TITLE 2. STRUCTURAL ANALYSIS

CHAPTER V. STRUCTURAL ANALYSIS

Section 17. General

The structural analysis consists of obtaining the effect of actions on all or part of the structure in order to check the ultimate limit states and serviceability limit states defined in Section 8.

Such an analysis must be conducted for the different design situations given in Section 7 using adequate structural models that consider the influence of all relevant variables.

Section 18. Idealisation of the structure

18.1. Structural models

In order to conduct the analysis, both the geometry of the structure and the actions and support conditions are idealised by means of an adequate mathematical model, which must also roughly reflect the stiffness conditions of the cross-sections, members, joints and interaction with the ground.

The structural models must allow to consider the effects of movements and deformations in those structures or part thereof, where second-order effects increase the effects of the actions significantly.

In certain cases, the model must incorporate the following into its stiffness conditions:

- the non-linear response of the material outside the elastic analysis;
- the effects of shear lag in sections with wide flanges;
- the effects of local buckling in compressed sheet panels;

- the effects of the catenary (using a reduced modulus of elasticity, for example) and of displacement on structures with cables;

- the shear deformability of certain structural members;

- the stiffness of the joints;

- interaction between the ground and the structure.

Where it is necessary to conduct dynamic analyses, the structural models must also consider the properties of mass, stiffness, resistance and damping of each structural member, as well as the mass of other, non-structural, members.

Where it is appropriate to perform a quasi-static approximation of the structure's dynamic effects in accordance with the codes or regulations in force, such effects may be included in the static values of the actions, or dynamic amplification factors equivalent to such static actions could even be applied.

In some cases (e.g. vibrations caused by wind or earthquake), the effects of the actions may be obtained from linear elastic analyses using the modal superposition method.

Structural analyses for fire require specific models that are considered in Chapter XII.

In some cases, the results of the structural analysis may undergo marked variations regarding to possible fluctuations in some model parameters or in the design hypotheses adopted. In such cases, the Designer shall perform a sensitivity analysis that allows to limit the probable range of fluctuations in the structural response.

18.2. Member models

For purposes of the analysis, structural members are classified as one-dimensional when one of the dimensions is much greater than the others, two-dimensional when one of the dimensions is small in comparison with the other two, and three-dimensional when none of the dimensions is significantly greater than the others.

The Designer shall in each case select the most suitable member type to show the structural response satisfactorily.

The directrix of the member will usually follow the alignment of the elastic centres of mass of the cross-sections.

18.2.1. Design spans

Unless especially justified, the design span of a one-dimensional member shall be the distance between the support axes or the intersection points of its directrix and those of adjacent members.

18.2.2. Static shear magnitudes of cross-sections

The global structural analysis may in most cases be performed using the gross cross-sections of the members, based on their nominal dimensions.

For one-dimensional members, the static magnitudes to be considered are the area, the principal moments of inertia and the uniform torsion modulus.

The shear area and effects of distortion on the section and warping torsion need only be taken into account in some special cases.

The effects of shear lag in sections with wide flanges, and the effects of local buckling of compressed sheet panels, on member stiffness must be taken into account when these may have a significant impact on the results of the structural analysis.

The effect of shear lag on flanges may be taken into account by considering effective widths, in accordance with Section 21.

The effect of local buckling of compressed panels on member stiffness may be taken into account by means of equivalent effective sections in the case of slender cross-sections of class 4, in accordance with subsection 20.7.

In the case of sections with principal axes that do not coincide with the planes where load is acting, the Designer must use structural models that a correct estimation of the actual response of the members subject to biaxial bending.

When the shear force centre does not coincide with the centre of mass of the cross-section, mainly in open sections, the structural model must also take due account of the actions, static magnitudes and geometry of the members, so as to reproduce the effects of bending and torsion on the structure reliably, as well as any mutual interaction and load eccentricities.

18.2.3. Consideration of the effects of distortion on closed section members

In members subjected to torsion, and areas where significant concentrated loads are applied, the effects resulting from deformations owing to distortion of the cross-section must be considered if they are significant.

