

PD 6688-1-4:2015



BSI Standards Publication

PUBLISHED DOCUMENT

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

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Summary of pages

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This Published Document supersedes PD 6688-1-4:2009, which is withdrawn.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

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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.

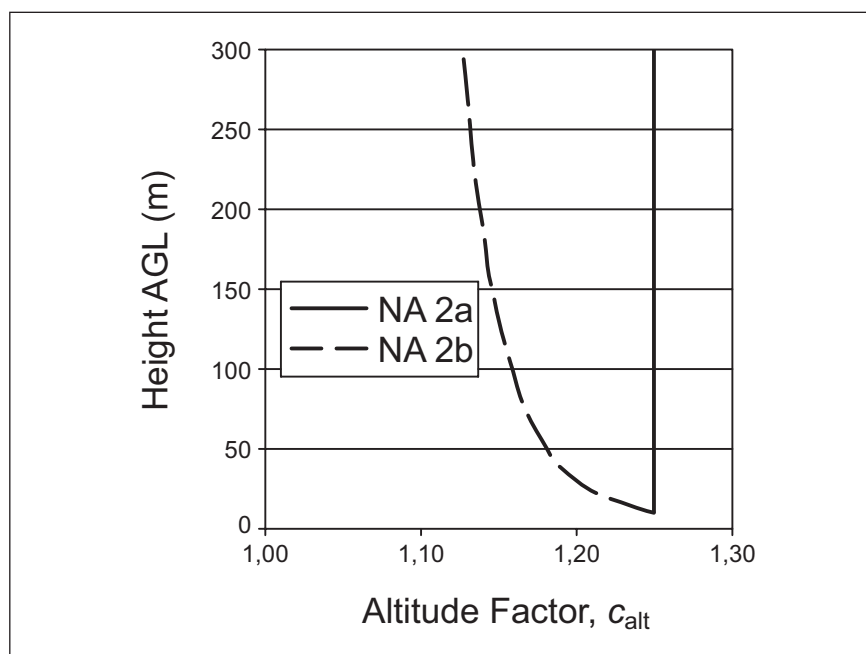
Altitude factor c_{alt} and corrections to account for changes of surface roughness are both National Choices. The former reduces the need to assess the effects of hills (orography) in many cases, while the latter allows conservatism to be reduced for sites further downwind of a coast or town boundary.

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 $c_0(z)$.

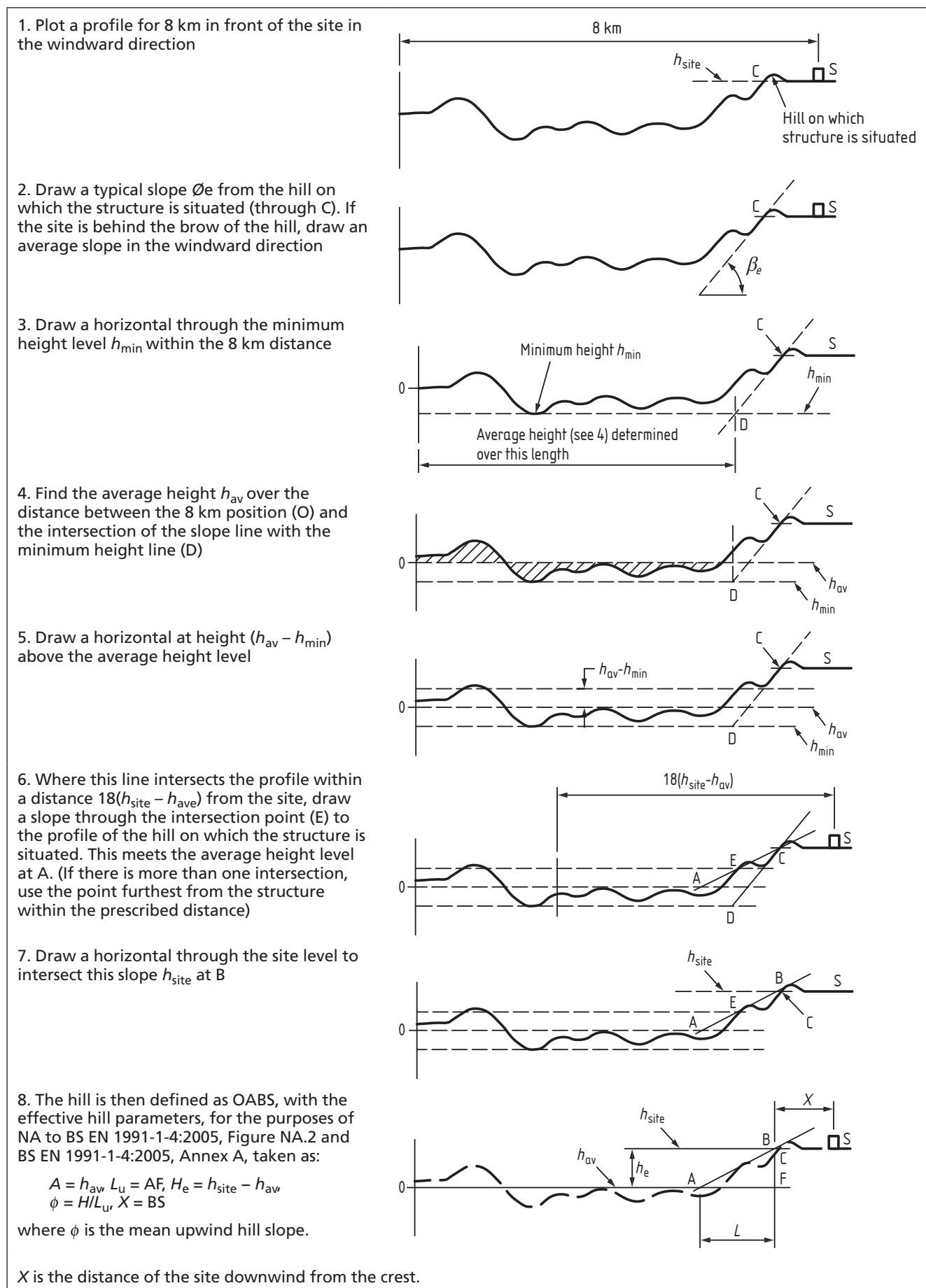
This procedure provides formulae (and graphs) to determine $c_0(z)$ for clearly defined cliffs and escarpments and hills and ridges. Unfortunately in the U.K. the majority of escarpments, hills and ridges occur in undulating terrain – only sea edge cliffs can clearly meet the configurations given in 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



