1 km	3 km	10 km	30 km	>600 km
10	4	4	5	6	7	8	10
15	8	7	8	9	11	13	17
20	13	10	11	13	15	18	23
30	23	17	17	19	23	27	35
40	32	24	23	25	30	36	46
50	40	29	27	30	36	44	56
60	46	34	32	35	42	51	66
70	52	39	35	40	48	58	75
80	57	43	38	44	54	65	84
90	62	47	41	47	59	72	93
100	66	51	44	51	64	79	102
120	74	60	50	56	75	93	121
140	80	70	56	62	85	108	140
160	87	79	62	67	95	123	159
180	93	88	69	72	104	138	179
200	100	97	76	77	112	153	200
250	116	115	96	90	129	189	251
300	130	130	116	102	141	219	299

Table B.6 **Length scale for $^{z}L_{u}$ for two roughness changes where x_1 = 10 km for an upwind fetch of x km**

z (m)	0,1 km	0,3 km	1 km	3 km	10 km	30 km	>600 km
10	4	5	5	6	7	8	10
15	9	8	8	10	11	13	17
20	13	11	12	13	15	18	23
30	24	18	18	20	23	27	35
40	33	25	24	26	31	36	46
50	40	30	29	32	37	44	56
60	47	35	33	37	43	51	66
70	53	40	36	41	49	59	75
80	58	44	39	45	55	66	84
90	62	48	42	49	61	73	93
100	66	52	45	52	66	80	102
120	74	61	51	58	77	94	121
140	80	70	57	64	87	109	140
160	87	79	63	69	97	124	159
180	93	89	70	74	106	140	179
200	100	97	77	79	115	155	200
250	116	115	96	91	132	191	251
300	131	131	117	103	143	221	299

Table B.7 **Length scale for $^{z}L_u$ for two roughness changes where x_1 = 30 km for an upwind fetch of x km**

z (m)	0,1 km	0,3 km	1 km	3 km	10 km	30 km	>600 km
10	5	5	6	6	7	8	10
15	9	8	9	10	12	13	17
20	14	12	12	14	16	18	23
30	24	19	19	21	25	28	35
40	34	26	25	28	32	37	46
50	41	32	30	34	39	45	56
60	47	37	35	39	45	53	66
70	53	41	38	44	52	60	75
80	58	45	41	48	58	67	84
90	62	49	44	52	63	74	93
100	66	53	47	55	69	81	102
120	74	62	53	61	80	96	121
140	80	71	59	67	91	112	140
160	87	80	65	72	101	127	159
180	93	89	71	77	110	143	179
200	100	97	78	82	119	159	200
250	116	116	97	94	136	195	251
300	131	131	118	105	147	226	299

B.4 Worked examples

B.4.1 General

A series of worked examples is presented to illustrate how the methodology given in **B.3** should be applied. The worked examples cover the following cases:

a) bending moment and shear at the base of the tower (see **B.4.2** and **B.4.3**);

b) bending moment and shear at an intermediate height (see **B.4.4** and **B.4.5**);

c) a shear patch loading case, as given in BS EN 1993-3-1:2006, Figure B.3.1 for eiffelised towers (see **B.4.6**);

d) a site with significant orography (see **B.4.7**); or

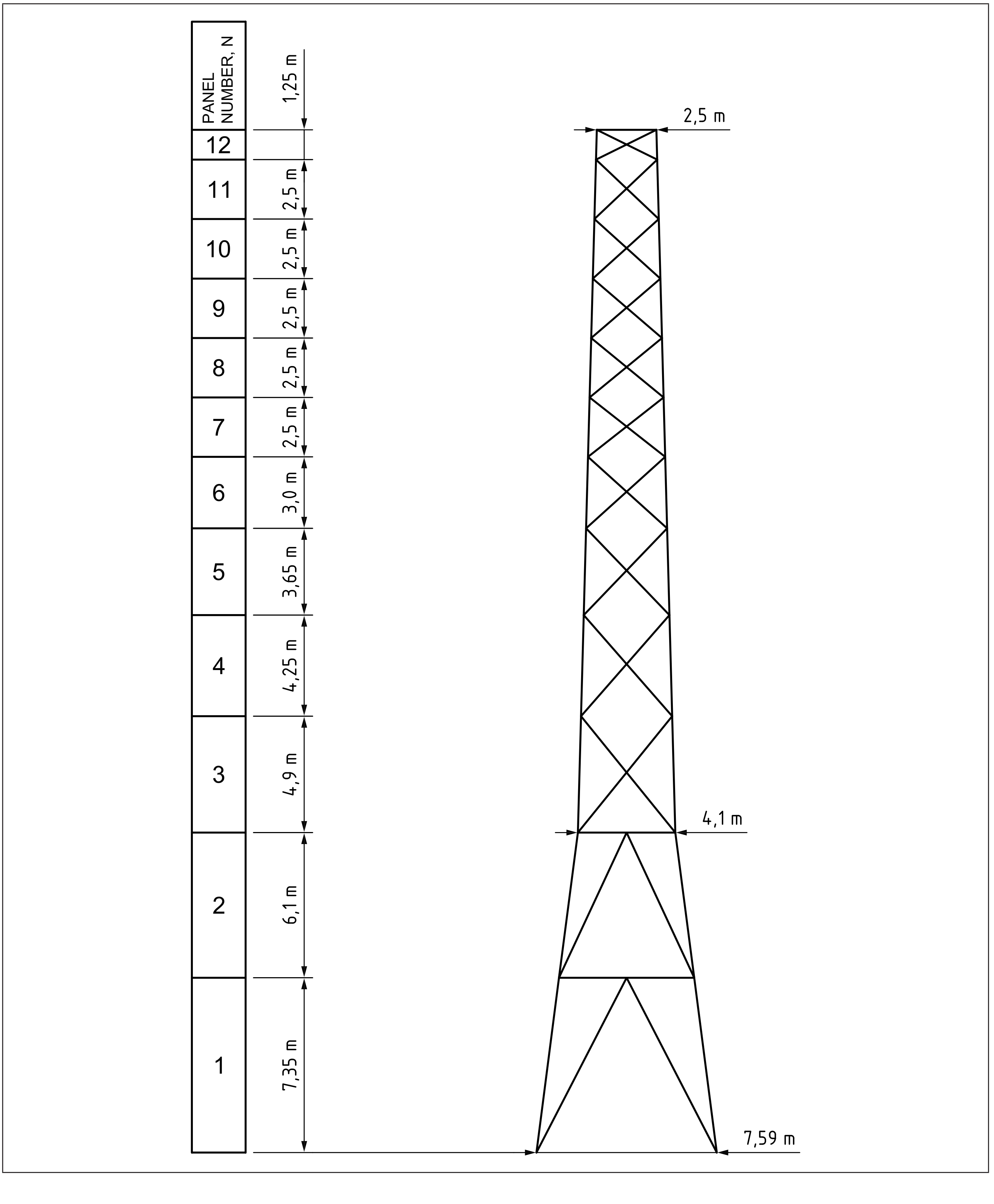
e) a tower with a large/complex attachment (see **B.4.8**).

All of the worked examples are based on the same freestanding lattice tower. Partial safety factors on load are not included.

Results are presented in terms of the global shear and bending moment at a particular level; individual member forces are not presented. If the total force in a member consists of components arising from global shear and global bending moment, each component should be factored by the appropriate gust factor, G_{EN} and partial load factor.

A fictitious square lattice tower with 12 panels has been created to facilitate the worked examples (see Figure B.3). Its key dimensions are shown in Figure B.3 and arbitrary total wind resistances have been assigned to each panel which are given in the worked examples (see **B.4.2** to **B.4.8**).

Figure B.3 **Fictitious square lattice tower with 12 panels**



NOTE The tower is symmetric so that wind resistances in the along and across-wind directions for each panel in turn are the same.

The following meteorological parameters are used for all worked examples in **B.4.2** to **B.4.8**.

The basic wind velocity (v_b) is taken as 23 m/s for a site in country terrain with an upwind fetch of 30 km. Arbitrary wind resistances have been allocated to each panel for the purposes of this example. The mean wind speed at the centre of each panel [$v_m(z)$] has been calculated as $v_m(z) = c_r(z)c_o(z)v_b$ and values of $c_r(z)$ are taken from the NA to BS EN 1991-1-4:2005, Figure NA.3.

In all examples (see **B.4.2** to **B.4.8**), it is assumed that the mean wind direction is perpendicular to one face of the tower; other wind directions should be taken into account in practice.

Values of turbulence intensity at the centre of each panel [$I_v(z)_{,flat}$] are taken from the NA to BS EN 1991-1-4:2005, Figure NA.5. Local orography only accelerates the mean wind flow, not the turbulent component of the wind; turbulence intensity is therefore defined as:

$$I_v(z) = I_v(z)_{flat}/c_o(z) \qquad \text{B.16}$$

The value at the arbitrary reference height is:

$$I_{v,0} = I_v(z_{ref})_{flat}/c_o(z_{ref}) \qquad \text{B.17}$$

The mean wind pressure at the centre of each panel is calculated as:

$$q_m(z) = 0.613 v_m^2 \qquad \text{B.18}$$

B.4.2 Bending moment at the base of the tower

There is no local orography at the site, so $c_o(z) = 1$ over the full height of the tower.

Table B.8 shows the calculation of mean wind speed, mean wind pressure and turbulence intensity at the mid-height of each panel.