In order to monitor the magnitude of such effects in large closed sections (bridge box girders, for example), it will usually be necessary to have an internal stiffness system using transverse members called diaphragms, which may be frames, triangulations or plate girders.

The effects of distortion may be discounted when the actual stiffness or dimensions of the cross-section (hollow sections, for example), and/or of any diaphragms, limit the effects of the distortion, once they have been added, by less than 10 % of the material's reduced yield strength in the member in question under the relevant local or eccentric actions.

Where diaphragms are necessary, they must be designed for the stresses resulting from their stiffening effect on the closed section, for the torsion actions (under eccentric loads or in members of curve directrix in plan) or when they are close to concentrated loads (intermediate and supports), according to Annex 3.

In the presence of dynamic actions, the effects of distortion on the members and any diaphragms must always be considered when checking the structure’s fatigue limit state.

18.2.4. Consideration of the effects of mixed torsion on members with open or closed sections

The content of this subsection only applies directly to linear members subjected to torsion where the distance between points where there is no moment is equal to or greater than 2.5 times its depth, and the width is less than or equal to four times its depth, and the directrix is straight or curved.

The response of linear members to torsion, where the effects of distortion on the members may be discounted, is the sum of two mechanisms:

a) uniform or Saint-Venant torsion that only generates shear stresses in the cross-section and the stiffness of which is characterised by the torsion modulus It of the cross-section;

b) non-uniform or warping torsion that generates both direct and shear stress in the different sheet panels of the cross-section. Its stiffness remains characterised by its warping modulus, I_w.

The response of a member to torsion may be obtained through an elastic analysis that incorporates the general equations for mixed torsion, depending on the static torsional magnitudes of the cross-sections, I_t and I_w, the material deformation modulus, E and G, the connecting factors for rotation and warping at the ends of the member, and the distribution of torsion action along it. Alternatively, the structural analysis for torsion may be approached through finite elements models for the part.

It may be permitted for the effects of warping stress to be discounted in a suitably approximate way, to analyse just the uniform torsion in members in the following cases:

a) members that have freedom to warp at their extremities and which are required solely for moment at such extremities;

b) members in which the warping module of the cross-section, I_w, is of negligible or small magnitude in comparison with the torsion module, I_t. This is the case for the following:

– solid sections (round, square, rectangular, etc.);

– open cross-sections made up of rectangles that are sheared at a given point (angles, cross-shaped sections, single T units, etc.);

– closed cross-sections (tubes, single-cell or multi-cell boxes with no distortion, etc.).

Additionally, by way of simplification, it may be permitted for the effects of uniform torsion to be discounted, and only analyse the warping stress, in the case of beams with thin-walled open sections such as double T, U, H, Z sections, etc.

Where the static equilibrium of a structure basically depends on the torsion resistance of one or more members, such members shall mainly be designed using closed sections. In such cases, open sections cannot usually be recommended for resisting torsion loads, although for bridges or special parts that are straight or slightly curve on plan it may be possible to use double-beam or double-girder open sections that are designed to provide sufficient resistance to torsion resulting from eccentric actions.

However, in hyperstatic structures there are often open-section members (transverse beams for grids or twin box bridge decks, for example) being subject to torsion owing solely to the compatibility conditions resulting from differential bending between longitudinal members.

The effects of warping stress must be taken into account, where they are significant, for the checking on the structure's serviceability limit states and fatigue limit state, including those members subjected to compatibility torsion. For ultimate limit states, these effects need only be considered for members loaded with equilibrium torsion and members subjected to compatibility torsion, the stiffness of which under torsion has been considered in the calculation of forces for the global analysis of the structure and has a significant influence on the results of the calculation.

The use of structural models, mainly of bars, that only incorporate the uniform torsion stiffness of the members usually underestimates the effects of torsion in open sections. Where greater precision is required, for example in the case of slender sections or fatigue checking, I_t torsion modulus must be used, corrected so that they approximate the uniform torsion stiffness to the member's actual mixed-torsion stiffness, estimated by means of analytical solutions or sub-models of finite elements, under the actual loading and connecting conditions to which they will be subjected.