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_t [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 $I_v(z) = k_t / (c_o(z) \ln(z/z_0))$ rather than k_t on its own, charts provide values of $k_t / \ln(z/z_0)$ 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 $1,226 \text{ kg/m}^3$, 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.

- 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.
- 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.
- 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 m² and 10 m². 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 m² or less and use the $c_{pe,10}$ values for all areas > 1 m². This is the procedure given in NA to BS EN 1991-1-4:2005, and for areas > 1 m² 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 $\pm 45^\circ$ 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 $\pm 15^\circ$ 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.

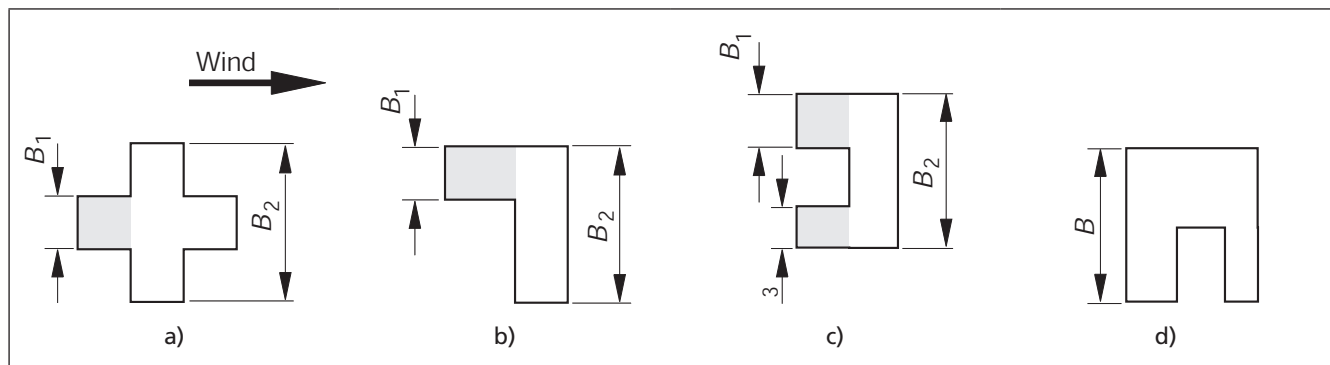
For internal wells and recessed bays in side faces [see Figure 3d)] the following apply, where the gap across the well or bay is smaller than $e/2$ where $e = B$ or $2H$, whichever is less.

