

PUBLISHED DOCUMENT

Recommendations for the design of bridges to BS EN 1993

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Contents

Foreword *iii*

1	Scope	<i>1</i>
2	Normative references	<i>1</i>
3	Terms and definitions	<i>1</i>
4	Global analysis	<i>2</i>
5	Non-dimensional slenderness for beam segments between effective lateral restraints	<i>3</i>
6	Non-dimensional slenderness for beams without intermediate restraints between support positions	<i>7</i>
7	Non-dimensional slenderness for cantilever beams without intermediate lateral restraints	<i>10</i>
8	Non-dimensional slenderness for beams with intermediate torsional restraints	<i>12</i>
9	Simplified method for verification of lateral buckling of truss chords and flanges in compression	<i>17</i>
10	Restraints at supports – effects due to restraint of main beams	<i>18</i>
11	Intermediate restraints – effects due to restraint of main beams	<i>25</i>
12	Buckling resistance of plates with out of plane loading	<i>27</i>
13	Resistance of members with flanges curved out of plane	<i>29</i>
14	Design of flanges and webs with large openings	<i>31</i>
15	Design of intermediate transverse web stiffeners	<i>31</i>
16	Design of bearing stiffeners	<i>32</i>
17	Connections – design of beam splices	<i>35</i>
18	Connections – design of gusset plates	<i>37</i>
19	Bolted connections	<i>38</i>
20	Welded connections	<i>38</i>
21	Cross beams and other transverse members in flanges	<i>40</i>
22	Shape limitations for stiffener outstands	<i>41</i>
23	Box girders – the effects of torsion and distortion	<i>43</i>
24	Box girders – the design of plated diaphragms	<i>51</i>
25	Box girders – the design of internal cross frames	<i>83</i>
26	Web breathing	<i>85</i>

Bibliography *86*

List of figures

Figure 1	– Slenderness factor for variation in bending moment	<i>6</i>
Figure 2	– Effective length parameter k_f – effect of rotational end restraint	<i>9</i>
Figure 3	– Effective length parameter for beams with discrete torsional restraints	<i>15</i>
Figure 4	– Rotations of paired beams subject to unit torques	<i>15</i>
Figure 5	– Restraint forces – continuous beams	<i>22</i>
Figure 6	– Restraint of compression flange by U-frames or deck or end torsional restraint	<i>23</i>
Figure 7	– Gusset plates	<i>38</i>
Figure 8	– Welds with packings	<i>39</i>
Figure 9	– Notation for angles and tees	<i>42</i>
Figure 10	– Distribution of torsional warping stresses	<i>44</i>
Figure 11	– Longitudinal stresses due to distortional warping	<i>46</i>
Figure 12	– System of diagonal forces	<i>48</i>

Figure 13 – Transverse distortional moments	50
Figure 14 – Geometric notation for diaphragms	52
Figure 15a – Openings in stiffened diaphragms – twin bearings	55
Figure 15b – Openings in stiffened diaphragms – single central bearing and positioning of bearings	55
Figure 16 – Coefficient K_c for plate panels under direct compression	60
Figure 17 – Reference point and notation for unstiffened diaphragms	61
Figure 18 – Load effects and notation for stiffened diaphragms	67
Figure 19 – Parameters for the design of stiffeners	72
Figure 20 – Stresses on panels	80
Figure 21a – Buckling coefficients K_1 , K_2 , K_q and K_b	81
Figure 21b – Values of K_q	82
Figure 21c – Values of K_b	83

List of tables

Table 1 – Diaphragm stiffness, S	49
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Summary of pages

This document comprises a front cover, an inside front cover, pages i to iv, pages 1 to 86, an inside back cover and a back cover.

Foreword

Publishing information

This Published Document is published by BSI Standards Limited, under licence from The British Standards Institution, and came into effect on 31 July 2008. It was prepared by Technical Committee B/525/10, *Bridges*, Working Group 3. A list of organizations represented on this committee can be obtained on request to its secretary.

Supersession

PD 6695-2:2008+A1:2012 supersedes PD 6695-2:2008, which is withdrawn.

Relationship with other publications

This Published Document gives recommended “non-contradictory complimentary information” (NCCI) that may be of assistance in the design of steel or steel/concrete bridges to BS EN 1993 and BS EN 1994.

Information about this document

The document has drawn upon material contained in BS 5400-3:2000, in particular upon the rules that do not have a similar detailed treatment in BS EN 1993. The guidance materials are primarily new aspects of bridge design not adequately covered in BS EN 1993 and have been presented in a format that should facilitate its complementary use with the rules in BS EN 1993. Nothing in this document should be used in any way that contradicts the principles or application rules of Eurocodes.

For guidance on fatigue and through thickness properties of steel the designers should consult the relevant standards and the accompanying National Annex.

The start and finish of text introduced or altered by Amendment No. 1 is indicated in the text by tags **[A1]** **[A1]**. Minor editorial changes are not tagged.

Amendment A1 introduces changes to **5.2**, **6.2**, **7.2**, **8.2**, **12.2**, **21.1**, **21.3**, **23.3** and **23.4**.

The start and finish of text introduced or altered by Corrigendum No. 1 is indicated in the text by tags **[C1]** **[C1]**.

A Corrigendum to PD 6695-2 was issued in 2010, and an Amendment was issued in 2012 (PD 6695-2:2008+A1:2012). The 2010 Corrigendum has now been incorporated into the text; the tags **[C1]** **[C1]** currently in the document refer to the 2013 Corrigendum.

Contractual and legal considerations

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

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

1 Scope

This Published Document contains non contradictory complementary information and additional guidance for use in the UK with BS EN 1993-2, BS EN 1993-1-5, BS EN 1993-1-7, BS EN 1993-1-8 and their UK National Annexes in the design of components of steel bridges. This document only covers bridges which are executed in accordance with BS EN 1090-2.

The information in this document relates only to static design of steel bridges covering slenderness parameters, buckling and bending resistance of diaphragms, bridge restraints and a few other selected topics. In some circumstances it may also be appropriate to the steel elements of composite bridges but such use needs to take full account of BS EN 1994.

This Published Document does not cover the detailing of components for seismic design or the design of parapets, safety fences and other ancillary items. It also does not fully cover cable-supported bridges in respect of strength and stiffness of cables, testing, etc., for which designers can refer to BS EN 1993-1-11. Guidance on fatigue and brittle fracture resistance on these can be obtained from PD 6695-1-9 and PD 6695-1-10 respectively.

2 Normative references

The following referenced documents are indispensable for the application. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

BS EN 1090-2, *Execution of steel structures and aluminium structures – Part 2: Technical requirements for steel structures*

BS EN 1990, *Eurocode – Basis of structural design*

BS EN 1993-1-1:2005, *Eurocode 3 – Design of steel structures – Part 1.1: General rules and rules for buildings*

BS EN 1993-1-5:2006, *Eurocode 3 – Design of steel structures – Part 1-5: Plated structural elements*

BS EN 1993-1-7, *Eurocode 3 – Design of steel structures – Part 1-7: Plated structures subject to out of plane loading*

BS EN 1993-1-8:2005, *Eurocode 3 – Design of steel structures – Part 1-8: Design of joints*

BS EN 1993-2:2006, *Eurocode 3 – Design of steel structures – Part 2: Steel bridges*

3 Terms and definitions

For the purposes of this Published Document, the terms and definitions given in BS EN 1990 apply.

4 Global analysis

4.1 General

This Published Document gives guidance to complement the requirements of BS EN 1993-2 for global analysis with respect to the modelling of joints and imperfections. Guidance on analysis for fatigue is given in PD 6695-1-9.

4.2 Joint modelling

Semi-rigid joints should not be used for bridges that are subjected to cyclic loadings.

4.3 Truss connections

The global load effects for trusses should be calculated in accordance with elastic theory, based on the assumption that all members are straight, and, providing joints are not semi-continuous as defined in BS EN 1993-1-8:2005, Clause 5, that either:

- a) all members are pin jointed and each joint lies at the intersection of the centroidal axes of the relevant members and all loads, including the self-weight of members, are applied at the joints; or
- b) the joints are stiff.