Table 18.2.4. Coordinates for the shear force centre, torsion modulus and warping modulus in some frequently used cross-sections

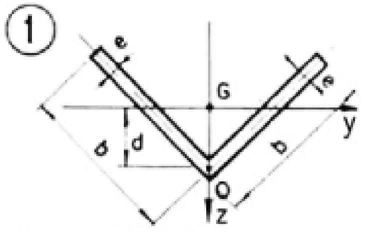
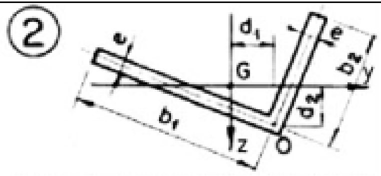
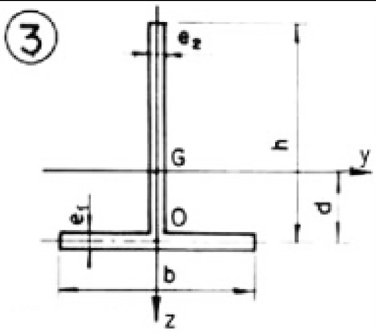
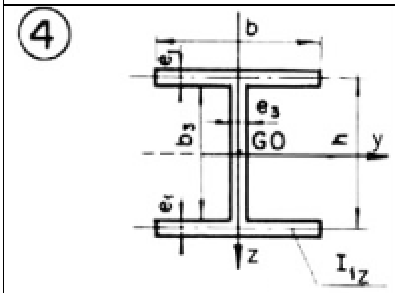
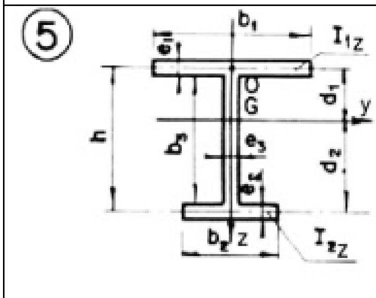
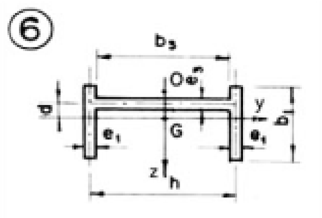
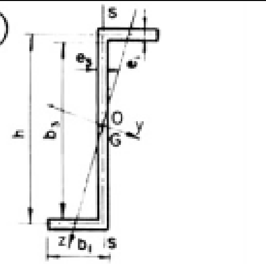
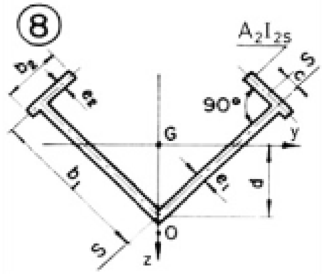
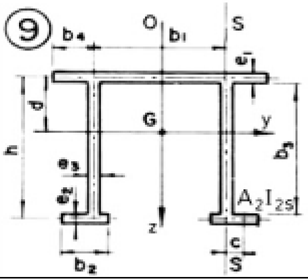
SECTION	COORDINATES OF THE SHEAR CENTRE	TORSION MODULUS I_T	WARPING MODULUS I_W
 ANGLES OF EQUAL SIDES	$y_0 = 0$ $z_0 = d$	$\frac{2}{3} b e^3$	$\frac{b^3 e^3}{18} = \frac{A^3}{144}$ The following may be assumed in practice: $I_W = 0$
 ANGLES OF UNEQUAL SIDES	$y_0 = d1$ $z_0 = d2$	$\frac{(b_1 + b_2) e^3}{3}$	$\frac{(b^3{}_1 + b^3{}_2) e^3}{36}$ The following may be assumed in practice $I_W = 0$
 SIMPLE T SECTION	$y_0 = 0$ $z_0 = d$	$\frac{b e_1^3 + b e_2^3}{3}$	$\frac{b^3 e_1^3}{144} + \frac{b^3 e_2^3}{36}$ The following may be assumed in practice: $I_W = 0$
 SYMMETRICAL DOUBLE T SECTION	$y_0 = 0$ $z_0 = d$	$\frac{2 b e_1^3 + b_3 e_3^3}{3}$	$I_{1_Z} \cdot \frac{h^2}{2} \approx I_Z \cdot \frac{h^2}{4}$
 ASYMMETRICAL DOUBLE T SECTION	$y_0 = 0$ $z_0 = \frac{d_2 \cdot I_{2_Z} - d_1 \cdot I_{1_Z}}{I_{1_Z} + I_{2_Z}}$ $\approx \frac{d_2 I_{2_Z} - d_1 I_{1_Z}}{I_Z}$	$\frac{b_1 e_1^3 + b_2 e_2^3 + b_3 e_3^3}{3}$	$\frac{h^2 \cdot I_{1_Z} \cdot I_{2_Z}}{I_{1_Z} + I_{2_Z}}$ $\approx \frac{h^2 \cdot I_{1_Z} \cdot I_{2_Z}}{I_Z}$