Table B.8 **Meteorological parameters**

Panel	**Panel height, Δz**	**Mid-height**	**Wind resistance, $b\Delta z$**	**$c_r(z)$**	**$c_o(z)$**	**$v_m(z)$**	**$q_m(z)$**	**$I_v(z)$**
	m	m	m^2			m/s	N/m^2	
12	1,25	42,375	1,25	1,289	1	29,6	539	0,144
11	2,50	40,500	3,80	1,280	1	29,4	532	0,145
10	2,50	38,000	3,80	1,268	1	29,2	522	0,146
9	2,50	35,500	3,80	1,255	1	28,9	511	0,148
8	2,50	33,000	3,80	1,241	1	28,6	500	0,149
7	2,50	30,500	4,17	1,227	1	28,2	488	0,151
6	3,00	27,750	4,96	1,209	1	27,8	474	0,153
5	3,65	24,425	6,17	1,185	1	27,3	456	0,156
4	4,25	20,475	7,39	1,153	1	26,5	431	0,159
3	4,90	15,900	8,76	1,107	1	25,5	397	0,164
2	6,10	10,400	11,31	1,030	1	23,7	344	0,172
1	7,35	3,675	13,78	0,845	1	19,4	231	0,195

The value of turbulence at the mid-height of each panel is given by:

$$\sigma_v(z) = v_m(z)I_v(z) \qquad \text{B.19}$$

The structural influence factor [β(z)] for calculation of the bending moment is the distance (or lever arm) between the point of application of the load and the height at which the load effect is required (i.e. z = 0 m at the base of the tower). For the calculation of shear, [β(z)] = 1.

Reference values are arbitrarily taken at the centre of the highest panel, giving V_0 = 29,6 m/s, $b_0\Delta z$ = 1,25 m^2, β_0 = 42,375 m for the calculation of moment, σ_0 = 4,26 m/s and $I_{v,0}$ = 0,144. Thus, the non-dimensional coefficient, γ_z can be calculated for each panel in turn.

Table B.9 shows the calculation of the non-dimensional coefficient, γ_z and the contribution of each panel to the shear and moment at the base of the tower.

Table B.9 **Non-dimensional coefficients, wind forces and wind moments**

Panel	**σ_v**	**β(z)**	**γ(z)**	**[γ(z)$I_{v,0}$/I_v(z)]**	**q_m(z)bΔz**	**q_m(z)bΔzb**
	m/s	m			N	Nm
12	4,26	42,375	1,00	1,00	673	28 539
11	4,27	40,500	2,89	2,87	2 020	81 807
10	4,27	38,000	2,68	2,64	1 982	75 307
9	4,27	35,500	2,48	2,42	1 942	68 927
8	4,27	33,000	2,28	2,20	1 899	62 673
7	4,26	30,500	2,29	2,17	2 035	62 058
6	4,26	27,750	2,43	2,29	2 351	65 240
5	4,25	24,425	2,61	2,41	2 811	68 659
4	4,22	20,475	2,53	2,28	3 185	65 208
3	4,18	15,900	2,21	1,94	3 479	55 314
2	4,08	10,400	1,70	1,42	3 891	40 467
1	3,80	3,675	0,56	0,41	3 190	11 722
					$\sum q_m(z)b\Delta z\beta =$	685 921

Aerodynamic admittance, J_a, can now be calculated as follows:

$$\sum_{n=1}^{N=12} \gamma_z = 25,66 \qquad \text{B.20}$$

and

$$\sum_{n=1}^{N=12} \gamma_z \frac{I_{v,0}}{I_v(z)} = 24,03 \qquad \text{B.21}$$

giving

$$J_a = \frac{25.66}{24.03} = 1,068 \qquad \text{B.22}$$

Aerodynamic admittance, J_p, requires greater computational effort because of the double summation. Table B.10 shows values of γ(z)γ(z′). There are 12 panels, so there are 144 values, and the table is symmetric about the diagonal.

Table B.10 **Values of γ(z) γ(z′)**

	z′	**42,375**	**40,500**	**38,000**	**35,500**	**33,000**	**30,500**	**27,750**	**24,425**	**20,475**	**15,900**	**10,400**	**3,675**
z	**γ(z)**	**1,00**	**2,89**	**2,68**	**2,48**	**2,28**	**2,29**	**2,43**	**2,61**	**2,53**	**2,21**	**1,70**	**0,56**
42,375	**1,00**	1,00	2,89	2,68	2,48	2,28	2,29	2,43	2,61	2,53	2,21	1,70	0,56
40,500	**2,89**	2,89	8,34	7,75	7,17	6,59	6,60	7,03	7,52	7,31	6,39	4,91	1,61
38,000	**2,68**	2,68	7,75	7,21	6,67	6,13	6,14	6,53	6,99	6,79	5,94	4,56	1,50
35,500	**2,48**	2,48	7,17	6,67	6,16	5,67	5,67	6,04	6,47	6,28	5,49	4,22	1,39
33,000	**2,28**	2,28	6,59	6,13	5,67	5,21	5,21	5,55	5,95	5,77	5,05	3,88	1,27
30,500	**2,29**	2,29	6,60	6,14	5,67	6,04	5,21	5,56	5,95	5,78	5,06	3,88	1,27
27,750	**2,43**	2,43	7,03	6,53	6,04	6,47	5,55	5,92	6,34	6,16	5,38	4,14	1,36
24,475	**2,61**	2,61	7,52	6,99	6,47	6,28	5,95	6,34	6,79	6,59	5,76	4,43	1,45
20,475	**2,53**	2,53	7,31	6,79	6,28	5,49	5,77	6,16	6,59	6,40	5,60	4,30	1,41
15,900	**2,21**	2,21	6,39	5,94	5,49	4,22	5,05	5,38	5,76	5,60	4,89	3,76	1,23
10,400	**1,70**	1,70	4,91	4,56	4,22	3,88	3,88	4,14	4,43	4,30	3,76	2,89	0,95
3,675	**0,56**	0,56	1,61	1,50	1,39	1,27	1,27	1,36	1,45	1,41	1,23	0,95	0,31

Table B.11 **Values of C(z-z′)**

z, z′	42,375	40,500	38,000	35,500	33,000	30,500	27,750	24,425	20,475	15,900	10,400	3,675
42,375	1,000	0,939	0,863	0,794	0,730	0,671	0,612	0,547	0,479	0,411	0,342	0,273
40,500	0,939	1,000	0,919	0,845	0,777	0,715	0,652	0,583	0,510	0,438	0,364	0,290
38,000	0,863	0,919	1,000	0,919	0,845	0,777	0,709	0,634	0,555	0,476	0,396	0,316
35,500	0,794	0,845	0,919	1,000	0,919	0,845	0,771	0,689	0,604	0,518	0,430	0,343
33,000	0,730	0,777	0,845	0,919	1,000	0,919	0,838	0,750	0,657	0,563	0,468	0,374
30,500	0,671	0,715	0,777	0,845	0,919	1,000	0,912	0,815	0,714	0,612	0,509	0,406
27,750	0,612	0,652	0,709	0,771	0,838	0,912	1,000	0,894	0,783	0,672	0,558	0,446
24,475	0,547	0,583	0,634	0,689	0,750	0,815	0,894	1,000	0,876	0,751	0,624	0,498
20,475	0,479	0,510	0,555	0,604	0,657	0,714	0,783	0,876	1,000	0,858	0,713	0,569
15,900	0,411	0,438	0,476	0,518	0,563	0,612	0,672	0,751	0,858	1,000	0,831	0,663
10,400	0,342	0,364	0,396	0,430	0,468	0,509	0,558	0,624	0,713	0,831	1,000	0,798
3,675	0,273	0,290	0,316	0,343	0,374	0,406	0,446	0,498	0,569	0,663	0,798	1,000

Table B.12 **Values of γ(z) γ(z′) C(z-z′)**

z, z′	42,375	40,500	38,000	35,500	33,000	30,500	27,750	24,425	20,475	15,900	10,400	3,675
42,375	1,00	2,71	2,32	1,97	1,67	1,53	1,49	1,43	1,21	0,91	0,58	0,15
40,500	2,71	8,34	7,13	6,06	5,12	4,72	4,58	4,38	3,73	2,80	1,79	0,47
38,000	2,32	7,13	7,21	6,13	5,18	4,77	4,63	4,43	3,77	2,83	1,81	0,47
35,500	1,97	6,06	6,13	6,16	5,21	4,80	4,66	4,46	3,79	2,84	1,82	0,48
33,000	1,67	5,12	5,18	5,21	5,21	4,80	4,66	4,46	3,79	2,84	1,82	0,48
30,500	1,53	4,72	4,77	4,80	4,80	5,22	5,07	4,85	4,13	3,10	1,98	0,52
27,750	1,49	4,58	4,63	4,66	4,66	5,07	5,92	5,67	4,82	3,62	2,31	0,61
24,475	1,43	4,38	4,43	4,46	4,46	4,85	5,67	6,79	5,77	4,33	2,76	0,72
20,475	1,21	3,73	3,77	3,79	3,79	4,13	4,82	5,77	6,40	4,80	3,07	0,80
15,900	0,91	2,80	2,83	2,84	2,84	3,10	3,62	4,33	4,80	4,89	3,13	0,82
10,400	0,58	1,79	1,81	1,82	1,82	1,98	2,31	2,76	3,07	3,13	2,89	0,76
3,675	0,15	0,47	0,47	0,48	0,48	0,52	0,61	0,72	0,80	0,82	0,76	0,31

The value of length scale, ${}^{z}L_u$, is taken from Table B.1 for x = 30 km and a height of 21,5 m [i.e. the elevation midway between the top of the tower (H = 43 m) and the height at which load effects are required (z = 0 m)]. By interpolation, ${}^{z}L_u$ = 30 m. Therefore, a further Table of 144 values can be populated giving C(z–z′) and is shown in Table B.11.