When considering the limit state of serviceability, either method b) should be used, or method a), modified by the inclusion of flexural stresses due to axial deformation, self-weight of the members and the stiffness of joints. If, at a joint, the centroidal axes of the adjacent members do not meet at a single point, the resulting flexural stresses in the members should be taken into account.

For members with loads not applied at truss joints, account should be taken of the following:

- a) the resulting stresses when a load is applied to a member in the plane of a truss other than at a joint;
- b) torsion and lateral flexure effects when the applied load is not in the plane of the truss. Where the load is applied to a cross member, the effect of interaction between the cross member so loaded and the truss and adjacent cross members should be taken into account.

The stiffnesses in BS EN 1993-2:2006, Table D.3, make no allowance for joint flexibility in semi-continuous joints in U-frame construction. In such cases, the flexibility of the joints should be included in the calculation of the stiffness C in accordance with BS EN 1993-1-8.

Supplementary guidance on the assessment of fatigue of bridges has been given in PD 6695-1-9.

4.4 Main beam splices and bracing member connections

Slip of bolts need not be considered at main beam splices in global analysis, but slip at connections of bracing systems should be considered in the design unless all connections are Category C, in accordance with BS EN 1993-1-8.

4.5 Imperfections

Imperfections in common planarity of bearings should be allowed for in the analysis of torsional moments and reactions for torsionally stiff superstructures, and should be compatible with tolerances, which the designer should specify, and the construction method and sequence used.

Where restraint forces are to be transmitted to end supports by a combination of plan bracing and discrete torsional bracing, the plan bracing system should also be designed to resist the forces F_{Ed} in accordance with BS EN 1993-2:2006, **6.3.4.2(5)** from each torsional restraint within a length equal to the half wavelength of buckling.

5 Non-dimensional slenderness for beam segments between effective lateral restraints

5.1 General

BS EN 1993-1-1 requires that a reduction factor (χ_{LT}) be applied to the moment resistance of the cross-section to give the lateral torsional buckling moment resistance ($M_{b,Rd}$). χ_{LT} is determined from a factor (Φ_{LT}) and the non-dimensional slenderness factor ($\bar{\lambda}_{LT}$). The expression given for $\bar{\lambda}_{LT}$ is:

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}};$$

where:

$W_y f_y$ is the moment resistance for the section;

M_{cr} is the elastic critical moment for lateral torsional buckling used as the basis for the methods given in design codes for determining the slenderness of a section;

This section presents a method for determining $\bar{\lambda}_{LT}$ without explicit determination of M_{cr} for segments of beams that are of uniform section between effective lateral restraints. The method takes account of section geometry and bending moment distribution.

The method is appropriate only to steel beams that are unrestrained over the length of the segment; i.e. where there is no connection to a deck or other lateral restraint, either continuous or discrete, at any level in the cross-section over the length of the segment.

5.2 Slenderness

For straight segments of beams $\langle A_1 \rangle$, other than those with a closed cross-section or a solid rectangular section, $\langle A_1 \rangle$ with effective lateral restraints to the compression flange at both ends of the segment, the value of λ_{LT} required by BS EN 1993-2:2006, 6.3.2.2 may be taken as:

$$\lambda_{LT} = \frac{1}{\sqrt{C_1}} UVD \frac{\lambda_z}{\lambda_1} \sqrt{\beta_w};$$

where:

C_1 is a parameter dependent on the shape of the bending moment diagram, such that the value of M_{cr} for the segment with the actual bending moment diagram is equal to C_1 times that for the same segment with a uniform bending moment along its full length. Values of $1/\sqrt{C_1}$ are given in Figure 1. Rigorous analysis may be used to obtain C_1 ;

D is a parameter to allow for the destabilizing effect of the applied loading. $D = 1,2$ if the load is applied to the top flange and both the flange and the load are free to move laterally. Otherwise $D = 1,0$;

U is a parameter dependent on the section geometry. Conservatively, U may be taken as 1,0 or, for doubly symmetric hot rolled sections, $U = 0,9$;

V is a parameter related to the slenderness and section geometry, given by:

$$\langle C_1 \rangle V = \left[\left\{ 4a(1-a) + 0,05\lambda_F^2 + \psi_a^2 \right\}^{0,5} + \psi_a \right]^{-0,5} \langle C_1 \rangle \quad \text{in which:}$$

$$\psi_a = 2a - 1 \text{ when } I_{z,c} < I_{z,t} \text{ and } \psi_a = 0,8(2a - 1) \text{ when } I_{z,c} \geq I_{z,t};$$

$$\lambda_F = \frac{L}{i_z} \times \frac{t_f}{h};$$

h is the depth of the cross-section;

L is the distance between points of restraint to the compression flange (the length of the segment);

t_f is the mean thickness of the two flanges of an I or channel section, or the mean thickness of the table of a tee, or leg of an angle section;

$I_{z,c}$, $I_{z,t}$ are the second moments of area of the compression and tension flanges, respectively, about their Z-Z axis;

$$a = \frac{I_{z,c}}{I_{z,c} + I_{z,t}};$$

$$\lambda_z = \frac{L}{i_z}, \text{ in which:}$$

i_z is the radius of gyration of the gross cross-section of the beam about the Z-Z axis;

$$\beta_w = \frac{W_y}{W_{pl,y}};$$

W_y is the modulus used to calculate $M_{b,Rd}$:

for Class 1 and 2 sections $W_y = W_{pl,y}$;

for Class 3 sections $W_y = W_{el,y}$;

for Class 4 sections $W_y = W_{eff,y}$;

$\lambda_1 = \pi \sqrt{\frac{E}{f_y}}$, in which f_y is the yield strength appropriate to the thickness of the steel.

[A1] NOTE For beams with a closed cross-section or a solid rectangular section, $\bar{\lambda}_{LT}$ can be determined using the generalized method given in 5.1. **[A1]**

5.3 Effective restraints

A discrete lateral restraint, or a plan bracing system, may be taken as fully effective provided that it has a stiffness such that $\delta_R < L^3/(40 EI_{z,c})$ where L and $I_{z,c}$ are as defined in 5.2 and δ_R is the displacement of the restraint due to a unit force acting laterally to the beam. Where restraint is provided by a plan bracing system between two beams, two unit forces should be applied, one to each beam, in such directions as to give the greatest value of δ_R . Where a plan bracing system restrains several positions along a beam, the displacement should be determined relative to the adjacent restraint positions.