Table 18.2.4 (continued)

SECTION	COORDINATES OF THE SHEAR CENTRE	TORSION MODULE I_T	WARPING MODULE I_W
 FORMULAE THAT ARE ALSO VALID FOR LAMINATED C BEAMS	$y_0 = 0$ $z_0 = -d(1 + \frac{h^2 \cdot A}{4I_Z})$	$\frac{2b_1e_1^3 + be_3^3}{3}$	$\frac{h^2}{4}[I_y + d^2 \cdot A(1 - \frac{h^2A}{4I_Z})]$
	$y_0 = 0$ $z_0 = d$	$\frac{2b_1e_1^3 + be_3^3}{3}$	$\frac{h^2}{4} \cdot I_S$
	$y_0 = 0$ $z_0 = d +$ $+\sqrt{2}[+c \cdot b_1^2 \cdot \frac{A_2}{I_Z} -$ $- b_1 \frac{I_{2S}}{I_Z}]$	$\frac{2b_1e_1^3 + 2b_2e_2^3}{3}$	$[2d^2 - z_0^2]I_Z +$ $+ 2b_1(b_1 - 2d)I_{2S} +$ $+ 4db_1^2cA_2$
	$y_0 = 0$ $z_0 = -d(1 + \frac{b_1^2A}{4I_Z}) +$ $+ 2h \cdot \frac{I_{2S}}{I_Z}$	$\frac{(b_1 + 2b_4)e_1^3 + 2b_2e_2^3 + 2b_3e_3^3}{3}$	$\frac{b_1^2}{4}\left[I_y + d^2A(1 - \frac{b_1^2A}{4I_Z})\right] +$ $+ 2h^2I_{2S} - 2b_1ch^2A_2 +$ $+ b_1^2hdA\frac{I_{2S}}{I_Z} - 4h^2\frac{I_{2S}^2}{I_Z}$

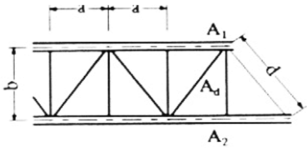
In cross-sections 8 and 9, “c” is the distance between the axis of the flange and the centre of gravity of the flaps at the ends.

18.2.5. Torsion stiffness of semi-closed sections with triangulations or frames on any of their sides

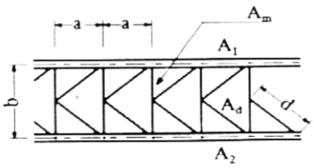
This is the case, for example, for open subsections of mixed box sections that provisionally close their torsion circuits during construction phases by means of triangulations or Vierendeel frames on any of their sides. There may also be members made only of steel that include such arrangements (columns and composite supports, for example). In order to design the uniform torsion modulus for such members, the equivalent thickness ‘t’ of a fictitious sheet panel may be considered, where the deformation energy of such a panel under uniform torsion is equal to that of the corresponding triangular panel or Vierendeel frame.