- 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

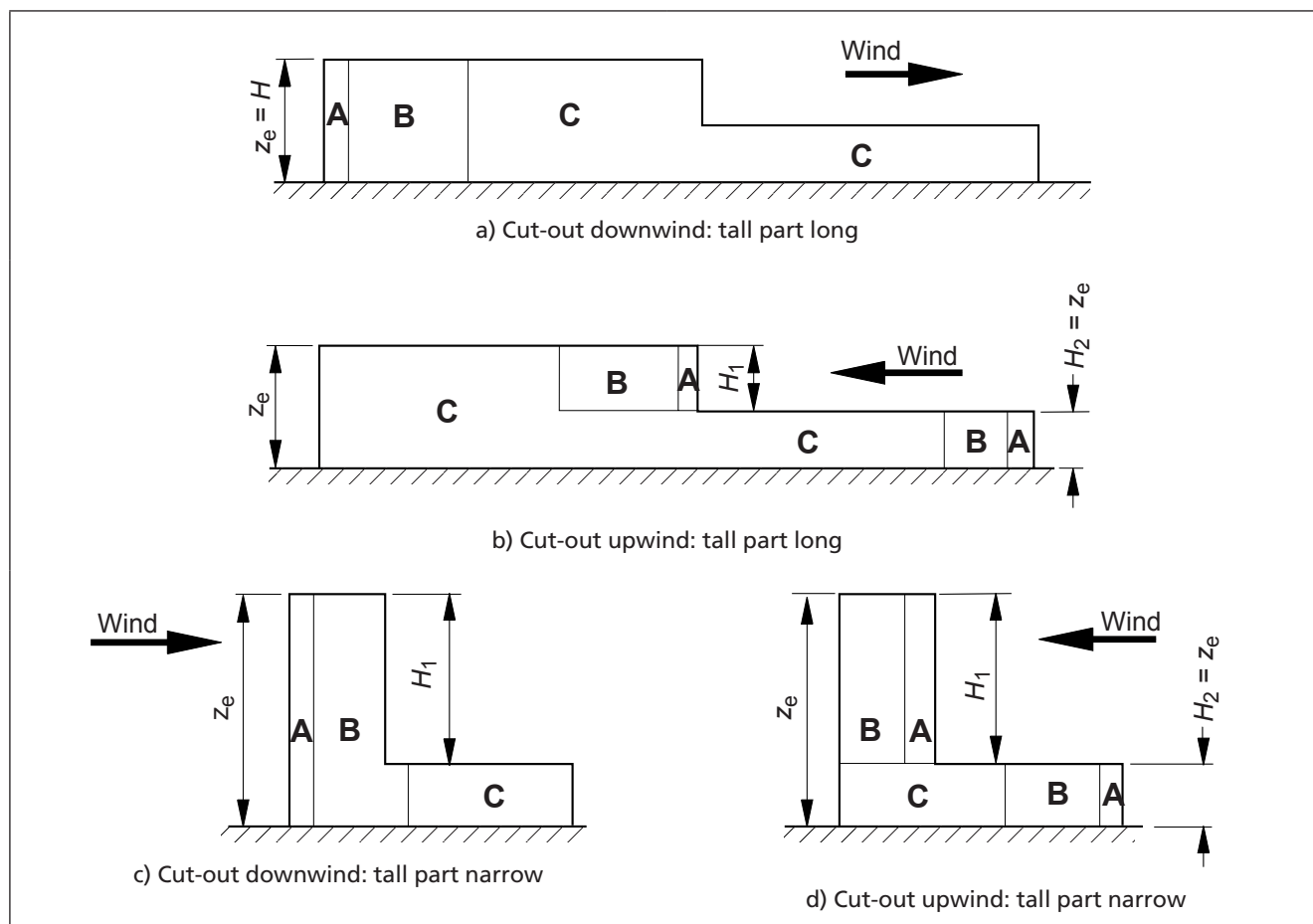
3.3.1 Irregular flush faces

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.