Figure 1 Slenderness factor for variation in bending moment

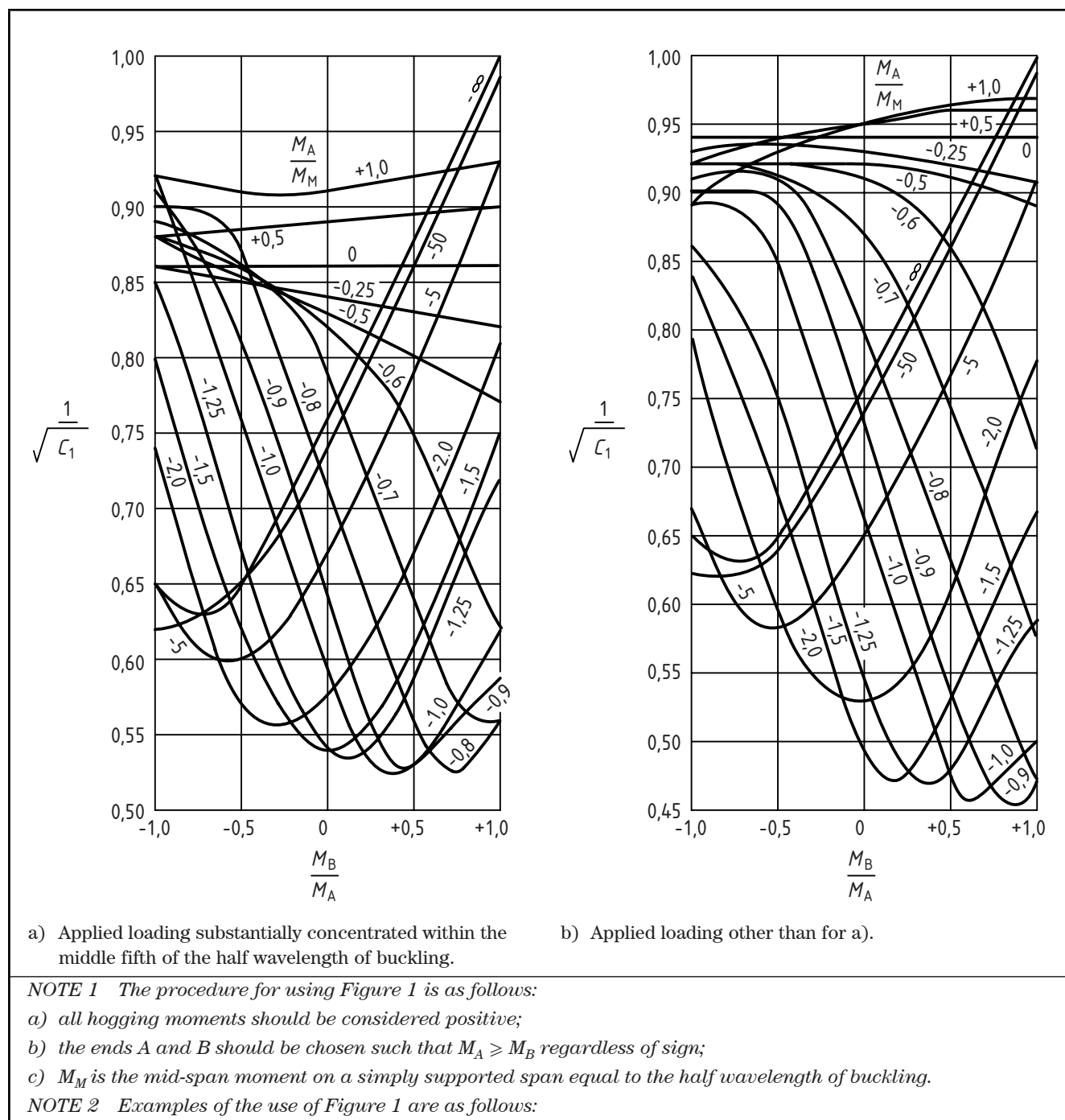
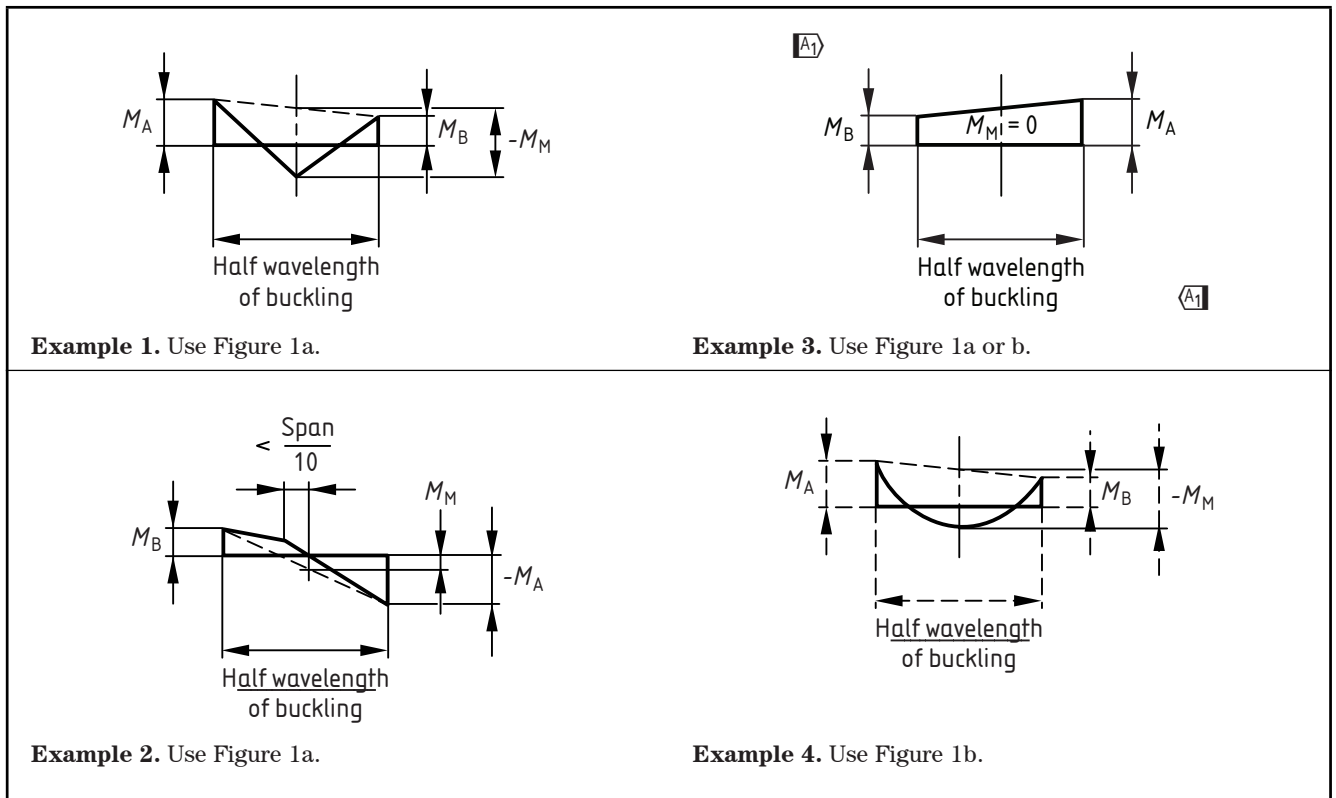


Figure 1 Slenderness factor for variation in bending moment (*continued*)



6 Non-dimensional slenderness for beams without intermediate restraints between support positions

6.1 General

The method outlined in 6.2 takes account of section geometry and bending moment distribution and of torsional flexibility at the supports. In bridges, unrestrained beams are usually found during the construction stage, before the deck of the bridge has been fixed or cast.

6.2 Slenderness

For a straight uniform beam $\boxed{A_1}$, other than those with a closed cross-section or a solid rectangular section, $\boxed{A_1}$ with effective lateral and torsional restraints at both ends of the span, the value of λ_{LT} required by BS EN 1993-2:2006, 6.3.2.2 may be taken as:

$$\bar{\lambda}_{LT} = \frac{1}{\sqrt{C_1}} UVD \frac{\lambda_z}{\lambda_1} \sqrt{\beta_w};$$

where:

C_1, D, U, V, h, t_f are as defined in 5.2. However in the derivation of V , λ_F should be defined as follows:

$$\lambda_F = \frac{kL}{i_z} \times \frac{t_f}{h};$$

where:

L is the span of the beam;

t_f, h, i_z is as defined in 5.2;

$\lambda_z = \frac{kL}{i_z}$, in which:

k is an effective length parameter, given by 6.3;

$$\beta_w = \frac{W_y}{W_{pl,y}};$$

W_y is the modulus used to calculate $M_{b,Rd}$:

for Class 1 and 2 sections $W_y = W_{pl,y}$;

for Class 3 sections $W_y = W_{el,y}$;

for Class 4 sections $W_y = W_{eff,y}$;

$\lambda_1 = \pi \sqrt{\frac{E}{f_y}}$, in which f_y is the yield strength appropriate to the thickness of the steel.