Figure 18.2.5 gives the expressions that yield the equivalent thickness ‘t’ for the most common arrangements:

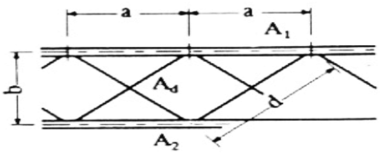
$$t = \frac{E}{G}\,\frac{ab}{\dfrac{d^3}{A_d} + \dfrac{a^3}{3}\left(\dfrac{1}{A_1} + \dfrac{1}{A_2}\right)}$$



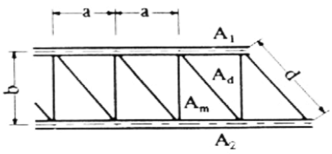
$$t = \frac{E}{G}\,\frac{ab}{\dfrac{2d^3}{A_d} + \dfrac{b^3}{4A_m} + \dfrac{a^3}{12}\left(\dfrac{1}{A_1} + \dfrac{1}{A_2}\right)}$$



$$t = \frac{E}{G}\,\frac{ab}{\dfrac{d^3}{2A_d} + \dfrac{a^3}{12}\left(\dfrac{1}{A_1} + \dfrac{1}{A_2}\right)}$$



$$t = \frac{E}{G}\,\frac{ab}{\dfrac{d^3}{A_d} + \dfrac{b^3}{A_m} + \dfrac{a^3}{12}\left(\dfrac{1}{A_1} + \dfrac{1}{A_2}\right)}$$



$$t = \frac{E}{G}\,\frac{1}{\dfrac{ab^2}{12I_m} + \dfrac{a^2b}{48}\left(\dfrac{1}{I_1} + \dfrac{1}{I_2}\right)}$$

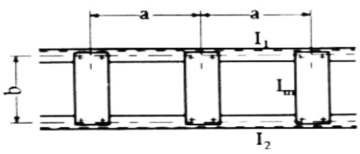


Figure 18.2.5. Equivalent thickness ‘t’ for the most common arrangements of semi-closed sections with triangulations or frames on any of their sides

where:

- A_1, A_2, I_1, I_2 area and second moment of area of each chord;
- A_d area of a diagonal;
- A_m, I_m area and second moment of area of an upright;
- a, b, d dimensions given in the adjoining diagrams;
- E and G modulus of the steel’s elasticity and transverse deformation.

18.3. Models for the stiffness of joints

Depending on their relative stiffness in relation to the members that are to be joined, subsection 57.4 classifies joints as follows: nominally pinned joints, rigid joints or fixing, and semi-rigid joints the deformability of which remains characterised by their moment-rotation curves (see subsection 57.2).

The stiffness, resistance and ductility requirements are discussed in Section 57.

In the case of semi-rigid joints, the structural model must be able of reproducing the effects of their non-linear performance on the distribution of forces throughout the structure and on the structure's global deformations, unless such effects are insignificant.

For bridges and structures subjected to dynamic loads, the check on joints must include verification of whether they respond to fatigue correctly.

Joint design is usually studied to minimise eccentricities between the barycentric axes of the connected members, in such a way as to minimise the secondary forces owing to possible stiffness of joints when they rotate.

Subsection 55.4 discusses the conditions that allow such effects to be discounted in the event of nodes in triangular structures. Section 64 also sets out specific conditions for direct joints in hollow section members. In other situations, the resistance and fatigue checking, both for the actual joints and for the connected members, must include such secondary forces, and the structural model must adequately incorporate the geometry of the eccentricities quoted.

18.4. Models for the stiffness of foundations

In some structures, the performance of which is affected significantly by the deformability conditions of the ground foundations, the analysis shall be approached using structural models that adequately incorporate the effects of interaction between the ground and the structure.

Where the structural response may be regarded as being significantly affected by possible variations in the deformation parameters of the ground in relation to their estimated mean value, the structural analysis shall include a sensitivity analysis to ensure that the structure responds correctly within the probably fluctuation range for such parameters, which shall be set out and justified in the geotechnical design report.

In order to incorporate the stiffness of connections between the foundations and the ground into the structure model, elastic or non-linear springs (for horizontal, vertical and turning displacement) or, if necessary, a model of finite elements from the adjacent area of ground may be used.

The stiffness of deep foundations must include any possible group effect of piles, as well as the stiffness of the piles and pile caps together.

Where the structural response is affected significantly by interaction with the ground, the structure’s design shall cover any uncertainties in the model so as to ensure that its global response has adequate ductility, as well as the different members affected together with their joints.