Table B.10 and Table B.11 are now combined to give Table B.12.

Aerodynamic admittance, J_p, can now be calculated as follows:

$$\sum_{n=1}^{N=12}\sum_{n=1}^{N=12}\gamma_z\gamma_{z'}C(z-z')=471 \qquad \text{B.23}$$

and

$$\left(\sum_{n=1}^{N=12}\gamma_z\right)^2=25{,}7^2=660 \qquad \text{B.24}$$

giving

$$J_p=\sqrt{\frac{471}{660}}=0{,}845 \qquad \text{B.25}$$

Therefore,

$$G_{EN}=3{,}6(2J_aJ_pI_{v,0})=3{,}6(2\times1{,}068\times0{,}845\times0{,}144)=0{,}935 \qquad \text{B.26}$$

The bending moment at the base of the tower under the 10 min mean wind speed is $\sum q_m(z)b\Delta z\beta$ and this equals 686 kNm. The total bending moment at the base of the tower under the effect of mean and gust wind loading is given by:

$$F_{T,W}(z) = F_{m,W}(z)\{1 + [1 + 0{,}2(z / h)^2]G_{EN} \qquad \text{B.27}$$

Thus, the total bending moment at the base of the tower = 1 327 kNm =686×[1+(1) × 0,935].

Note the factor of $\left\{1+\left[1+0{,}2(z/h)^2\right]G_{EN}\right\}$ is only applied to the load effect, $F_{m,W}(z)$, once it has been summed for all the panels above the height of interest (i.e. z = 0 m in this example). This applies generally except when the tower supports large ancillaries. When calculating a gust factor for a large ancillary, z should be taken as the height of the ancillary (see **B.4.8**).

B.4.3 Shear at the base of the tower

The example in **B.4.2** is repeated but with the structural influence factor, β, set to unity. Only those elements of the calculation that have changed are presented. β_0 = 1 for the calculation of shear.

Table B.13 shows the calculation of the non-dimensional coefficient, γ_z and the contribution of each panel to the shear at the base of the tower.

Table B.13 **Non-dimensional coefficients and wind forces**

Panel	σ_v m/s	$\beta(z)$	$\gamma(z)$	$[\gamma(z)I_{v,0}/I_v(z)]$	$q_m(z)b\Delta z\beta$
12	4,26	1	1,00	1,00	673
11	4,27	1	3,02	3,00	2 020
10	4,27	1	2,99	2,94	1 982
9	4,27	1	2,96	2,88	1 942
8	4,27	1	2,93	2,82	1 899
7	4,26	1	3,17	3,02	2 035
6	4,26	1	3,72	3,49	2 351
5	4,25	1	4,52	4,17	2 811
4	4,22	1	5,24	4,73	3 185
3	4,18	1	5,90	5,17	3 479
2	4,08	1	6,93	5,78	3 891
1	3,80	1	6,43	4,74	3 190
					$\Sigma q_m(z)b\Delta z\beta$ = 29 457

Aerodynamic admittance, J_a, can now be calculated as follows:

$$\sum_{n=1}^{N=12} \gamma_z = 48,81 \qquad \text{B.28}$$

and

$$\sum_{n=1}^{N=12} \gamma_z \frac{I_{v,0}}{I_v(z)} = 43,74 \qquad \text{B.29}$$

giving

$$J_a = \frac{48,81}{43,74} = 1,116 \qquad \text{B.30}$$

Table B.14 and Table B.15 show those parts of the calculation that have changed from **B.4.2** by calculating shear at the base of the tower rather than bending moment.

Table B.14 **Values of γ(z) γ(z′)**

	z′	**42,375**	**40,500**	**38,000**	**35,500**	**33,000**	**30,500**	**27,750**	**24,425**	**20,475**	**15,900**	**10,400**	**3,675**
z	**γ(z)**	**1,00**	**3,02**	**2,99**	**2,96**	**2,93**	**3,17**	**3,72**	**4,52**	**5,24**	**5,90**	**6,93**	**6,43**
42,375	**1,00**	1,00	3,02	2,99	2,96	2,93	3,17	3,72	4,52	5,24	5,90	6,93	6,43
40,500	**3,02**	3,02	9,13	9,05	8,95	8,85	9,59	11,23	13,65	15,82	17,81	20,92	19,44
38,000	**2,99**	2,99	9,05	8,96	8,87	8,77	9,51	11,13	13,53	15,68	17,65	20,73	19,26
35,500	**2,96**	2,96	8,95	8,87	8,78	8,68	9,41	11,02	13,40	15,52	17,47	20,53	19,07
33,000	**2,93**	2,93	8,85	8,77	8,68	8,59	9,30	10,89	13,24	15,34	17,28	20,29	18,85
30,500	**3,17**	3,17	9,59	9,51	9,41	9,30	10,08	11,80	14,35	16,62	18,72	21,99	20,43
27,750	**3,72**	3,72	11,23	11,13	11,02	10,89	11,80	13,81	16,80	19,46	21,91	25,74	23,91
24,475	**4,52**	4,52	13,65	13,53	13,40	13,24	14,35	16,80	20,43	23,67	26,65	31,30	29,08
20,475	**5,24**	5,24	15,82	15,68	15,52	15,34	16,62	19,46	23,67	27,42	30,87	36,26	33,68
15,900	**5,90**	5,90	17,81	17,65	17,47	17,28	18,72	21,91	26,65	30,87	34,77	40,84	37,93
10,400	**6,93**	6,93	20,92	20,73	20,53	20,29	21,99	25,74	31,30	36,26	40,84	47,96	44,55
3,675	**6,43**	6,43	19,44	19,26	19,07	18,85	20,43	23,91	29,08	33,68	37,93	44,55	41,39

Table B.15 **Values of γ(z) γ(z′) C(z-z′)**

z, z′	**42,375**	**40,500**	**38,000**	**35,500**	**33,000**	**30,500**	**27,750**	**24,425**	**20,475**	**15,900**	**10,400**	**3,675**
42,375	1,00	2,84	2,58	2,35	2,14	2,13	2,27	2,47	2,51	2,42	2,37	1,75
40,500	2,84	9,13	8,32	7,57	6,88	6,86	7,32	7,96	8,08	7,80	7,62	5,64
38,000	2,58	8,32	8,96	8,16	7,42	7,39	7,89	8,58	8,70	8,40	8,21	6,08
35,500	2,35	7,57	8,16	8,78	7,99	7,96	8,49	9,24	9,37	9,05	8,84	6,55
33,000	2,14	6,88	7,42	7,99	8,59	8,55	9,13	9,93	10,08	9,73	9,50	7,04
30,500	2,13	6,86	7,39	7,96	8,55	10,08	10,76	11,70	11,87	11,47	11,20	8,30
27,750	2,27	7,32	7,89	8,49	9,13	10,76	13,81	15,02	15,24	14,72	14,37	10,65
24,475	2,47	7,96	8,58	9,24	9,93	11,70	15,02	20,43	20,73	20,02	19,55	14,49
20,475	2,51	8,08	8,70	9,37	10,08	11,87	15,24	20,73	27,42	26,48	25,85	19,16
15,900	2,42	7,80	8,40	9,05	9,73	11,47	14,72	20,02	26,48	34,77	33,95	25,16
10,400	2,37	7,62	8,21	8,84	9,50	11,20	14,37	19,55	25,85	33,95	47,96	35,55
3,675	1,75	5,64	6,08	6,55	7,04	8,30	10,65	14,49	19,16	25,16	35,55	41,39

Aerodynamic admittance, J_p, can now be calculated as follows:

$$\sum_{n=1}^{N=12}\sum_{n=1}^{N=12}\gamma_z\gamma_{z'}C(z-z')=1589 \qquad \text{B.31}$$

and

$$\left(\sum_{n=1}^{N=12}\gamma_z\right)^2=48,8^2=2381 \qquad \text{B.32}$$

giving

$$J_p=\sqrt{\frac{1589}{2381}}=0,817 \qquad \text{B.33}$$

Therefore,

$$G_{EN}=3,6(2J_aJ_pI_{v,0})=3,6(2\times1,116\times0,817\times0,144)=0,945 \qquad \text{B.34}$$

The shear force at the base of the tower under the 10 min mean wind speed is $\sum q_m(z)b\Delta z\beta$ and this equals 29,5 kN. The total shear force at the base of the tower under the effect of mean and gust wind loading is given by:

$$F_{T,W}(z) = F_{m,W}(z)\{1 + [1 + 0,2(z / h)^2]G_{EN}\} \qquad \text{B.35}$$

Thus, the total shear force at the base of the tower is (29,5)[1+(1×0,945)] = 57,4kN.

B.4.4 Bending moment at an intermediate height

The example shown in **B.4.2** is repeated but with bending moment calculated at the intermediate height of 29,25 m. Only those elements of the calculation that have changed are presented.

Table B.16 shows the calculation of the non-dimensional coefficient, γ_z and the contribution of each panel to the shear and moment at the intermediate height.