NOTE For beams with a closed cross-section or a solid rectangular section, $\bar{\lambda}_{LT}$ can be determined using the generalized method given in 5.1. **A1**

6.3 Effective length parameter

6.3.1 With a half wavelength over the span

The value of the effective length parameter k is the product of two parameters:

$$k = k_f k_e;$$

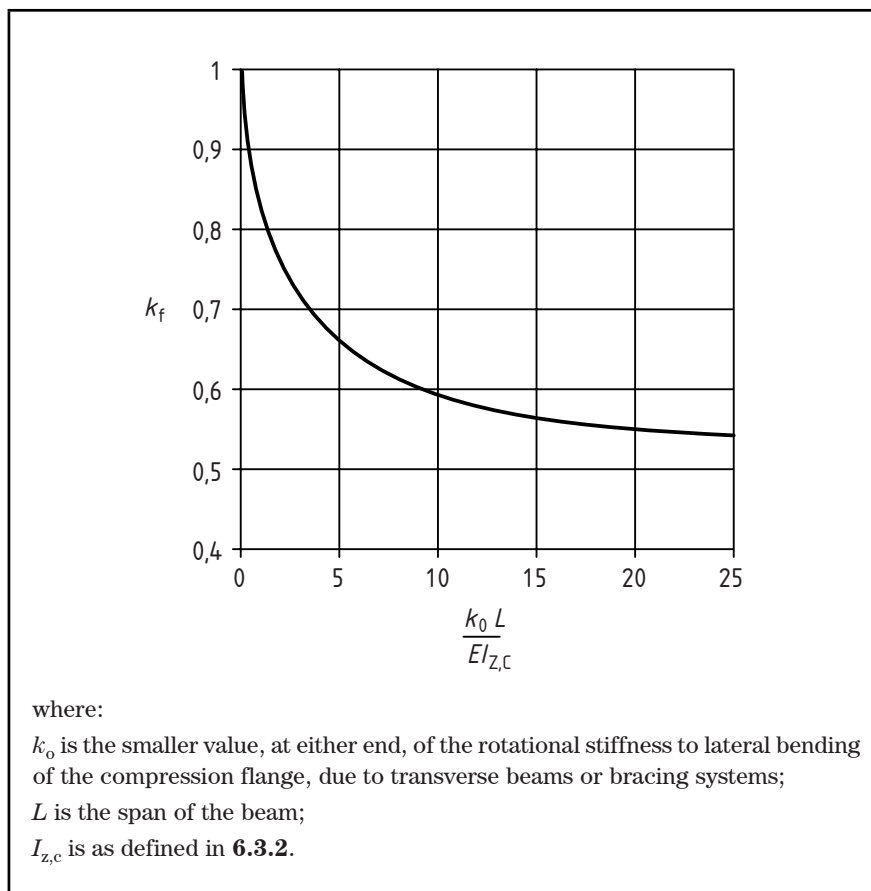
where:

k_f is a parameter that allows for the in-plane restraint of the compression flange at the supports; and

k_e is a parameter that allows for the torsional flexibility of the restraints at the supports.

The parameter k_f may conservatively be taken as 1,0 or a more accurate value, allowing for the degree of restraint in plan, may be obtained from Figure 2.

Figure 2 Effective length parameter k_f – effect of rotational end restraint



The parameter k_e is given by:

$$k_e = \sqrt{\frac{1}{1 - \frac{60Et_{fmax} \beta \delta_t R_v}{W \left(\frac{L}{i_z} \right)^3 V^4}}}$$

where:

t_{fmax} is the maximum thickness of the compression flange in the span;

δ_t is the larger relative lateral deflection at a support, at either end of the span under consideration, of the centroid of one flange of the beam with respect to the centroid of the other flange which would occur when equal and opposite unit forces act laterally on the torsional restraint at a support only at the same levels as shown in Figure 6. Where the end restraint to two or more beams is provided by a common lateral member interconnecting their ends, the value of δ_t should be calculated for equal and opposite unit forces applied at the end of each beam applied in directions to produce the greatest value of δ_t at the end of any of the beams;

β is 1,0 for a simply supported beam, or for an internal support to a continuous beam without restraint in plan to the compression flange at the support; or 2,0 for an internal support to a continuous beam with restraint in plan at that support;

W is the total vertical load on the span;

R_v is the vertical reaction at the support under consideration;

V is the value of V calculated in accordance with 6.2, which may be derived using $k = 1,0$ in calculating λ_F ;

i_z is as defined in 5.2.

NOTE 1 k_e should be taken as the greater of the values obtained for either support.

NOTE 2 The restraint should be such that the denominator has a positive sign.

6.3.2 With a half wavelength over two spans

Where spans are continuous and the flexibility of the torsional restraints at the supports is relatively high, a greater effective length and a lower overall slenderness may occur when the mode of buckling is such that the half wavelength extends over two spans. For buckling in this mode, the effective length kL to be used in the calculation of λ_F in 6.2 is given by:

$$kL = k_f \Sigma L \sqrt{\frac{1}{1 + 2 \frac{(\Sigma L)^3}{\pi^4 EI_{z,c} \left(\delta_i + \frac{\delta_e}{2} \right)}}};$$

where:

k_f is as defined in 6.3.1;

ΣL is the sum of the two adjacent spans;

$I_{z,c}$ is the second moment of area of the compression flange about its centroidal axis parallel to the web of the beam at the point of maximum bending moment;

δ_i is the value of δ_i (as defined in 6.3.1) for middle support;

δ_e is the average value of δ_i (as defined in 6.3.1) for the supports at the opposite ends of the adjacent spans.

NOTE When this mode is being considered, the value of the parameter C_1 in 6.2 should be taken as 1,0, irrespective of the shape of the bending moment diagram.

7 Non-dimensional slenderness for cantilever beams without intermediate lateral restraints

7.1 General

The method takes account of section geometry and restraint conditions at the support and at the cantilever tip. Unrestrained cantilever beams may occur during construction, before the deck is fixed or cast, but may also occur as a permanent detail, supporting other structural elements of the bridge. The tip of the cantilever may or may not be restrained laterally, in either case.

7.2 Slenderness

For straight segments of cantilever beams, $\overline{\lambda}_1$ other than those with a closed cross-section or a solid rectangular section, $\overline{\lambda}_1$ the value of λ_{LT} required by BS EN 1993-2:2006, 6.3.2.2, may be taken as:

$$\overline{\lambda}_{LT} = UV \frac{\lambda_z}{\lambda_1} \sqrt{\beta_w} ;$$

where:

U, V is as defined in 5.2;

$$\alpha = \frac{I_{z,c}}{I_{z,c} + I_{z,t}} ;$$

$$\lambda_F = \frac{k_c L}{i_z} \times \frac{t_f}{h} ;$$

h is the depth of the cross-section;

L is the length of the cantilever;

t_f is the mean thickness of the two flanges of an I or channel section, or the mean thickness of the table of a tee or leg of an angle section;

$I_{z,c}, I_{z,t}$ is as defined in 5.2;

$$\lambda_z = \frac{k_c L}{i_z} , \text{ in which:}$$

k_c is an effective length parameter (see 7.3);

$$\beta_w = \frac{W_y}{W_{pl,y}} ;$$

W_y is the modulus used to calculate $M_{b,Rd}$:

for Class 1 and 2 sections $W_y = W_{pl,y}$;

for Class 3 sections $W_y = W_{el,y}$;

for Class 4 sections $W_y = W_{eff,y}$;

$\lambda_1 = \pi \sqrt{\frac{E}{f_y}}$, in which f_y is the yield strength appropriate to the thickness of the steel.

NOTE For beams with a closed cross-section or a solid rectangular section, $\overline{\lambda}_{LT}$ can be determined using the generalized method given in 5.1. $\overline{\lambda}_1$

7.3 Effective length parameter

The value of the effective length parameter k_c is dependent on the restraint conditions at supports and the tip of the cantilever as well as the position of the load, i.e. whether on tension flange where there is no lateral restraint to load or flange or any other location in the section of the beam. The value should be obtained from specialist literature or text books.

8 Non-dimensional slenderness for beams with intermediate torsional restraints

8.1 General

The method takes account of section geometry and bending moment distribution. Such torsional restraint normally only occurs during the construction stage, before the deck is fixed or cast, when beams are restrained by pairing them together with “channel” or “triangulated” bracing at intervals along the span.

8.2 Slenderness

For straight uniform beams $\boxed{A_1}$, other than those with a closed cross-section or a solid rectangular section, $\boxed{A_1}$ with equally spaced torsional restraints but without lateral restraints, the value of λ_{LT} required by BS EN 1993-2:2006, **6.3.2.2**, may be taken as:

$$\lambda_{LT} = \frac{1}{\sqrt{C_1}} UVD \frac{\lambda_z}{\lambda_1} \sqrt{\beta_w};$$

where:

C_1 is as previously defined in **5.2** except that it relates to the half wavelength of buckling;

NOTE Conservatively, C_1 may be taken as 1,0.