Section 19. Global analysis

19.1. Analysis methods

Any structural analysis must satisfy the equilibrium and compatibility conditions, taking into account the laws that affect the performance of the materials.

Methods for the global analysis of a structure are classified as follows:

a) linear analyses, based on the elastic-linear performance hypothesis for the materials considering the equilibrium on the structure without deformation (first-order analysis);

b) non-linear analyses, which take account of mechanical non-linearity, i.e. the non-linear performance of materials, and geometric non-linearity, i.e. considering the equilibrium conditions on the deformed structure (second-order analysis);

c) non-linear analyses in turn may study one or more of the non-linear causes mentioned.

Non-linear performance implies the invalidity of the superposition principle, which must be taken into account when applying the safety format described in Chapters II, III and IV.

In cases of non-linearity, the structural response depends on the load history, and it is usually worth proceeding gradually, covering the elastic and elastic-plastic ranges until the structure fails.

A non-linear analysis usually requires an iterative process of successive linear analyses for a given load level until they converge on a solution that satisfies the material equilibrium, compatibility and performance conditions. Such conditions are checked on a set of sections, depending on discretisation, which must be sufficient to ensure that the structural response is adequately approximated.

The corresponding fatigue limit state checks shall be conducted based on the results of a linear global analysis of the structure.

Serviceability limit state checks are also usually to be conducted by means of linear analyses. Exceptions to this are certain one-off structures that are very slender or anchored, where it may be necessary to consider the effect of deformations under service loads. Section 41 also considers the possibility of allowing limited plasticising in service situations for certain structures subjected to predominantly static loads.

Possible consideration of the effects of shear lag on the global analysis of the structure is discussed in subsections 18.2.2 and 21.2.

The effects of the instability of thin, compressed sheets may dictate the type of global analysis to be performed on the structure, in accordance with Section 20. The effects of local buckling on the stiffness of members to be considered in the global analysis of the structure are discussed in subsection 18.2.2. For slender sections of class 4, please also see subsection 19.3.

19.2. Consideration of material non-linearity

Depending on the way in which the effects of material non-linearity are considered or not, the methods for global analysis of the structure are classified as follows:

a) elastic global analysis;

b) plastic global analysis;

c) elastic-plastic global analysis.

The elastic global analysis may be used in all cases, with the precautions set out in subsection 20.6.

Conventional building structures may, in certain cases, use an elastic linear analysis with limited redistribution, in accordance with subsection 19.3.1.

The elastic-plastic global analysis described in subsection 19.5 may always be used to test the ultimate limit states.

The plastic global analysis cannot be used for bridges or structures subject to mobile or iterative significant overloads.

19.3. Elastic global analysis

The elastic global analysis is based on the assumption that the stress-strain performance of the material is linear, whatever the stress level is.

It is a linear method that allows for the principle of superposition.

Its application to serviceability limit state and fatigue limit state checking for steel structures means that the effects of the following must be considered:

– the different resistance diagrams and load application sequences in the case of progressive assembly;

– heat actions (expansion and gradient);

– actions caused by supports drops or any imposed deformation applied to the structure (pre-stressing, imposed movement of supports, etc.).

Such effects may be discounted when checking the ultimate limit states of the structure, if all the critical or potentially critical sections are of class 1 (see Section 20).

The elastic global analysis may be applied to obtain the forces in the structure, including even when the resistance checks on sections for ultimate limit states are determined by local buckling of their sheets (sections of class 4), or take their plastic reserves into account (sections of class 1 or 2), with the exceptions made in subsection 20.6.

19.3.1. Elastic global analysis with limited redistribution

This is where the forces of the combined actions to be considered are obtained from a linear elastic global analysis, in order to check the structure’s ultimate limit states, as described in subsection 19.3, and redistribution of forces subsequently made.

Its application is limited to continuous beams in conventional building structures in which adequate ductility conditions are ensured. To this end, the following conditions must be fulfilled:

a) redistribution in the elastic laws of bending moments for each span are limited to 15 % of its maximum value in the member;

b) the forces in the structure must be in equilibrium with the loads applied, once such forces have been redistributed;

c) the cross-sections of all members for which redistributions are made must be of class 1 or class 2, in accordance with Section 20;

d) the lateral stability of the beams and of their compressed flanges must be adequately controlled.