Table B.16 **Non-dimensional coefficients, wind forces and moments**

Panel	σ_v	$\beta(z)$	$\gamma(z)$	$[\gamma(z)I_{v,0}/I_v(z)]$	$q_m(z)\beta\Delta z$	$q_m(z)b\Delta z\beta$
	m/s	m			N	Nm
12	4,26	13,125	1,00	1,00	673	8 839
11	4,27	11,250	2,59	2,57	2 020	22 724
10	4,27	8,750	2,00	1,96	1 982	17 340
9	4,27	6,250	1,41	1,37	1 942	12 135
8	4,27	3,750	0,84	0,81	1 899	7 122
7	4,26	1,250	0,30	0,29	2 035	2 543
6	4,26	0	0	0	2 351	0
5	4,25	0	0	0	2 811	0
4	4,22	0	0	0	3 185	0
3	4,18	0	0	0	3 479	0
2	4,08	0	0	0	3 891	0
1	3,80	0	0	0	3 190	0
						$\sum q_m(z)b\Delta z\beta=$ 70 704

Aerodynamic admittance, J_a, can now be calculated as follows:

$$\sum_{n=7}^{n=12} \gamma_z = 8,14 \qquad \text{B.36}$$

and

$$\sum_{n=7}^{N=12} \gamma_z \frac{I_{v,0}}{I_{v(z)}} = 8,00 \qquad \text{B.37}$$

giving

$$J_a = \frac{8,14}{8,00} = 1,017 \qquad \text{B.38}$$

Table B.17, Table B.18 and Table B.19 show those parts of the calculation that have changed from **B.4.2** by calculating bending moment at an intermediate height rather than at the base of the tower.

Table B.17 **Values of γ(z) γ(z′)**

	z′	**42,375**	**40,500**	**38,000**	**35,500**	**33,000**	**30,500**	**27,750**	**24,425**	**20,475**	**15,900**	**10,400**	**3,675**
z	**γ(z)**	**1,00**	**2,59**	**2,00**	**1,41**	**0,84**	**0,30**	**0**	**0**	**0**	**0**	**0**	**0**
42,375	**1,00**	1,00	2,59	2,00	1,41	0,84	0,30	0	0	0	0	0	0
40,500	**2,59**	2,59	6,71	5,17	3,65	2,17	0,78	0	0	0	0	0	0
38,000	**2,00**	2,00	5,17	3,98	2,82	1,67	0,60	0	0	0	0	0	0
35,500	**1,41**	1,41	3,65	2,82	1,99	1,18	0,43	0	0	0	0	0	0
33,000	**0,84**	0,84	2,17	1,67	1,18	0,70	0,25	0	0	0	0	0	0
30,500	**0,30**	0,30	0,78	0,60	0,43	0,25	0,09	0	0	0	0	0	0
27,750	**0**	0	0	0	0	0	0	0	0	0	0	0	0
24,475	**0**	0	0	0	0	0	0	0	0	0	0	0	0
20,475	**0**	0	0	0	0	0	0	0	0	0	0	0	0
15,900	**0**	0	0	0	0	0	0	0	0	0	0	0	0
10,400	**0**	0	0	0	0	0	0	0	0	0	0	0	0
3,675	**0**	0	0	0	0	0	0	0	0	0	0	0	0

Table B.18 **Values of C(z-z′)**

z, z′	**42,375**	**40,500**	**38,000**	**35,500**	**33,000**	**30,500**	**27,750**	**24,425**	**20,475**	**15,900**	**10,400**	**3,675**
42,375	1,000	0,957	0,902	0,850	0,801	0,755	0,707	0,654	0,595	0,534	0,469	0,400
40,500	0,957	1,000	0,942	0,888	0,837	0,789	0,739	0,683	0,622	0,558	0,490	0,418
38,000	0,902	0,942	1,000	0,942	0,888	0,837	0,784	0,725	0,660	0,592	0,520	0,443
35,500	0,850	0,888	0,942	1,000	0,942	0,888	0,832	0,769	0,700	0,629	0,552	0,470
33,000	0,801	0,837	0,888	0,942	1,000	0,942	0,883	0,816	0,743	0,667	0,585	0,499
30,500	0,755	0,789	0,837	0,888	0,942	1,000	0,937	0,866	0,789	0,708	0,621	0,530
27,750	0,707	0,739	0,784	0,832	0,888	0,937	1,000	0,924	0,842	0,755	0,663	0,565
24,475	0,654	0,683	0,725	0,769	0,816	0,866	0,924	1,000	0,911	0,817	0,717	0,612
20,475	0,595	0,622	0,660	0,700	0,743	0,789	0,842	0,911	1,000	0,897	0,788	0,672
15,900	0,534	0,558	0,592	0,629	0,667	0,708	0,755	0,817	0,897	1,000	0,878	0,749
10,400	0,469	0,490	0,520	0,552	0,584	0,621	0,663	0,717	0,788	0,878	1,000	0,853
3,675	0,400	0,418	0,443	0,470	0,499	0,530	0,565	0,612	0,672	0,749	0,853	1,000

Table B.19 **Values of γ(z) γ(z′) C(z-z′)**

z, z′	**42,375**	**40,500**	**38,000**	**35,500**	**33,000**	**30,500**	**27,750**	**24,425**	**20,475**	**15,900**	**10,400**	**3,675**
42,375	1,00	2,48	1,80	1,20	0,67	0,23	0	0	0	0	0	0
40,500	2,48	6,71	4,87	3,25	1,82	0,62	0	0	0	0	0	0
38,000	1,80	4,87	3,98	2,65	1,48	0,51	0	0	0	0	0	0
35,500	1,20	3,25	2,65	1,99	1,11	0,38	0	0	0	0	0	0
33,000	0,67	1,82	1,48	1,11	0,70	0,24	0	0	0	0	0	0
30,500	0,23	0,62	0,51	0,38	0,24	0,09	0	0	0	0	0	0
27,750	0	0	0	0	0	0	0	0	0	0	0	0
24,425	0	0	0	0	0	0	0	0	0	0	0	0
20,475	0	0	0	0	0	0	0	0	0	0	0	0
15,900	0	0	0	0	0	0	0	0	0	0	0	0
10,400	0	0	0	0	0	0	0	0	0	0	0	0
3,675	0	0	0	0	0	0	0	0	0	0	0	0

The value of length scale, ${}^{z}L_u$, is taken from Table B.1 for x = 30 km and a height of 36 m, i.e. the elevation midway between the top of the tower (H = 43 m) and the height at which load effects are required (z = 29,25 m). By interpolation, ${}^{z}L_u$ = 42 m. Therefore, a further Table of 144 values can be populated giving C(z–z') and is shown in Table B.18.

Table B.17 and Table B.18 are now combined to give Table B.19.

Aerodynamic admittance, J_p, can now be calculated as follows:

$$\sum_{n=7}^{N=12}\sum_{n=7}^{N=12}\gamma_z\gamma_{z'}C(z-z')=61{,}1 \qquad \text{B.39}$$

and

$$\left(\sum_{n=7}^{N=12}\gamma_z\right)^2=8{,}14^2=66{,}2 \qquad \text{B.40}$$

giving

$$J_p=\sqrt{\frac{61{,}1}{66{,}2}}=0{,}961 \qquad \text{B.41}$$

Therefore,

$$G_{EN}=3{,}6(2J_aJ_pI_{v,0})=3{,}6(2\times1{,}017\times0{,}961\times0{,}144)=1{,}012 \qquad \text{B.42}$$

The bending moment at the base of panel 7 (z = 29,25 m) under the 10 min mean wind speed is $\sum q_m(z)b\Delta z\beta$ and this equals 70,7 kNm. The total bending moment at the base of panel 7 under the effect of mean and gust wind loading is given by:

$$F_{T,W}(z) = F_{m,W}(z)\ \{1 + [1 + 0{,}2(z / h)^2]G_{EN}\} \qquad \text{B.43}$$

Thus, the total bending moment at the base of panel 7 (z = 29,25 m) = 149 kNm = 70,7×{1+[(1,093)×1,012]}.

B.4.5 Shear at an intermediate height

The example shown in **B.4.3** is repeated but with the structural influence factor, β, set to unity. β_0 = 1 for the calculation of shear. Shear force is calculated at the intermediate height of 29,25 m; this is the height of the only intersection point within the height of the tower so patch loading cases do not need to be considered. Only those elements of the calculation that have changed are presented.

Table B.20 shows the calculation of non-dimensional coefficient, γ_z and the contribution of each panel to the shear at the intermediate height.

Table B.20 **Non-dimensional coefficients and wind forces**

Panel	σ_v m/s	$\beta(z)$	$\gamma(z)$	$[\gamma(z)I_{v,0}/I_v(z)]$	$q_m(z)b\Delta z\beta$ N
12	4,26	1	1,00	1,00	673
11	4,27	1	3,02	3,00	2 020
10	4,27	1	2,99	2,94	1 982
9	4,27	1	2,96	2,88	1 942
8	4,27	1	2,93	2,82	1 899
7	4,26	1	3,17	3,02	2 035
6	4,26	0	0,00	0,00	0
5	4,25	0	0,00	0,00	0
4	4,22	0	0,00	0,00	0
3	4,18	0	0,00	0,00	0
2	4,08	0	0,00	0,00	0
1	3,80	0	0,00	0,00	0
					$\sum q_m(z)b\Delta z\beta$ = 10 551

Aerodynamic admittance, J_a, can now be calculated as follows:

$$\sum_{n=7}^{N=12} \gamma_z = 16,08 \qquad \text{B.44}$$

and

$$\sum_{n=7}^{N=12} \gamma_z \frac{I_{v,0}}{I_v(z)} = 15,67 \qquad \text{B.45}$$

giving

$$J_a = \frac{16,08}{15,67} = 1,026 \qquad \text{B.46}$$

Table B.21 and Table B.22 show those parts of the calculation that have changed from **B.4.3** by calculating shear at an intermediate height rather than at the base of the tower.