U, V, D are as previously defined in **5.2**;

$$\alpha = \frac{I_{z,c}}{I_{z,c} + I_{z,t}};$$

$$\lambda_F = \frac{L_w}{i_z} \times \frac{t_f}{h};$$

L_w is the half wavelength of buckling (see **8.3**);

h is the depth of the cross-section;

t_f is the mean thickness of the two flanges of an I or channel section, or the mean thickness of the Table of a tee or leg of an angle section;

$I_{z,c}, I_{z,t}$ is as defined in **5.2**;

$$\lambda_z = \frac{k_t L_w}{i_z}, \text{ in which:}$$

k_t is an effective length parameter, given by **8.5**;

$$\beta_w = \frac{W_y}{W_{pl,y}};$$

W_y is the modulus used to calculate $M_{b,Rd}$:

for Class 1 and 2 sections $W_y = W_{pl,y}$;

for Class 3 sections $W_y = W_{el,y}$;

for Class 4 sections $W_y = W_{eff,y}$;

$\lambda_1 = \pi \sqrt{\frac{E}{f_y}}$, in which f_y is the yield strength appropriate to the thickness of the steel.

NOTE For beams with a closed cross-section or a solid rectangular section, $\bar{\lambda}_{LT}$ can be determined using the generalized method given in 5.1.

8.3 Half wavelength of buckling

With the form of restraint described in Clause 8, the mode of buckling is usually with a single half wave over the span. However, in some circumstances the particular combination of flexibility of the torsional restraints and their spacing may mean that a higher mode, with two or more half waves, may have a lower elastic critical moment. The beam should therefore be verified for at least the first two modes.

Thus $L_w = L, L/2, L/3$, depending on the mode, where L is the span of the beam.

8.4 Effective length parameter

When a beam without lateral restraints is provided with a central torsional restraint or a number of equally spaced torsional restraints of the same stiffness in a span, the effective length parameter k_t may be derived from Figure 3 as appropriate, using the restraint parameter:

$$\frac{V_{eq}^4 L_w^3}{EI_{z,c} \theta_R^2 d_f^2 (1-a)};$$

but for beams with multiple restraints, k_t should not be less than:

$$\frac{(1,7 - 0,7V_{eq})L_r}{L_w};$$

where:

d_f is the vertical distance between the centroid of the compression and tension flanges respectively at the position of the torsional restraint;

L_w is the half wavelength of buckling (see 8.3);

θ_R is the greatest value of the rotation of a restraint about the longitudinal axis of the beam, due to a torque equal to a unit torque multiplied by $1/n$, applied to each restraint. When restraint is provided by uniform diaphragms interconnecting beams, the value of θ_R should be taken as $\theta_{R1} + \theta_{R2}$ where:

- θ_{R1} is the rotation due to the flexibility of the diaphragm calculated as the greatest rotation about the longitudinal axis of a beam at a connection between the diaphragm and the beam under unit moments in the plane of the diaphragm multiplied by $1/n$, applied to each connection, in the same sense on each beam (see Figure 3);
- θ_{R2} is the greatest value of rotation of a beam at the middle of a half wavelength of buckling due to the vertical deflections of the beams. A unit torque multiplied by $1/n$ should be applied to each beam at each diaphragm connection, the diaphragms being assumed to be rigid for this calculation, in directions of opposite sense in consecutive waves and the same sense on each beam (see Figure 3);

n is the number of discrete restraints in the half wavelength of buckling (= 1,0 for a single restraint at the centre of a half wavelength);

V_{eq} is a parameter that takes account of warping properties of the section and is calculated as follows:

- a) for sections symmetrical about both their major and minor axis,
 $V_{eq} = V$ where the value of V is determined in accordance with 8.2;
- b) for sections symmetrical about their minor axis only;

$$V_{eq} = \left[\frac{2 a \chi}{(\sqrt{4 + \tau \chi} + \psi_i \sqrt{\chi})^2} \right]^{0.25};$$

where:

$$\tau = 4a(1 - a) + \psi_a^2;$$

a and ψ_i are as defined in 5.2;

$$\chi = \frac{\pi^2 d_f^2 EI_z}{GI_T L_w^2};$$

I_z is the second moment of area of the beam about its minor axis;

I_T is the St Venant torsional constant.

When such restraint is provided by diaphragms interconnecting two or more beams the rotations should be calculated assuming the above torque to be applied in the same rotational direction by equal and opposite horizontal forces on the diaphragms. In such cases the restraining torques must be resisted by equal and opposite vertical forces on the connected beams, equal to the value of the torque divided by the beam spacing, and account should be taken of the deflections of the beams due to all the restraining torques, w_R being taken as that at the restraint where there is the greatest total rotation.

L is the span of the beam;

L_r is the spacing of the torsional restraints ($= L/(1 + n)$);

$I_{z,c}$ is the second moment of area of the compression flange about the z-z axis.

NOTE 1 The effective length relationships in Figure 3 do not apply when considering buckling with a half-wavelength equal to L_r . In such a case, the slenderness should be determined assuming that there is fully effective intermediate lateral restraint.

NOTE 2 For beams with a single central restraint, L_w should be taken as equal to L , unless the restraint parameter derived from that value is such that it is to the right of the diagonal arrow on the appropriate curve for the value of V_{eq} in which case either:

- a) $k_t L$ and L_w should be taken as $L/2$;

or

- b) L_w should be taken as L and k_t should be derived from Figure 3, whichever gives the lower moment of resistance.

NOTE 3 Where, for multiple restraints, the value of $V_{eq}^4 L_w^3 / [EI_{z,c} \theta_R d_f^2 (1 - a)]$ exceeds the maximum value shown in Figure 3, the procedure given in 8.5 may be used to derive the appropriate value of k_t .

NOTE 4 When only pairs of beams are interconnected by diaphragms, θ_{R2} may be taken as $(n + 1)L_w^3 / 24nEI_y B^2$ where I_y is the second moment of area of each beam about its major axis, B is the spacing of the beams.