- 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.
- 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



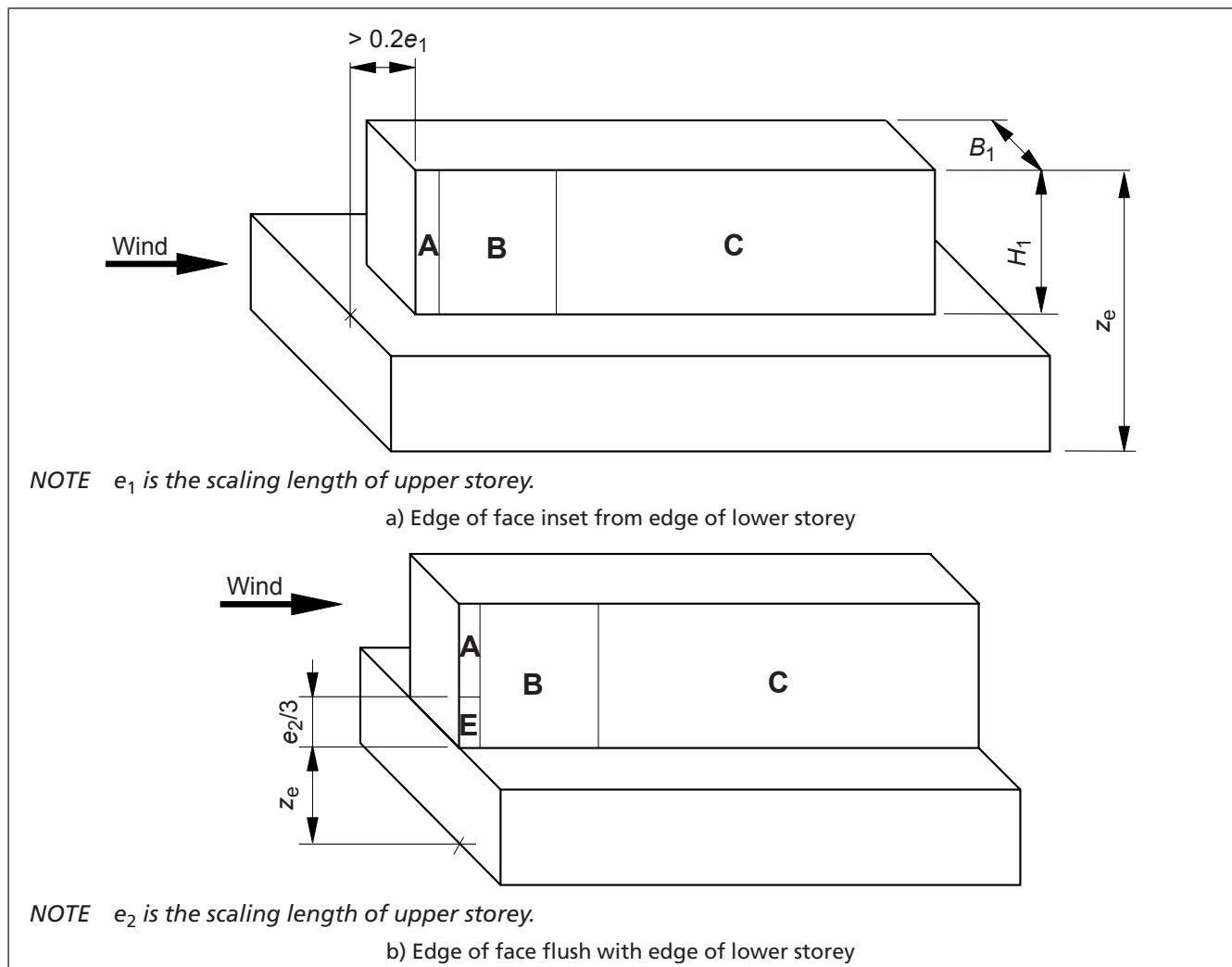
3.3.2 Walls of inset storeys

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

- 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.
- 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