Table B.21 **Values of $\gamma(z)\,\gamma(z')$**

	z′	**42,375**	**40,500**	**38,000**	**35,500**	**33,000**	**30,500**	**27,750**	**24,425**	**20,475**	**15,900**	**10,400**	**3,675**
z	**γ(z)**	**1,00**	**3,02**	**2,99**	**2,96**	**2,93**	**3,17**	**0,00**	**0,00**	**0,00**	**0,00**	**0,00**	**0,00**
42,375	**1,00**	1,00	3,02	2,99	2,96	2,93	3,17	0	0	0	0	0	0
40,500	**3,02**	3,02	9,13	9,05	8,95	8,85	9,59	0	0	0	0	0	0
38,000	**2,99**	2,99	9,05	8,96	8,87	8,77	9,51	0	0	0	0	0	0
35,500	**2,96**	2,96	8,95	8,87	8,78	8,68	9,41	0	0	0	0	0	0
33,000	**2,93**	2,93	8,85	8,77	8,68	8,59	9,30	0	0	0	0	0	0
30,500	**3,17**	3,17	9,59	9,51	9,41	9,30	10,08	0	0	0	0	0	0
27,750	**0,00**	0	0	0	0	0	0	0	0	0	0	0	0
24,425	**0,00**	0	0	0	0	0	0	0	0	0	0	0	0
20,475	**0,00**	0	0	0	0	0	0	0	0	0	0	0	0
15,900	**0,00**	0	0	0	0	0	0	0	0	0	0	0	0
10,400	**0,00**	0	0	0	0	0	0	0	0	0	0	0	0
3,675	**0,00**	0	0	0	0	0	0	0	0	0	0	0	0

Table B.22 **Values of $\gamma(z)\,\gamma(z')\,C(z-z')$**

z, z′	**42,375**	**40,500**	**38,000**	**35,500**	**33,000**	**30,500**	**27,750**	**24,425**	**20,475**	**15,900**	**10,400**	**3,675**
42,375	1,00	2,89	2,70	2,52	2,35	2,40	0	0	0	0	0	0
40,500	2,89	9,13	8,52	7,95	7,41	7,57	0	0	0	0	0	0
38,000	2,70	8,52	8,96	8,36	7,79	7,96	0	0	0	0	0	0
35,500	2,52	7,95	8,36	8,78	8,19	8,36	0	0	0	0	0	0
33,000	2,35	7,41	7,79	8,19	8,59	8,77	0	0	0	0	0	0
30,500	2,40	7,57	7,96	8,36	8,77	10,08	0	0	0	0	0	0
27,750	0	0	0	0	0	0	0	0	0	0	0	0
24,425	0	0	0	0	0	0	0	0	0	0	0	0
20,475	0	0	0	0	0	0	0	0	0	0	0	0
15,900	0	0	0	0	0	0	0	0	0	0	0	0
10,400	0	0	0	0	0	0	0	0	0	0	0	0
3,675	0	0	0	0	0	0	0	0	0	0	0	0

Aerodynamic admittance, J_p, can now be calculated as follows:

$$\sum_{n=7}^{N=12}\sum_{n=7}^{N=12}\gamma_z\gamma_{z'}C(z-z')=234 \quad \text{B.47}$$

and

$$\left(\sum_{N=7}^{N=12}\gamma_z\right)^2=16,08^2=259 \quad \text{B.48}$$

giving

$$J_p=\sqrt{\frac{234}{259}}=0,951 \quad \text{B.49}$$

Therefore,

$$G_{EN}=3,6(2J_aJ_pI_{v,0})=3,6(2\times1,026\times0,951\times0,144)=1,012 \quad \text{B.50}$$

The shear force at the base of panel 7 (z = 29,25 m) under the 10 min mean wind speed is $\sum q_m(z)b\Delta z\beta$ and this equals 10,55 kN. The total shear force at the base of panel 7 under the effect of mean and gust wind loading is given by:

$$F_{T,W}(z) = F_{m,W}(z)\ \{1 + [1 + 0,2(z / h)^2]G_{EN}\} \quad \text{B.51}$$

Thus, the total shear force at the base of panel 7 = (10,55){1+[(1,09)$\times$1,012]} = 22,2 kN.

B.4.6 A shear patch loading case, as given in BS EN 1993-3-1:2006, Figure B.3.1 for an eiffelised tower

A tower is categorized as eiffelised when the projection of inclined leg members meet at an intersection point below the top of the tower. The tower created for this example has a single intersection point at an elevation of z = 29,25 m, which corresponds to the generic case given in BS EN 1993-3-1:2006, Figure B.3.1, Case 2.

It can be shown by statics that a single horizontal load applied at the level of the intersection point does not give rise to load effects in the bracing members; the load is carried entirely by axial load effects in the leg members. It follows that horizontal forces applied above and below the intersection point generate moments about the intersection point, but in opposite senses. In turn, this results in load-effects in the bracing members.

An influence line can be drawn for the load effect in the bracing members in any panel below the intersection point. Contributions to the load effect in the bracing member from wind loading above and below the intersection point are in the opposite sense. Therefore, application of a gust wind profile over the full height of the tower above the panel of interest leads to an unconservative answer; positive and negative areas counteract. In practice, the tower might experience non-simultaneous loading from wind gusts of different sizes and at different heights. BS EN 1993-3-1 addresses this issue by applying mean wind loading over the full height of the tower and gust wind loading over the positive and negative areas of the influence line (i.e. above and below the intersection point) in turn.

The axial member force in the diagonal bracing in panel 1 should be calculated for a mean wind profile between z = 0 m and the intersection point at z = 29,25 m, and a mean plus gust wind profile above the intersection point. The calculation process should then be repeated for mean plus gust wind loading between z = 0 m and the intersection point at z = 29,25 m, and mean wind loading above the intersection point. Gust factors should be calculated for the sections of the tower over which gust loading is applied.

Table B.23 shows the calculation of the non-dimensional coefficient, γ_z and the contribution of each panel to the shear above and below the intersection point.

Table B.23 **Non-dimensional coefficients and wind forces**

Panel	σ_v m/s	$\beta(z)$	$\gamma(z)$	$[\gamma(z)I_{v,0}/I_v(z)]$	$q_m(z)b\Delta z\beta$ N
12	4,26	1	1,00	1,00	673
11	4,27	1	3,02	3,00	2 020
10	4,27	1	2,99	2,94	1 982
9	4,27	1	2,96	2,88	1 942
8	4,27	1	2,93	2,82	1 899
7	4,26	1	3,17	3,02	2 035
6	4,26	1	3,72	3,49	2 351
5	4,25	1	4,52	4,17	2 811
4	4,22	1	5,24	4,73	3 185
3	4,18	1	5,90	5,17	3 479
2	4,08	1	6,93	5,78	3 891
1	3,80	1	6,43	4,74	3 190

Figure B.4 shows the geometry of the tower, the mean wind load on each panel, the elevation of the mid-height of each panel and the derivation of lever arm to calculate the axial force in the diagonal bracing members in the base panel. Figure B.4 and the following calculations are included for illustrative purposes only; in practice member forces should be calculated by linear elastic stiffness matrix analysis under the appropriate set of applied wind loads.