Figure 3 Effective length parameter for beams with discrete torsional restraints

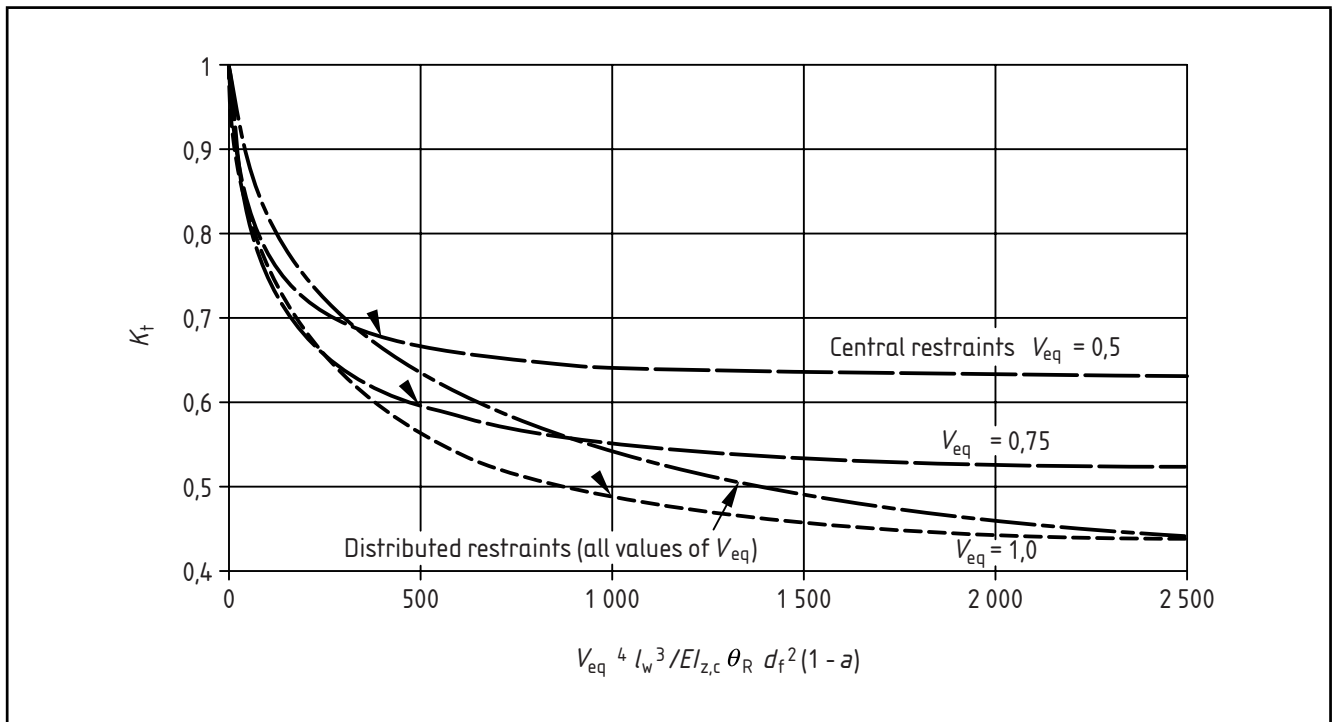
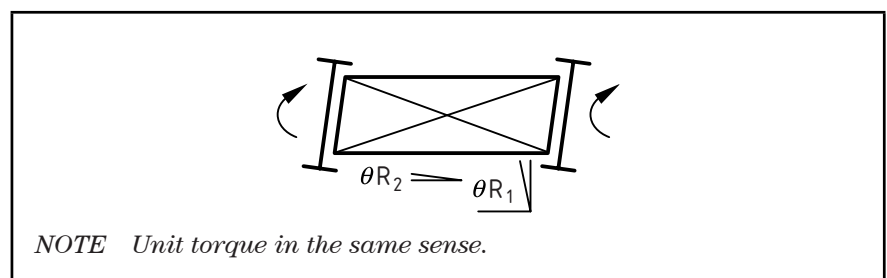


Figure 4 Rotations of paired beams subject to unit torques



8.5 Equations for deriving k_t

8.5.1 Relationship for multiple torsional restraints

The value of k_t is given by:

$$k_t = \left[1 + \frac{V_{eq}^4 L_w^3}{\pi^4 EI_{z,c} d_f^2 \theta_R (1-a)} \right]^{-0,25} ;$$

where the parameters are as defined in 8.4.

8.5.2 Relationship for single central torsional restraint

The relationships k_t , V_{eq} and $V_{eq}^4 L_w^3 / [EI_{z,c} \theta_R d_f^2 (1-a)]$ can be derived from the following equation:

$$2\pi^2 (1 - V_{eq}^4) \left[\frac{\alpha \left(2\mu (nL)^3 \cosh \frac{nL}{2} - (mL)^3 \cos \frac{mL}{2} \right)}{\left(\sin \frac{mL}{2} + 2\mu \sinh \frac{nL}{2} \right)} \right] = \frac{V_{eq}^4 L_w^3}{\theta_R EI_{z,c} d_f^2 (1-a)},$$

where:

$$mL = \sqrt{-\frac{1}{2\alpha} + \sqrt{\frac{1}{4\alpha^2} + \frac{\pi^2 c^2 (1 + \pi^2 \alpha)}{\alpha}}} \quad \text{and}$$

$$nL = \sqrt{\frac{1}{2\alpha} + \sqrt{\frac{1}{4\alpha^2} + \frac{\pi^2 c^2 (1 + \pi^2 \alpha)}{\alpha}}}, \quad \text{in which:}$$

$$\alpha = \frac{V_{eq}^4}{\pi^2 (1 - V_{eq}^4)} \quad \text{for } V_{eq} \leq 0.999 \text{ or otherwise } = 25;$$

$$c = \sqrt{\frac{1}{k_t}};$$

$$\mu = - \frac{mL \cos \left(\frac{mL}{2} \right)}{2nL \cosh \left(\frac{nL}{2} \right)};$$

and the other parameters are as defined in 8.4.

These expressions may be used directly to derive the restraint stiffness parameter k_t required to achieve a given effective length for buckling with a half wavelength equal to L .

The value of the effective length at which the critical buckling moment for a beam with a central restraint and a half wavelength equal to L equals that for buckling in the second mode with a half wavelength equal to $L/2$ is given by:

$$k_t = \left[\frac{1 + \pi^2 \alpha}{4(1 + 4\pi^2 \alpha)} \right]^{0.25}.$$

The values of k_t for three values of V_{eq} (0,5, 0,8 and 1,0) are shown on Figure 3.

9 Simplified method for verification of lateral buckling of truss chords and flanges in compression

Additional guidance is given on the use of BS EN 1993-2:2006, **6.3.4.2**, for cases where there are flexible intermediate restraints. It also provides guidance on design where there are flexible end restraints, which is not specifically covered by BS EN 1993-2:2006, **6.3.4.2**, other than through direct global analysis of the structure.

Where the connections are “semi-continuous” in accordance with BS EN 1993-1-8, the stiffness of the joint should be calculated in accordance with BS EN 1993-1-8 or determined by tests and included in the calculation of spring stiffness C_d . The spring stiffness may be determined from:

$$C_d = 1/\delta_R;$$

where:

δ_R is equal to the lateral deflection which would occur in any intermediate U-frame at the level of the centroid of the flange being considered, when a unit force acts laterally to the U-frame only at this point and simultaneously at each corresponding point on the other flange or flanges connecting to the same U-frame. The direction of each unit force should be such as to produce the maximum aggregate value of δ_R . The U-frame deflection should be calculated allowing for joint flexibility between the cross member and the verticals as required above.

In the absence of specific calculation of joint stiffness, the following joint rotational flexibilities, f , between the cross member and the verticals of the U-frame may be used, expressed in radians per unit moment:

- a) $0,5 \times 10^{-10}$ rad/Nmm when the cross member is bolted or riveted through unstiffened end-plates or cleats; or
- b) $0,2 \times 10^{-10}$ rad/Nmm when the cross member is bolted or riveted through stiffened end plates; or
- c) $0,1 \times 10^{-10}$ rad/Nmm when the cross member is welded right round its cross-section or the connection is by bolting or riveting between stiffened end-plates on the cross member and a stiffened part of the vertical.

The above values are generally conservative as they were derived from studies of shallow members.

The effects on U-frame stiffeners and main beam compression flanges caused by local loading on cross members should be considered. A method for calculating these effects is provided in Clause 11.

If the end supports defining the length L as defined in BS EN 1993-2:2006, **6.3.4.2(6)** are not fully rigid in accordance with BS EN 1993-2:2006, **6.3.4.2**, expression (6.5), either the slenderness should be determined in accordance with BS EN 1993-2:2006, **6.3.4.2(3)**, or the definition of m as required in BS EN 1993-2:2006, **6.3.4.2(6)**, may be modified as follows:

$$m = \frac{\sqrt{\gamma}}{\left(\frac{\pi}{\sqrt{2}} + \frac{0,69}{X + 0,5}\right)^2};$$

with $X = \frac{C_e}{\sqrt{2}} \left(\frac{l^3}{C_d^3 EI} \right)^{0,25}$ and C_e is the stiffness of the end support

determined in the same way as the stiffness of intermediate supports, C_d , other terms are as defined in BS EN 1993-2:2006, **6.3.4.2**.

If a deck slab in steel-concrete composite construction is assumed to contribute to c (as defined in BS EN 1993-2:2006, **6.3.4.2**), the main beam web, shear studs and deck slab should be checked for the moments and forces commensurate with this assumption.

10 Restraints at supports – effects due to restraint of main beams

10.1 General

Simplified guidance on torsional restraints at supports is provided because BS EN 1993 does not cover the subject other than through the carrying out second order analysis of the bracing system with modelled imperfections. In undertaking this method designers should use the critical distribution of imperfections.

The design of restraints provided to beams to prevent lateral torsional buckling should be compatible with the assumptions made in the beam buckling verification.