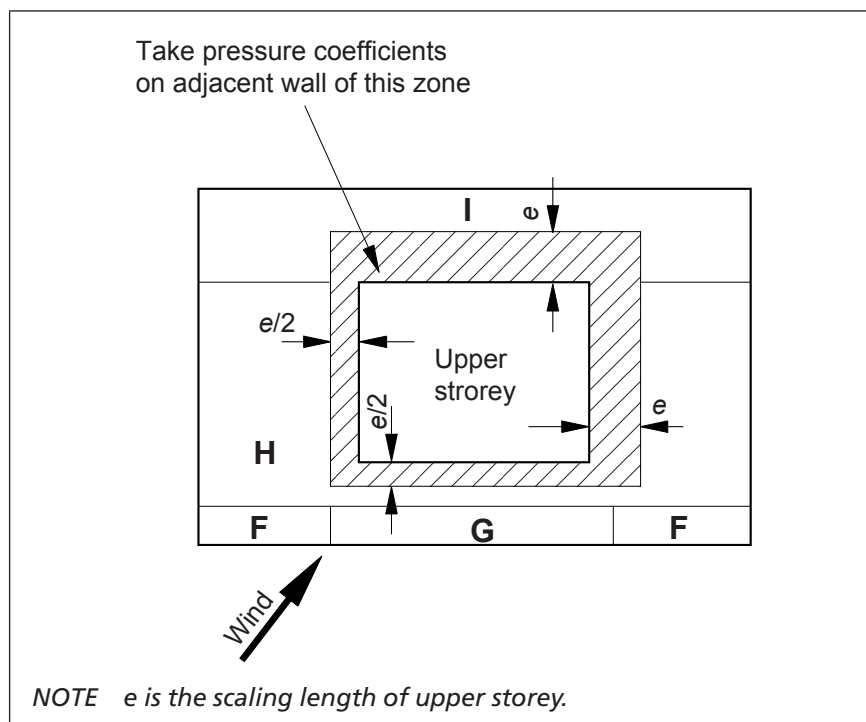


3.4 Flat roofs with inset storeys

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.

- 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.
- 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



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 $\theta = 0^\circ$ apply to canopy on windward wall and those for $\theta = 90^\circ$ apply to canopies fixed to the side wall. Positive forces act downwards.

Figure 7 Key to canopies attached to buildings

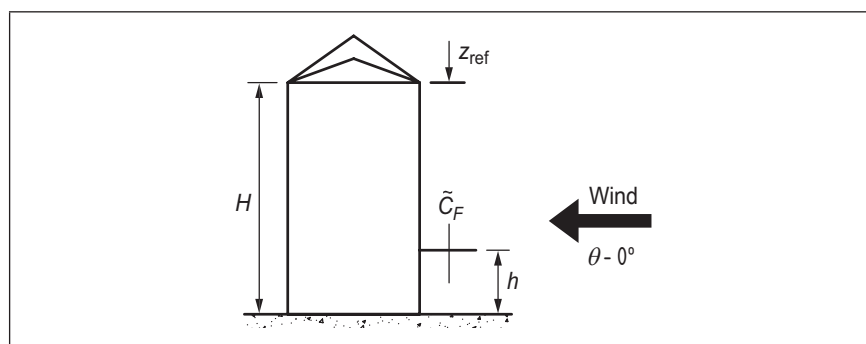


Table 1 Global vertical force coefficients for canopies attached to tall buildings

H/h	2	3	6	12	18	24	30	36
$\theta = 0^\circ$	+0,3	+0,4	+0,69	+0,87	+0,92	+0,93	+0,93	+0,91
$\theta = 90^\circ$	-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 $\theta = 0^\circ$ 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 $\theta = 45^\circ$ 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 $\geq 0,3$	-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.