Figure B.4 **Illustration of parameters for shear patch loading**

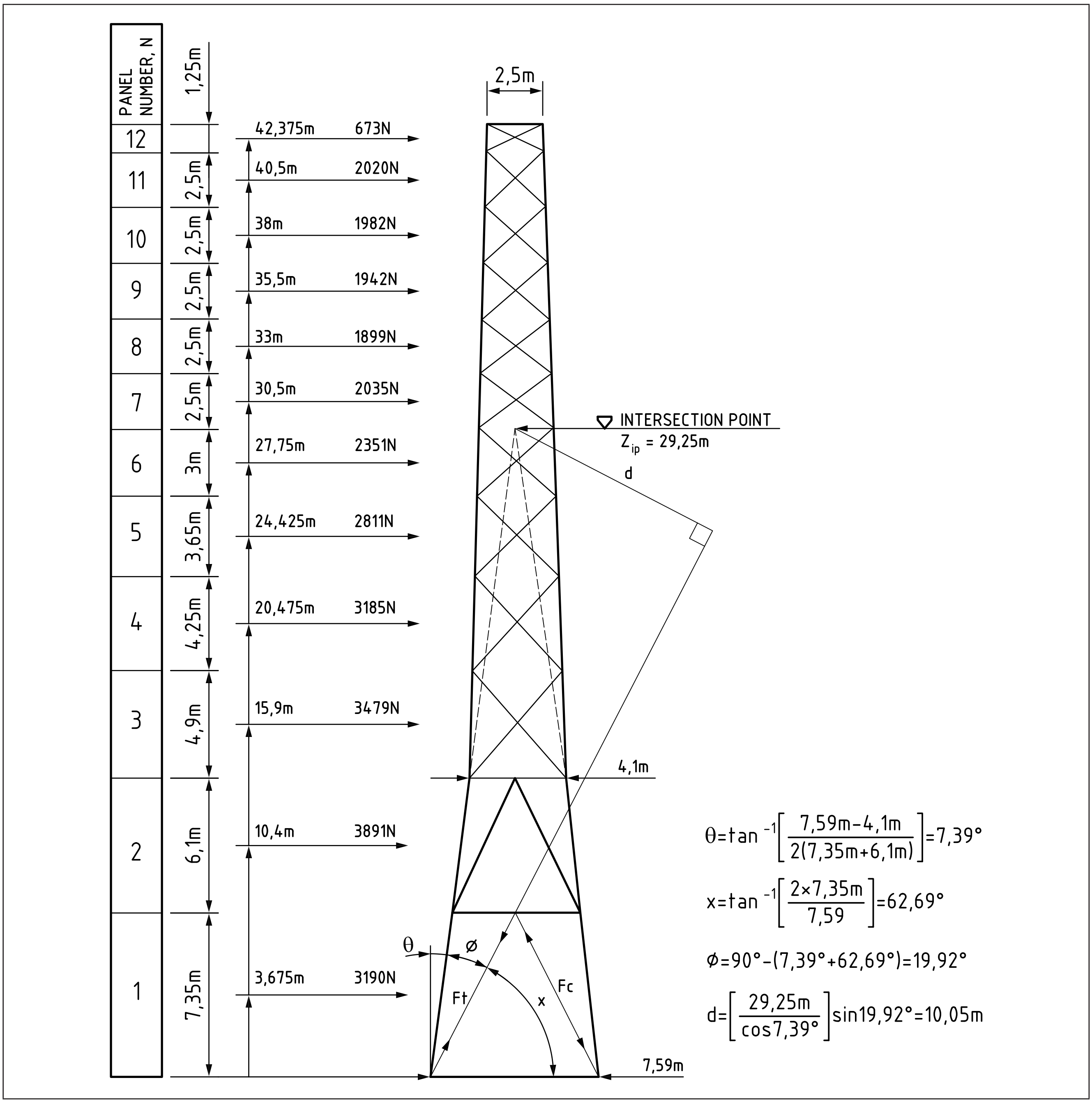


Table B.24 and Table B.25 show the calculation of moment about the intersection point, z_{ip}.

Table B.24 **Lever arms, wind loads and moments above z_{ip}**

Panel	Panel height	Elevation at panel base	Mid panel height	Lever arm	Mean wind load	Moment about z_{ip}
	m	m	m	m	N	Nm
Tower top	-	43,00	-	-	-	-
12	1,25	41,75	42,375	13,125	673	8 839
11	2,50	39,25	40,500	11,250	2 020	22 724
10	2,50	36,75	38,000	8,750	1 982	17 340
9	2,50	34,25	35,500	6,250	1 942	12 135
8	2,50	31,75	33,000	3,750	1 899	7 122
7	2,50	29,25	30,500	1,250	2 035	2 543
						$\sum$ (moment above z_{ip}) = 70 704

NOTE Intersection point height, z_{ip} = 29,25 m.

Table B.25 **Lever arms, wind loads and moments below z_{ip}**

Panel	Panel height	Elevation at panel base	Mid panel height	Lever arm	Mean wind load	Moment about z_{ip}
	m	m	m	m	N	Nm
6	3,000	26,250	27,750	–1,500	2 351	–3 526
5	3,650	22,600	24,425	–4,825	2 811	–13 563
4	4,250	18,350	20,475	–8,775	3 185	–27 946
3	4,900	13,450	15,900	–13,350	3 479	–46 443
2	6,100	7,350	10,400	–18,850	3 891	–73 346
1	7,350	0	3,675	–25,575	3 190	–81 577
						$\sum$ (moment below z_{ip}) = –246 401

NOTE Intersection point height, z_{ip} = 29,25 m.

The axial force in the diagonal bracing member is obtained by dividing the moment calculated about the intersection point by the lever arm d shown in Figure B.4 and the number of diagonal bracing members.

NOTE 1 For wind perpendicular to the face of the square tower, there are two diagonal bracing members in each of the two faces parallel to the wind direction.

Therefore, the member force due to mean wind load applied above the intersection point = 70 704 Nm / 10,05 m / 4 = 1 759 N; for mean wind load applied below the intersection point, the member force = –246 401 Nm / 10,05 m / 4 = –6 129 N.

For convenience, this example has been configured so that the base of panel 7 and the intersection point coincide. Thus, the gust factor (G_{EN}) for gust loading above the intersection point can be taken directly from **B.4.5** (as this is for gust loading over the same portion of the tower) giving G_{EN} = 1,012.

For gust wind loading below the intersection point, the gust factor, G_{EN}, is calculated by the calculation process in Table B.26 to Table B.28 as follows:

$$\sum_{n=1}^{N=6} \gamma_z = 32{,}73 \tag{B.52}$$

and

$$\sum_{n=1}^{N=6} \gamma_z \frac{I_{v,0}}{I_v(z)} = 28{,}07 \tag{B.53}$$

giving

$$J_a = \frac{32{,}73}{28{,}07} = 1{,}166 \tag{B.54}$$

For consistency, between worked examples (see **B.4.2** to **B.4.8**), reference values are arbitrarily taken at the centre of the highest panel, giving V_0 = 29,6 m/s, $b_0\Delta z$ = 1,25 m^2, β_0 = 42,375 m for the calculation of moment, σ_0 = 4,26 m/s and $I_{v,0}$ = 0,144. Thus, the non-dimensional coefficient, γ_z can be calculated for each panel in turn.

Table B.26, Table B.27 and Table B.28 show the calculation process to obtain G_{EN} for patch loading below the intersection point.

Table B.26 **Values of γ(z) γ(z′)**

	z′	**42,375**	**40,500**	**38,000**	**35,500**	**33,000**	**30,500**	**27,750**	**24,425**	**20,475**	**15,900**	**10,400**	**3,675**
z	**γ(z)**	**0,00**	**0,00**	**0,00**	**0,00**	**0,00**	**0,00**	**3,72**	**4,52**	**5,24**	**5,90**	**6,93**	**6,43**
42,375	**0,00**	0	0	0	0	0	0	0	0	0	0	0	0
40,500	**0,00**	0	0	0	0	0	0	0	0	0	0	0	0
38,000	**0,00**	0	0	0	0	0	0	0	0	0	0	0	0
35,500	**0,00**	0	0	0	0	0	0	0	0	0	0	0	0
33,000	**0,00**	0	0	0	0	0	0	0	0	0	0	0	0
30,500	**0,00**	0	0	0	0	0	0	0	0	0	0	0	0
27,750	**3,72**	0	0	0	0	0	0	13,81	16,80	19,46	21,91	25,74	23,91
24,425	**4,52**	0	0	0	0	0	0	16,80	20,43	23,67	26,65	31,30	29,08
20,475	**5,24**	0	0	0	0	0	0	19,46	23,67	27,42	30,87	36,26	33,68
15,900	**5,90**	0	0	0	0	0	0	21,91	26,65	30,87	34,77	40,84	37,93
10,400	**6,93**	0	0	0	0	0	0	25,74	31,30	36,26	40,84	47,96	44,55
3,675	**6,43**	0	0	0	0	0	0	23,91	29,08	33,68	37,93	44,55	41,39

Table B.27 **Valuesof C(z-z′)**

z, z′	42,375	40,500	38,000	35,500	33,000	30,500	27,750	24,425	20,475	15,900	10,400	3,675
42,375	1,000	0,918	0,818	0,730	0,651	0,581	0,512	0,440	0,367	0,297	0,231	0,170
40,500	0,918	1,000	0,892	0,795	0,709	0,633	0,558	0,479	0,400	0,324	0,252	0,185
38,000	0,818	0,892	1,000	0,892	0,795	0,709	0,625	0,537	0,448	0,363	0,283	0,208
35,500	0,730	0,795	0,892	1,000	0,892	0,795	0,701	0,602	0,503	0,408	0,317	0,233
33,000	0,651	0,709	0,795	0,892	1,000	0,892	0,786	0,675	0,564	0,457	0,355	0,261
30,500	0,581	0,633	0,709	0,795	0,892	1,000	0,882	0,757	0,632	0,512	0,398	0,293
27,750	0,512	0,558	0,625	0,701	0,786	0,882	1,000	0,859	0,717	0,581	0,452	0,332
24,425	0,440	0,479	0,537	0,602	0,675	0,757	0,859	1,000	0,835	0,677	0,526	0,387
20,475	0,367	0,400	0,448	0,503	0,564	0,632	0,717	0,835	1,000	0,811	0,630	0,463
15,900	0,297	0,324	0,363	0,408	0,457	0,512	0,581	0,677	0,811	1,000	0,777	0,571
10,400	0,231	0,252	0,283	0,317	0,355	0,398	0,452	0,526	0,630	0,777	1,000	0,735
3,675	0,170	0,185	0,208	0,233	0,261	0,293	0,332	0,387	0,463	0,571	0,735	1,000

Table B.28 **Values of γ(z) γ(z′) C(z-z′)**

z, z′	42,375	40,500	38,000	35,500	33,000	30,500	27,750	24,425	20,475	15,900	10,400	3,675
42,375	0	0	0	0	0	0	0	0	0	0	0	0
40,500	0	0	0	0	0	0	0	0	0	0	0	0
38,000	0	0	0	0	0	0	0	0	0	0	0	0
35,500	0	0	0	0	0	0	0	0	0	0	0	0
33,000	0	0	0	0	0	0	0	0	0	0	0	0
30,500	0	0	0	0	0	0	0	0	0	0	0	0
27,750	0	0	0	0	0	0	13,81	14,43	13,95	12,74	11,63	7,94
24,425	0	0	0	0	0	0	14,43	20,43	19,75	18,04	16,47	11,24
20,475	0	0	0	0	0	0	13,95	19,75	27,42	25,04	22,86	15,61
15,900	0	0	0	0	0	0	12,74	18,04	25,04	34,77	31,74	21,67
10,400	0	0	0	0	0	0	11,63	16,47	22,86	31,74	47,96	32,75
3,675	0	0	0	0	0	0	7,94	11,24	15,61	21,67	32,75	41,39

The value of length scale, ${}^{z}L_u$, is taken from Table B.1 for x = 30 km and a height of 14,625 m [i.e. the elevation midway between the base of panel 1 (z = 0 m) and the height of the intersection point (z = 29,25 m)]. This gives ${}^{z}L_u$ = 22 m.