All beams, including cantilever beams, should be restrained against rotation about their own axes at each support in accordance with the appropriate provisions and **10.2** and **10.3**.

10.2 Restraining forces

10.2.1 General

The restraining system should be capable of resisting, in addition to the co-existent effects of wind, frictional and other applied forces, two equal and opposite forces F_s applied normal to the beam and in the planes of its two flanges.

Where two or more beams are restrained by a common lateral member interconnecting their ends, pairs of such forces, from the beam being considered and from one adjacent beam, should be taken at the end of each beam, applied such as to produce the most severe effect in the restraining member.

The value of each force F_s in a direction normal to the longitudinal axis of the beam should be taken as follows:

$$F_s = F_{s1} + F_{s2} + F_{s3} + F_{s4}$$

where:

F_{s1} , F_{s2} , F_{s3} and F_{s4} are as given by **10.2.2** to **10.2.4** below.

10.2.2 Force due to bow of compression flange

The force F_{s1} on a support due to the end torque on the beam resulting from the initial bow of the compression flange may be taken as:

$$F_{s1} = 0,006 \frac{M}{d_f \left(1 - \left(\frac{\sigma_{fc}}{\sigma_{crit}} \right)^2 \right)}.$$

10.2.3 Force due to non verticality of web at supports

The force F_{s2} on the support due to magnification resulting from compressive force in the flange of the initial departure from verticality of the supports may be taken as:

$$\frac{\beta(\Delta_{e1} + \Delta_{e2}) \sigma_{fc}}{(\sigma_{crit} - \sigma_{fc}) \sum \delta}.$$

10.2.4 Force due to eccentricity of bearing reaction

The force F_{s3} at a support due to the eccentricity of the lateral location of the centre of the applied loading relative to the centre of bearing reaction resulting from initial out of verticality of the support combined with that due to change in slope of an end of a beam bearing on a skew may be taken as:

$$\frac{Rd_L [\Delta/D + \theta_L \tan \alpha]}{D}.$$

10.2.5 Force due to twist at skew supports

The force F_{s4} at a support due to twisting of a beam without intermediate restraints caused by changes in longitudinal slope of the beam on bearings aligned with skew supports may be taken as follows:

a) For a simply supported non composite beam:

$$F_{s4} = \frac{GI_T(\theta_{LA} \tan \alpha_A + \theta_{LB} \tan \alpha_B)}{d_f L \left(1 + \frac{GI_T}{Ad_f^2} \right)}.$$

b) For an end support to a two span continuous non-composite beam without intermediate restraints or one side of a support adjacent to an internal support in a multi span non-composite continuous beam without intermediate restraints,

either:

$$F_{s4} = \frac{KEI_{z,c} d_f (\theta_{LA} \tan \alpha_A + \theta_{LB} \tan \alpha_B)}{L^3 \left(1 + \frac{GI_T}{Ad_f^2} \right)};$$

or

$$F_{s4} = \frac{CGI_T(\theta_{LA} \tan \alpha_A + \theta_{LB} \tan \alpha_B)}{d_f L \left(1 + \frac{GI_T}{Ad_f^2} \right)}.$$

NOTE 1 The two expressions for F_{s4} give the same values and either may be used.

- c) For beams rigidly attached to a composite deck by shear connectors the force to be resisted by an end support due to skew support should be taken as:

$$F_{s4} = \frac{D\theta_{LA} \tan \alpha_A}{\lambda \delta'_R}.$$

When the flanges not connected to the composite deck are restrained against plan rotation, the value of F_{s4} should be doubled.

NOTE 2 When there is a flexible intermediate restraint only at midspan, the restraint should be ignored for the purposes of deriving the value of F_{s4} according to the given expression.

- d) For an internal support in a continuous beam F_{s4} is the algebraic difference between the values of F_{s4} on each side of the support, where:

$$A = \frac{L}{(\delta_{e1} + \delta_{e2})} \text{ for a simply supported beam; or}$$

$$A = \frac{L}{(2\delta_i + \delta_e)} \text{ for a continuous beam;}$$

Δ is the initial lateral deflection of the compression flange relative to the tension flange at the support, which should be taken as $D/200$ or to BS EN 1090-2;

Δ_{e1} , Δ_{e2} are the values of Δ in opposite directions at the supports at each end of a simply supported span or at the end and internal supports or restraints respectively for a continuous beam;

δ_{e1} , δ_{e2} are the values of δ_e for an end support or for an internal support, as appropriate, as defined in **6.3.2**;

$$\begin{aligned} \Sigma \delta &= \delta_{i1} + \delta_{i2} \text{ for a simply supported span;} \\ &= \delta_e + 2\delta_i \text{ for a continuous beam;} \end{aligned}$$

L is the distance between the support considered and either the support at the other end of the span considered or any effective intermediate torsional restraint within that span, whichever is the lesser;

d_L is the vertical distance between the levels of the bearing support and the applied loads respectively. For composite beams the levels of the applied loads should be taken as those of the tops of the steel beams. For cambered beams d_L should be taken as the sum of the individual applied loads multiplied by their relevant values of d_L , divided by the total applied loads;

G is the shear modulus;

I_T is the St Venant torsion constant for the beam;

$I_{z,c}$ is as defined in **8.4**;

M is the largest bending moment occurring at a support or within lengths L_w either side of it, whether hogging or sagging;

σ_{fc} is the maximum compressive stress in the flange, averaged over the whole flange width, either at the support under consideration or in the span either side of it;

δ_{i1} , δ_{i2} are the values of δ_i for an end support at each end of a simply supported beam as defined in **6.3.1**;

δ_e, δ_i are the values for an end support and internal support respectively for a continuous beam as defined in **6.3.2**;

σ_{crit} is as defined in Clause **11**;

D is the overall depth of the beam at the support;

d_f is as defined in **8.4**;

R is the bearing reaction;

$\beta = 1$ for an end support;

$\beta = 2$ for an internal support or restraint in continuous beams with lateral restraints;

θ_L is the change in longitudinal slope of the beam adjacent to a support;

θ_{LA}, θ_{LB} are the coincident changes in the longitudinal slopes of the beam adjacent to the supports in the span considered, taken as positive when in a sagging direction away from the relevant support, and with θ_{LA} taken as the slope change adjacent to the support being considered and θ_{LB} as that at the opposite end of the span;

C, K are obtained from Figure 5;

NOTE For a two span beam the values of C and K are related to $(d_f/L)(I_{z,c}/I_T)^{0,5}$. For an internal support in a multi span continuous beam they are obtained from Figure 5 by replacing $(d_f/L)(I_{z,c}/I_T)^{0,5}$ by $(2d_f/L)(I_{z,c}/I_T)^{0,5}$.

α is the angle of skew, i.e. between the normal to the longitudinal axis of the beam and the axis of the support in plan for a skewed bridge;

α_A is the value of α at the support being considered;

α_B is the value of α at the other end of the span being considered;

$$\lambda = \left(\frac{1}{4\delta'_R EI_F} \right)^{0,25};$$

I_F is the lateral second moment of area of the bottom flange;

δ'_R is the value of $\delta_R l$;

l is the spacing of U-frames.

For beams with intermediate U-frame restraints δ_R is defined in Clause **9**.

For beams without intermediate restraints and which are restrained by the deck slab only, δ_R may be obtained from BS EN 1994-2:2005, **6.4.3.2**, and BS EN 1993-2:2006, **D.2.4**, or, in cases where the deck and webs of the beam are of constant thickness throughout the span, and the beam is of constant depth, it may be assumed that:

$$\delta_R = \frac{d_1^3}{3EI_1} + \frac{uBd_2^2}{EI_2};$$

where:

d_1 is the distance from the centroid of the compression flange to the nearest surface of the structural deck (see Figure 6);

d_2 is the distance from the centroid of the compression flange to the centroidal axis of the deck (see Figure 6);

$$I_1 = \frac{t_w^3}{12};$$

t_w is the thickness of the web of the beam;

I_2 is the second moment of area of the deck per unit length, about its axis of bending, neglecting concrete in tension but including reinforcement;

$u = 0,5$ for an outer beam; or

$= 0,33$ for an inner beam, if there are three or more beams interconnected by U-frames;

B is the distance between centres of consecutive beams, or the maximum distance when the beams are not exactly parallel.