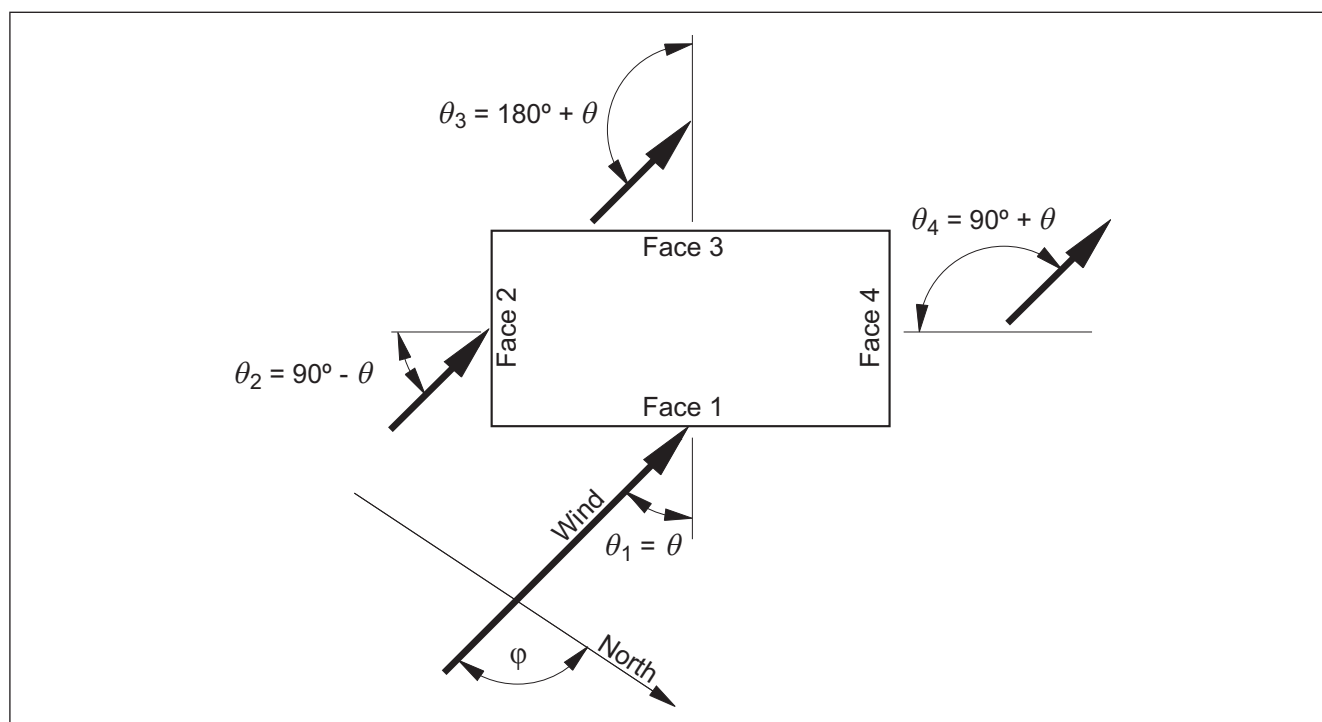
3.9.1 Wind direction

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

- in degrees east of north, represented by φ , used to determine wind speeds and dynamic pressure;
- 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 $\pm 45^\circ$ from normal to the axis of the gap. In this circumstance, the following apply.

- a) Over the range of wind angle $-45^\circ < \theta < +45^\circ$ 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 $-135^\circ < \theta < -45^\circ$ and $+45^\circ < \theta < +135^\circ$ funnelling occurs. Values for zone A at $\theta = \pm 90^\circ$ should be multiplied by 1,2. Values for zones B at $\theta = \pm 90^\circ$ 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 $-180^\circ < \theta < -135^\circ$ and $+135^\circ < \theta < +180^\circ$ 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	$d/H \leq 1$				$d/H \geq 4$			
	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

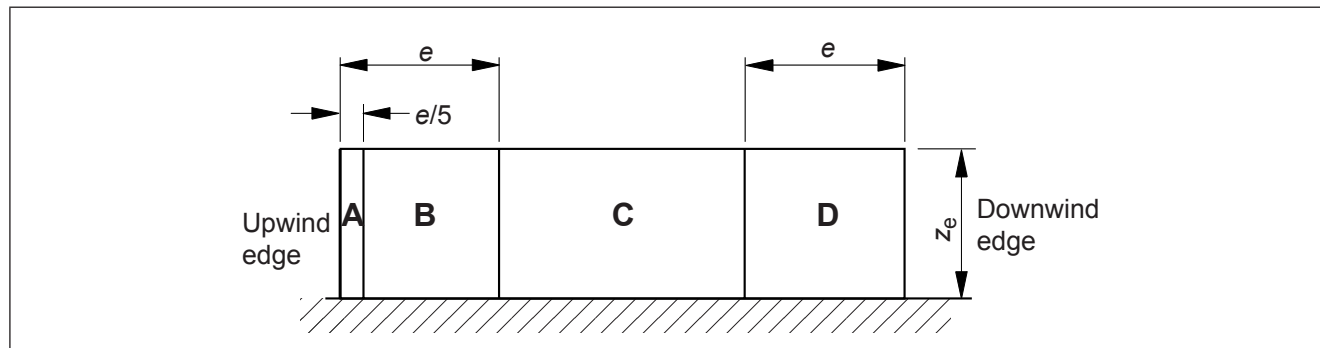
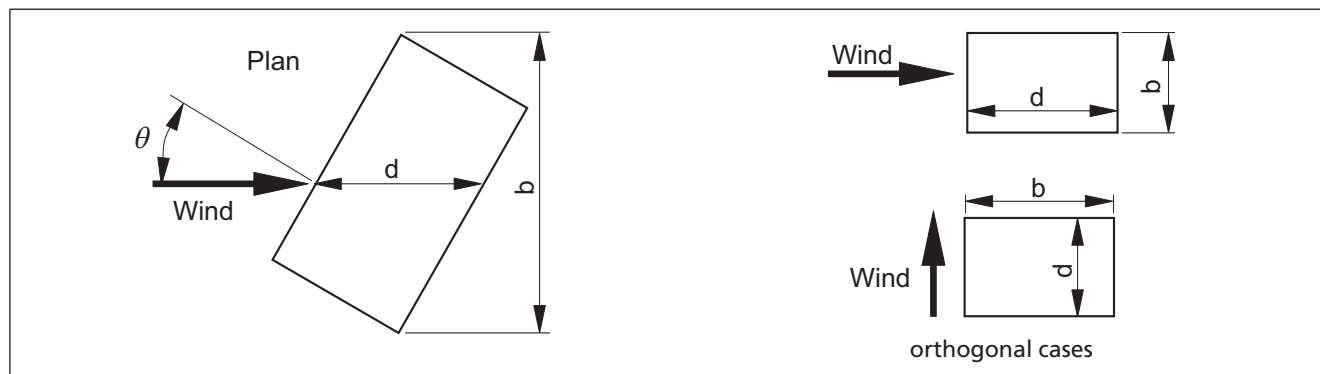