Aerodynamic admittance, J_p, can now be calculated as follows:

$$\sum_{N=1}^{N=6}\sum_{N=1}^{N=6}\gamma_z\gamma_{z'}\,C(z-z')=737 \qquad \text{B.55}$$

and

$$\left(\sum_{n=1}^{N=6}\gamma_z\right)^2=32,73^2=1071 \qquad \text{B.56}$$

giving

$$J_p=\sqrt{\frac{737}{1071}}=0,830 \qquad \text{B.57}$$

Therefore,

$$G_{EN}=3,6(2J_aJ_pI_{v,0})=3,6(2\times1,166\times0,830\times0,144)=1,002 \qquad \text{B.58}$$

and

$$F_{T,W}(z) = F_{m,W}(z)\ \{1 + [1 + 0,2(z / h)^2]G_{EN}\} \qquad \text{B.59}$$

As the shear is to be calculated in the base panel z = 0 m, the term equals $1+0,2(z/h)^2$ = 1,00.

The force in the diagonal bracing member in the lowest panel due to mean wind over the full height of the tower and gust wind loading above the intersection point = {1 759 N × [1 + (1,00 × 1,012)]} + (–6,129) = –2 590 N.

The force in the diagonal bracing member in the lowest panel due to mean wind over the full height of the tower and gust wind loading below the intersection point = (1 759 N) + {–6,129 × [1 + (1,00 × 1,002)]} = –10 511 N.

NOTE 2 The example shown in ***B.4.6*** *only applies to direct shear. It does not apply to shear in the faces of a panel arising from global torsion.*

B.4.7 Bending moment at the base of the tower for a site with orography

The calculation presented in **B.4.2** is repeated for a site with significant local orography. It is assumed that the tower is located on the crest of a hill (x = 0 m) with an upwind slope of Φ = 0,1 and an upwind and downwind slope length of 500 m. Therefore, the orography factor should be calculated over the height of the tower in accordance with BS EN 1991-1-4:2005, **A.3**. Only those elements of the calculation that have changed are presented.

Table B.29 shows the calculation of mean wind speed, mean wind pressure and turbulence intensity at the mid-height of each panel.

Table B.29 **Meteorological parameters**

Panel	Panel height, Δz	Mid-height	Wind resistance $b\Delta z$	$c_r(z)$	$c_o(z)$	$v_m(z)$	$q_m(z)$	$I_v(z)_{,flat}/c_O$
	m	m	m^2			m/s	N/m^2	
12	1,250	42,375	1,25	1,289	1,173	34,8	741	0,123
11	2,500	40,500	3,80	1,280	1,174	34,6	732	0,123
10	2,500	38,000	3,80	1,268	1,175	34,3	721	0,124
9	2,500	35,500	3,80	1,255	1,177	34,0	708	0,126
8	2,500	33,000	3,80	1,241	1,179	33,7	694	0,127
7	2,500	30,500	4,17	1,227	1,180	33,3	680	0,128
6	3,000	27,750	4,96	1,209	1,182	32,9	663	0,130
5	3,650	24,425	6,17	1,185	1,185	32,3	639	0,131
4	4,250	20,475	7,39	1,153	1,187	31,5	608	0,134
3	4,900	15,900	8,76	1,107	1,191	30,3	563	0,138
2	6,100	10,400	11,31	1,030	1,195	28,3	491	0,144
1	7,350	3,675	13,78	0,845	1,200	23,3	333	0,163

Reference values are arbitrarily taken at the centre of the highest panel, giving V_0 = 34,8 m/s, $b_0\Delta z$ = 1,25 m^2, β_0 = 42,375 m for the calculation of moment, σ_0 = 4,26 m/s and $I_{v,0}$ = 0,123 = 0,144/1,173.

Table B.30 shows the calculation of the non-dimensional coefficient, γ_z and the contribuion of each panel to the shear and moment at the base of the tower.

Table B.30 **Non-dimensional coefficients, wind forces and wind moments**

Panel	σ_v	$\beta(z)$	$\gamma(z)$	$[\gamma(z)I_{v,0}/I_v(z)]$	$q_m(z)b\Delta z$	$q_m(z)b\Delta z\beta$
	m/s	m			N	Nm
12	4,26	42,375	1,00	1,00	926	39 240
11	4,27	40,500	2,89	2,87	2 783	112 716
10	4,27	38,000	2,69	2,65	2 738	104 050
9	4,27	35,500	2,49	2,43	2 690	95 503
8	4,27	33,000	2,29	2,22	2 639	87 085
7	4,26	30,500	2,30	2,20	2 835	86 479
6	4,26	27,750	2,45	2,32	3 287	91 203
5	4,25	24,425	2,63	2,46	3 945	96 356
4	4,22	20,475	2,56	2,34	4 490	91 941
3	4,18	15,900	2,25	2,00	4 932	78 420
2	4,08	10,400	1,73	1,47	5 554	57 757
1	3,80	3,675	0,57	0,43	4 591	16 871
						$\Sigma q_m(z)b\Delta z\beta$ = 957 619

Aerodynamic admittance, J_a, can now be calculated as follows:

$$\sum_{N=1}^{N=12} \gamma_z = 25,87 \quad \text{B.60}$$

and

$$\sum_{n=1}^{N=12} \gamma_z \frac{I_{v,0}}{I_v(z)} = 24,40 \quad \text{B.61}$$

giving

$$J_a = \frac{25,87}{24,4} = 1,060 \quad \text{B.62}$$

Table B.31, Table B.32 and Table B.33 show those parts of the calculation that have changed from **B.4.2** by calculating bending moment at the base of the tower for a site with orography.

Table B.31 **Values of $\gamma(z)\,\gamma(z')$**

	z′	**42,375**	**40,500**	**38,000**	**35,500**	**33,000**	**30,500**	**27,750**	**24,425**	**20,475**	**15,900**	**10,400**	**3,675**
z	$\gamma(z)$	**1,00**	**2,89**	**2,69**	**2,49**	**2,29**	**2,30**	**2,45**	**2,63**	**2,56**	**2,25**	**1,73**	**0,57**
42,375	**1,00**	1,00	2,89	2,69	2,49	2,29	2,30	2,45	2,63	2,56	2,25	1,73	0,57
40,500	**2,89**	2,89	8,36	7,78	7,20	6,63	6,65	7,09	7,61	7,41	6,49	5,01	1,65
38,000	**2,69**	2,69	7,78	7,24	6,71	6,17	6,19	6,61	7,08	6,90	6,05	4,66	1,54
35,500	**2,49**	2,49	7,20	6,71	6,21	5,72	5,73	6,12	6,56	6,39	5,60	4,32	1,42
33,000	**2,29**	2,29	6,63	6,17	5,72	5,26	5,28	5,63	6,04	5,88	5,15	3,97	1,31
30,500	**2,30**	2,30	6,65	6,19	5,73	5,28	5,29	5,65	6,06	5,89	5,17	3,98	1,31
27,750	**2,45**	2,45	7,09	6,61	6,12	5,63	5,65	6,02	6,46	6,29	5,51	4,25	1,40
24,425	**2,63**	2,63	7,61	7,08	6,56	6,04	6,06	6,46	6,93	6,74	5,91	4,56	1,50
20,475	**2,56**	2,56	7,41	6,90	6,39	5,88	5,89	6,29	6,74	6,56	5,76	4,44	1,46
15,900	**2,25**	2,25	6,49	6,05	5,60	5,15	5,17	5,51	5,91	5,76	5,05	3,89	1,28
10,400	**1,73**	1,73	5,01	4,66	4,32	3,97	3,98	4,25	4,56	4,44	3,89	3,00	0,99
3,675	**0,57**	0,57	1,65	1,54	1,42	1,31	1,31	1,40	1,50	1,46	1,28	0,99	0,33

Table B.32 **Values of C(z-z′)**

z, z′	42,375	40,500	38,000	35,500	33,000	30,500	27,750	24,425	20,475	15,900	10,400	3,675
42,375	1,000	0,939	0,863	0,794	0,730	0,671	0,612	0,547	0,479	0,411	0,342	0,273
40,500	0,939	1,000	0,919	0,845	0,777	0,715	0,652	0,583	0,510	0,438	0,364	0,290
38,000	0,863	0,919	1,000	0,919	0,845	0,777	0,709	0,634	0,555	0,476	0,396	0,316
35,500	0,794	0,845	0,919	1,000	0,919	0,845	0,771	0,689	0,604	0,518	0,430	0,343
33,000	0,730	0,777	0,845	0,919	1,000	0,919	0,838	0,750	0,657	0,563	0,468	0,374
30,500	0,671	0,715	0,777	0,845	0,919	1,000	0,912	0,815	0,714	0,612	0,509	0,406
27,750	0,612	0,652	0,709	0,771	0,838	0,912	1,000	0,894	0,783	0,672	0,558	0,446
24,425	0,547	0,583	0,634	0,689	0,750	0,815	0,894	1,000	0,876	0,751	0,624	0,498
20,475	0,479	0,510	0,555	0,604	0,657	0,714	0,783	0,876	1,000	0,858	0,713	0,569
15,900	0,411	0,438	0,476	0,518	0,563	0,612	0,672	0,751	0,858	1,000	0,831	0,663
10,400	0,342	0,364	0,396	0,430	0,468	0,509	0,558	0,624	0,713	0,831	1,000	0,798
3,675	0,273	0,290	0,316	0,343	0,374	0,406	0,446	0,498	0,569	0,663	0,798	1,000