Figure 5 Restraint forces – continuous beams

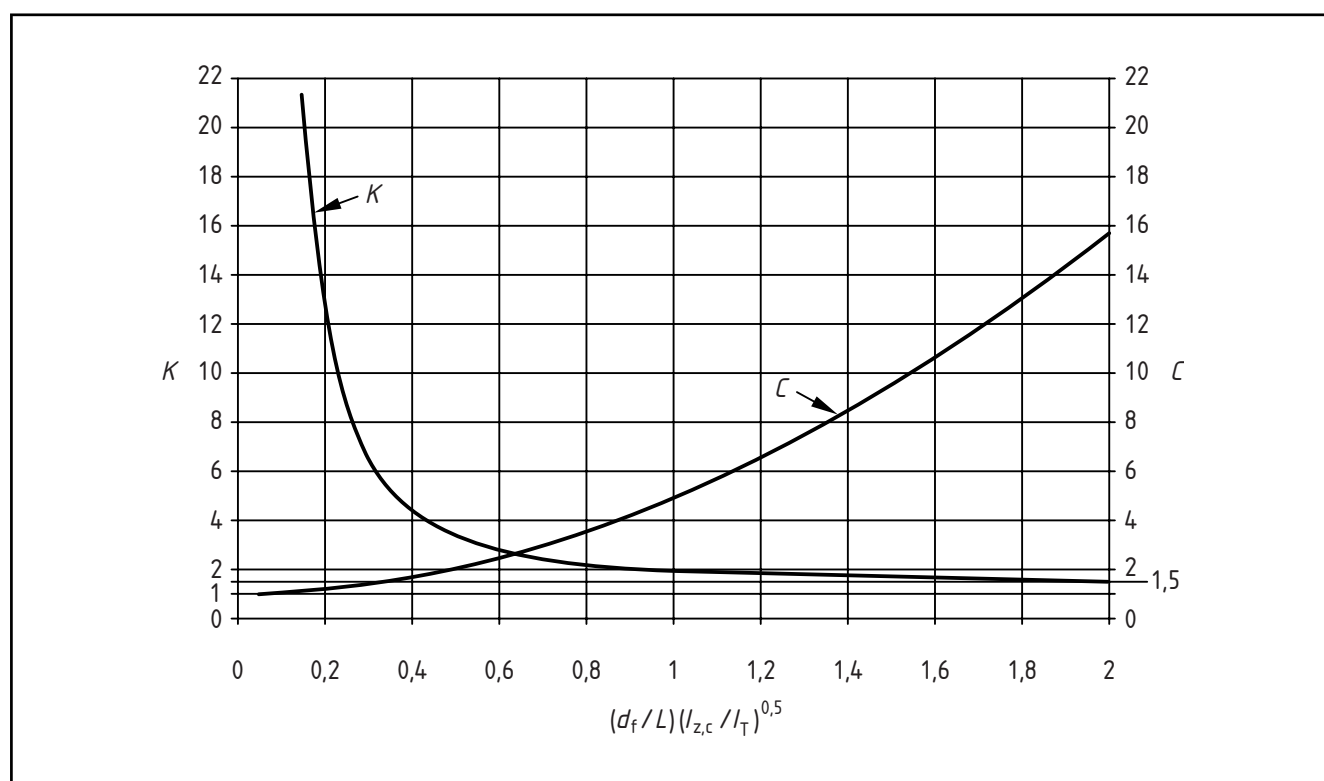
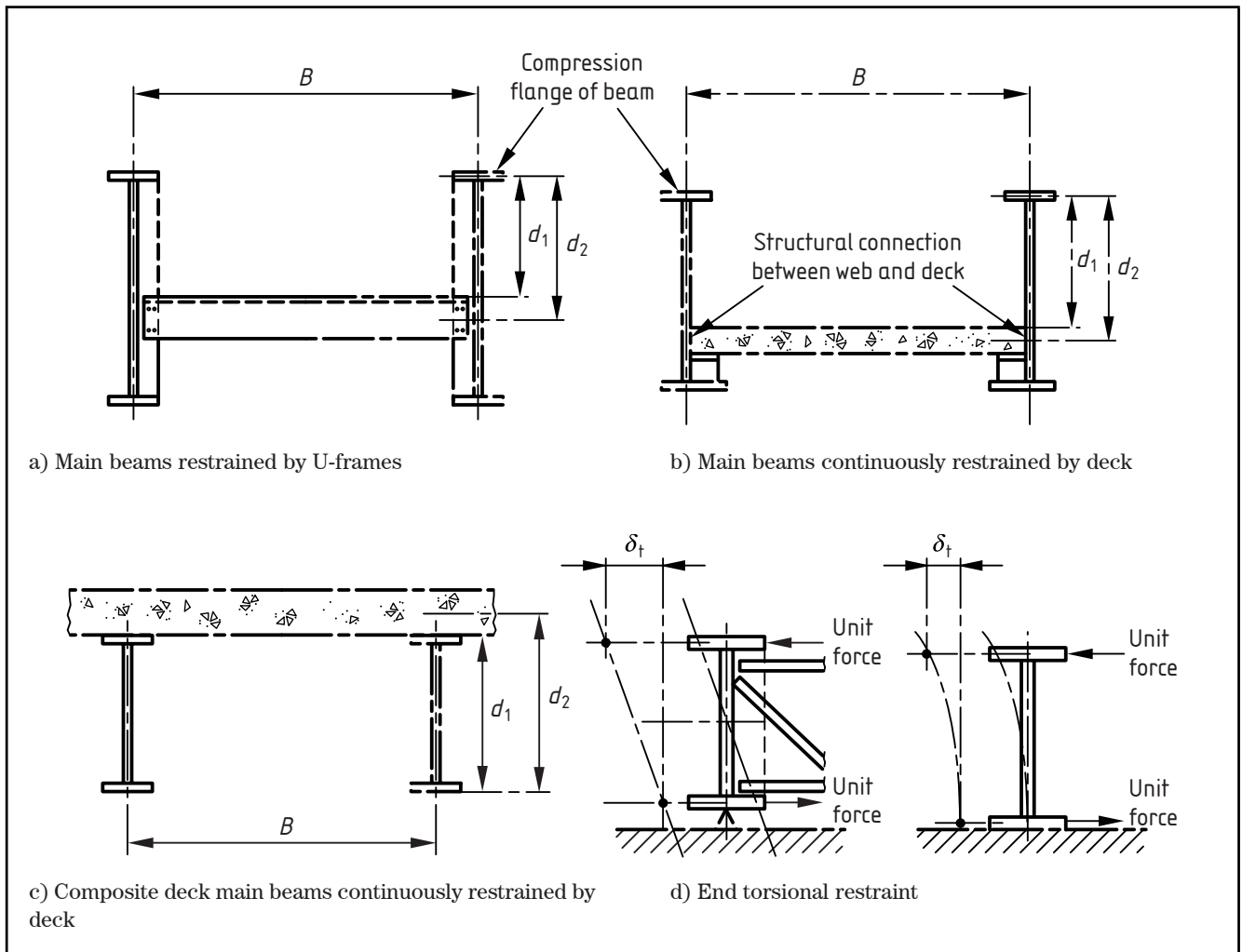


Figure 6 Restraint of compression flange by U-frames or deck or end torsional restraint



10.2.6 Additional force due to cross members in U-frames subjected to vertical loading

When the compression flange of a beam is restrained laterally between points of support by a system of U-frames with cross members subject to vertical loading, a force F_L should be added to F_S as defined above where F_L may be taken as follows:

$$F_L = \sum F_c \frac{(L_w - x)}{L_w} ;$$

where:

F_c is as defined in Clause 11 and derived for each loaded U-frame within a length L_w adjacent to the support considered;

x is the distance of the loaded frame from the support;

L_w is the half wavelength of buckling. Where intermediate lateral or U-frame restraints are provided which are not fully effective, L_w should be determined by taking L/L_w as the next integer below L/l_e but not less than unity;

Σ denotes summation for all loaded frames.