Figure 10 Definitions of crosswind breadth and in wind depth



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 $60^\circ < \theta < 120^\circ$, 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 $\beta = 90^\circ$ 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
<i>NOTE Interpolation is allowed in the range $60^\circ < \beta < 150^\circ$</i>	

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

$$v_{\text{crit},i} > 1,25v_m \quad \text{A.1}$$

where

$v_{\text{crit},i}$ 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 $1,25v_m$. 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 $(v/v_m)^2$, where $v_m < v < 1,25v_m$.

A.1.3 Basic parameters for vortex shedding

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

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.

$$v_{\text{crit},i} = \frac{an_i}{St} \quad \text{A.2}$$

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.

$$v_{crit,i} = \frac{bn_{i,0}}{2St} \quad \text{A.3}$$

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

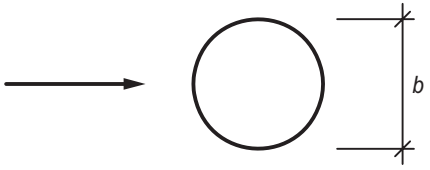
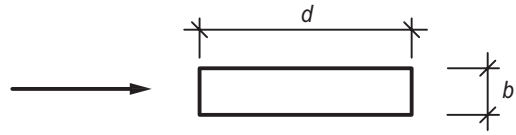
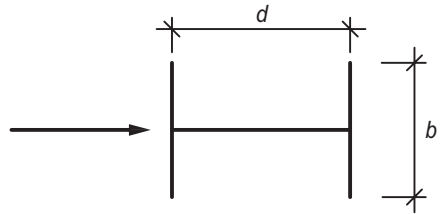
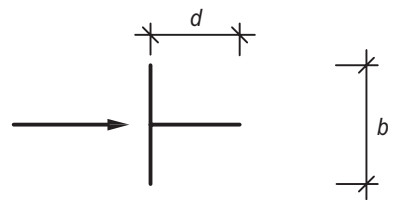
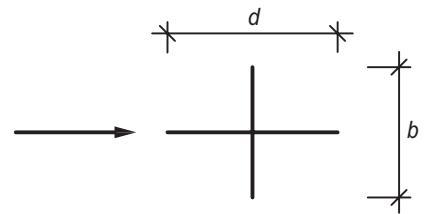
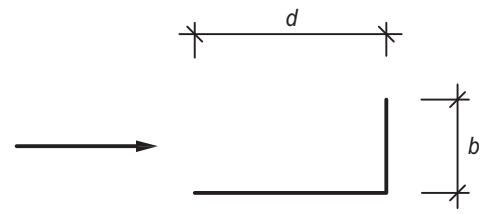
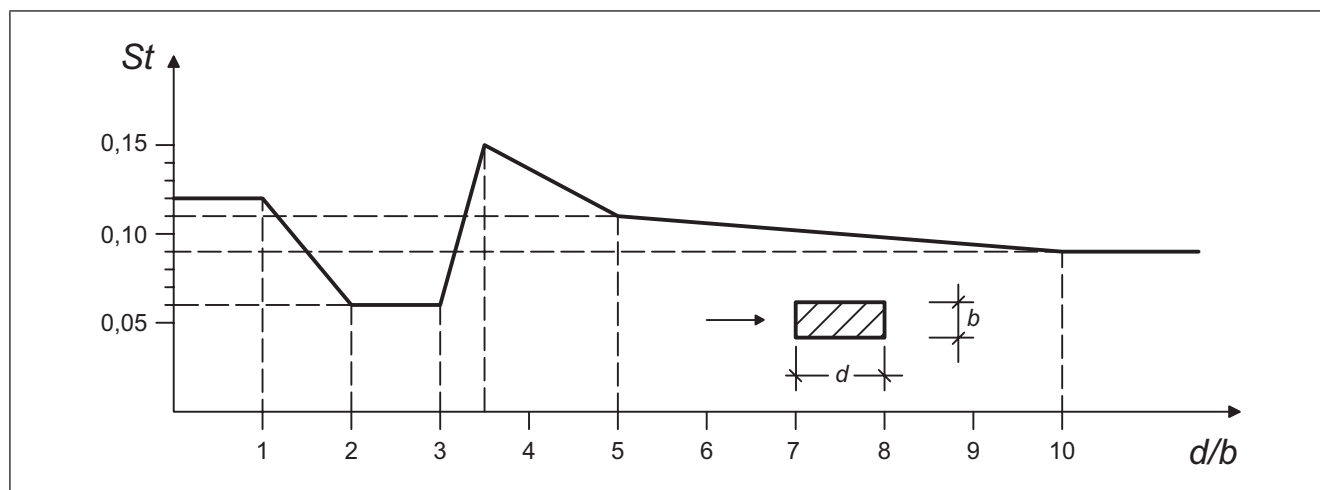
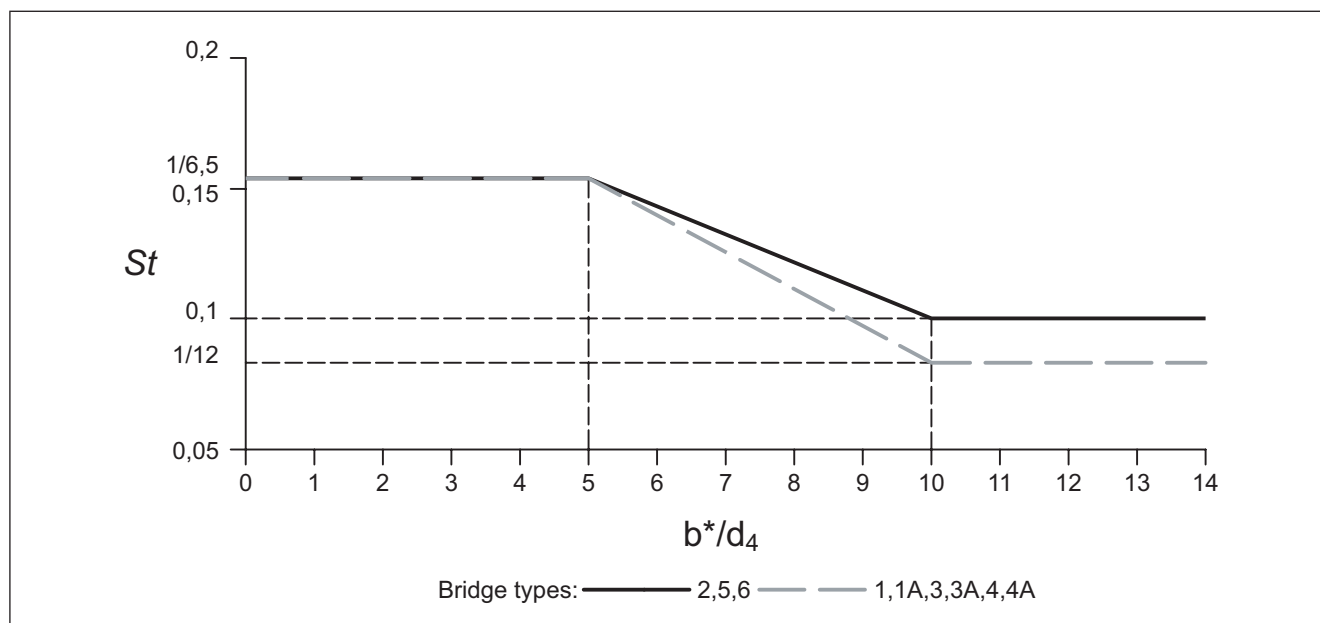
Cross-section		St
		0,18
for all Re -numbers		
		From Figure A.1
$0,5 \leq d/b \leq 10$		
	$d/b = 1$	0,11
	$d/b = 1,5$	0,10
	$d/b = 2$	0,14
linear interpolation		
	$d/b = 1$	0,13
	$d/b = 2$	0,08
linear interpolation		
	$d/b = 1$	0,16
	$d/b = 2$	0,12
linear interpolation		
	$d/b = 1,3$	0,11
	$d/b = 2$	0,07
linear interpolation		
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.