Table B.33 **Values of γ(z) γ(z′) C(z-z′)**

z, z′	42,375	40,500	38,000	35,500	33,000	30,500	27,750	24,425	20,475	15,900	10,400	3,675
42,375	1,00	2,71	2,32	1,98	1,67	1,54	1,50	1,44	1,23	0,92	0,59	0,16
40,500	2,71	8,36	7,15	6,09	5,15	4,75	4,62	4,43	3,78	2,84	1,82	0,48
38,000	2,32	7,15	7,24	6,17	5,22	4,81	4,68	4,49	3,83	2,88	1,84	0,49
35,500	1,98	6,09	6,17	6,21	5,26	4,85	4,72	4,52	3,86	2,90	1,86	0,49
33,000	1,67	5,15	5,22	5,26	5,26	4,85	4,72	4,53	3,86	2,90	1,86	0,49
30,500	1,54	4,75	4,81	4,85	4,85	5,29	5,15	4,94	4,21	3,17	2,03	0,53
27,750	1,50	4,62	4,68	4,72	4,72	5,15	6,02	5,78	4,92	3,70	2,37	0,62
24,425	1,44	4,43	4,49	4,52	4,53	4,94	5,78	6,93	5,91	4,44	2,85	0,75
20,475	1,23	3,78	3,83	3,86	3,86	4,21	4,92	5,91	6,56	4,94	3,16	0,83
15,900	0,92	2,84	2,88	2,90	2,90	3,17	3,70	4,44	4,94	5,05	3,23	0,85
10,400	0,59	1,82	1,84	1,86	1,86	2,03	2,37	2,85	3,16	3,23	3,00	0,79
3,675	0,16	0,48	0,49	0,49	0,49	0,53	0,62	0,75	0,83	0,85	0,79	0,33

As in the example shown in **B.4.2**, the length scale $^{z}L_{u}$ = 30 m.

Table B.31 and Table B.32 are now combined to give Table B.33.

Aerodynamic admittance, J_p, should now be calculated as follows:

$$\sum_{n=1}^{N=12}\sum_{n=1}^{N=12}\gamma_z\gamma_{z'}C(z-z')=478 \qquad \text{B.63}$$

and

$$\left(\sum_{n=1}^{N=12}\gamma_z\right)^2=25{,}9^2=671 \qquad \text{B.64}$$

giving

$$J_\text{p}=\sqrt{\frac{478}{671}}=0{,}844 \qquad \text{B.65}$$

Therefore,

$$G_\text{EN}=3{,}6(2J_\text{a}J_\text{p}I_\text{v,0})=3{,}6(2\times1{,}060\times0{,}844\times0{,}123)=0{,}791 \qquad \text{B.66}$$

The bending moment at the base of the tower under the 10 min mean wind speed is $\sum q_m(z)b\Delta z\beta$ and this equals 958 kNm. The total bending moment at the base of the tower under the effect of mean and gust wind loading is given by:

$$F_{T,W}(z) = F_{m,W}(z)\ \{1 + [1 + 0{,}2(z / h)^2]\ G_{EN} \qquad \text{B.67}$$

Thus, the total bending moment at the base of the tower = 1 716 kNm = 958{1+[1+0,2(0)]×0,791}.

B.4.8 Large ancillary

Unsymmetrical towers, or towers that contain unsymmetrically placed large ancillaries and/or cables imposing significant torsional and crosswind loads, should be treated separately. This worked example addresses the case of a single large unsymmetrically placed large ancillary.

The load generated by wind on the ancillary should be subdivided into the following components.

a) Mean wind: depending on the drag and lift characteristics of the ancillary and its position in relation to the centroid of the tower, this results in steady state along-wind, across-wind and torsional loads.

b) Along-wind turbulence: depending on the drag and lift characteristics of the ancillary and its position in relation to the centroid of the tower, this results in fluctuating along-wind, across-wind and torsional loads.

c) Cross wind turbulence: depending on the drag and lift characteristics of the ancillary and its position in relation to the centroid of the tower, this results in fluctuating across-wind, along-wind and torsional loads.

It is assumed that the fluctuating components are uncorrelated and so their load effects are combined by taking the square root of the sum of the squares and this is then added to the mean load effect (see BS EN 1993-3-1:2006, equation B.20).

The tower should be analyzed without the large ancillary as described in **B.4.2** to **B.4.7** as appropriate. The wind resistance of the tower may be reduced locally to take account of shielding of the tower by the large ancillary. This course of action is often a matter of engineering judgement. For the purposes of the worked example, mutual shielding has not been included.

In this example, the bending moment and shear are calculated at the base of the tower, without local orography. Mutual shielding is not included, so the load effects for the tower without the large ancillary are as given in **B.4.2**, namely, moment = 1 327 kNm (of which 686 kN m arises from mean wind loading) and shear = 57,4 kN (of which 29,5 kN arises from mean wind loading).

Consider a single 3,7 m diameter dish antenna mounted at a height of 33 m, with a line of shoot at 30 degrees to the wind bearing. The centre of pressure of the antenna is at an eccentricity of 3 m from the tower's centroid. The antenna has the following (arbitrary) wind resistances as shown in Table B.34.

Table B.34 **Large ancillary wind resistance**

Component	Wind resistance
Drag (C_DA)	14,5 m^2
Lift (C_LA)	10,8 m^2

The 10 min mean wind pressure at the height of the antenna is $q_m(z)$ = 500 N/m^2. The 1 s gust wind pressure at this height is given by $q_p(z)=q_bc_e(z)$. For 30 km of country fetch and a height of 33 m, $C_e(z)$ 3,23; q_b = 0,613(23 m/s)^2 = 324 N/m^2, giving q_p(33 m) = 3,23 × 324 = 1 047 N/m^2. The 1 s gust wind pressure has been selected because gust wind loading on a 3,7 m diameter dish is expected to be fully correlated. For large unsymmetric linear ancillaries, a longer averaging period for the gust wind pressure might be appropriate to account for lack of gust correlation.

The load effects arising from the mean wind are summarized as follows:

a) the along-wind shear is 500 N/m^2 × 14,5 m^2 = 7,25 kN;

b) the across-wind shear is 500 N/m^2 × 10,8 m^2 = 5,40 kN;

c) the torsion is 7,25 kN × 3sin30° – 5,4 kN × 3cos30° = –3,15 kNm;

d) the along-wind moment is 7,25 kN × 33 m = 239 kNm;

e) the across-wind moment is 5,4 kN × 33 m = 178 kNm.

Fluctuating load effects are summarized as below. The factor: $[1+0,2(z/h)^2]$ is calculated at the height of the large antenna; in this example, it has a value of 1,118.

The load effects arising from the in-line fluctuating component of the wind are summarized as follows:

a) the along-wind shear is (1 047 N/m^2 – 500 N/m^2) × 1,118 × 14,5 m^2 = 8,87 kN;

b) the across-wind shear is (1 047 N/m^2 – 500 N/m^2) × 1,118 × 10,8 m^2 = 6,60 kN;

c) the torsion is 8,87 kN × 3sin30° – 6,60 kN × 3cos30° = –3,84 kNm;

d) the along-wind moment is 8,87 kN $\times$ 33 m = 293 kNm;

e) the across-wind moment is 6,60 kN $\times$ 33 m = 218 kNm.

The NA to BS EN 1993-3-1 sets the factor to allow for cross-wind turbulence to K_x = 0,5. Thus, the load effects arising from the cross wind fluctuating component of the wind are summarized as follows:

a) the along-wind shear is $0,5 \times (1\,047\ \text{N/m}^2 - 500\ \text{N/m}^2) \times 1,118 \times 14,5\ \text{m}^2 = 4,43$ kN;

b) the across-wind shear is $0,5 \times (1\,047\ \text{N/m}^2 - 500\ \text{N/m}^2) \times 1,118 \times 10,8\ \text{m}^2 = 3,30$ kN;

c) the torsion is 4,43 kN $\times$ 3sin30° – 3,30 kN $\times$ 3cos30° = –1,92 kNm;

d) the along-wind moment is 4,43 kN $\times$ 33 m = 146 kNm;

e) the across-wind moment is 3,30 kN $\times$ 33 m = 109 kNm.

The total load effects arisng from wind on the large ancillary are summarized as follows:

a) the along-wind shear is $7,25+\sqrt{8,87^2+4,43^2=}17,2$ kN;

b) the across-wind shear is $5,4+\sqrt{6,60^2+3,30^2=}12,8$ kN;

c) the torsion is $-3,15-\sqrt{3,84^2+1,92^2=}-7,44$ kNm;

d) the along-wind moment is $239+\sqrt{293^2+146^2=}566$ kNm;

e) the across-wind moment is $178+\sqrt{218^2+109^2=}421$ kNm.

So finally, the total load effects arising from wind on the tower and the large ancillary are summarized as follows:

a) the along-wind shear is 57,3 + 17,2 = 74,5 kN;

b) the across-wind shear is 0,5(57,3 – 29,5) + 12,8 = 26,7;

c) the torsion is –7,44 kNm;

d) the along-wind moment is 1 327 + 566 = 1 893 kNm;

e) the across-wind moment is 0,5(1 327 – 686) + 421 = 742 kNm.

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