19.4. Plastic global analysis

Methods based on the plastic global analysis of steel structures may only be applied to the checking of ultimate limit states of conventional building structures, or structures subjected to predominantly static loads, and in the absence of iterative significant overloads.

Plastic methods are approached in accordance with the theory of plastic hinges, and allow for full redistribution of forces inside the structure, ensuring that the plastic resistance moment obtained by successive plastic hinges remain unchanged until the last plastic hinge is formed, which converts the structure into a mechanism.

Plastic methods may be based on any one of the basic plasticity theorems: the static or of lower limit and the kinematic or of upper limit.

These methods do not allow to consider the loading sequences and phases for developing structures, nor thermal action, imposed deformation or any self-balancing system of actions that load the structure, and it may be assumed an steady growth in the amplification factors of the actions until the collapse mechanism is achieved for the different combinations of actions considered. The principle of superposition cannot be applied.

A plastic global analysis is only permitted where the different members of the structure have sufficient ductility to ensure redistribution of the forces required by the plastic collapse mechanisms that are considered. Verifying that the conditions set out in subsection 20.5 have been met ensures this.

In the case of supports or lintels subjected to compression forces, any estimate of their rotary capacity must consider the influence of the compression axil on reducing the ductility of the moment-curvature (M-χ) laws governing the cross-sections.

Plastic analyses must not usually be used where the second-order effects due to deformations cannot be disregarded, since in such cases the collapse of the structure may be achieved before all the plastic hinges of the first-order plastic failure mechanism are developed. In such cases, the general, non-linear analysis method described in subsection 24.4 must be used.

19.5. General, elastic-plastic, non-linear analysis method

The elastic-plastic method considers the influence of the non-linear response of steel on the moment-curvature diagrams for different cross-sections, which are usually obtained under steadily increasing loads until ultimate resistance is achieved. Moment-curvature diagrams must include consideration of possible axial force that might act at the same time.

The cross-sections remain elastic until they achieve deformation corresponding to the yield strength in the most stressed fibre. Under increasing loads, the section undergoes gradual plastification until the maximum strains are achieved for compression or traction in the most stressed fibre.

The maximum strain for steel is given in subsections 19.5.1 to 19.5.3, including the consideration of possible instability phenomenon in compressed sheets.

The properties of the reduced design section, for considering the effects of sheet instability on slender cross-sections of class 4, are obtained depending on the maximum strain of the compressed members, increasing steadily, in accordance with subsection 20.7.

The effects of shear lag under increasing loads are taken into account based on the effective widths given in subsections 21.3 and 21.4 for the elastic phase, and in subsection 21.5 for the elastic-plastic phase.

The elastic-plastic global analysis is approached for the combinations of actions to be considered for the ultimate limit states by means of non-linear design algorithms based on the moment-curvature (M-χ) laws governing the different cross-sections. The principle of superposition cannot be applied.

19.5.1. Sections without longitudinal stiffeners

The following limit strains shall be adopted:

a) compressed steel members:

$\varepsilon_{cu} = 6\ \varepsilon_y$ for sections of class 1;

$\varepsilon_{cu} = 3\ \varepsilon_y$ for sections of class 2;

$\varepsilon_{cu} = \varepsilon_y$ for sections of classes 3 and 4;

b) tensioned steel members:

εtu = 2 % for sections of classes 1 and 2;

εtu = 6 εy for sections of classes 3 and 4,

where ε_y is the strain corresponding to the reduced yield strength of the steel.

In slender cross-sections of class 4, the effective widths of the sections are obtained using the criteria set out in subsection 20.7 and in Tables 20.7.a and 20.7.b, based on the deformity plane under consideration. In order to calculate the reduction factor ρ for compressed panels, the maximum compression strain of the panel shall be adopted to evaluate $\bar{\lambda}_p$ for this deformity plane. This shall hold for both the compressed flange and the fully or partially compressed web:

$$\bar{\lambda}_p = \sqrt{\frac{\varepsilon_{cmax}}{\varepsilon_{cr}}}$$

The effective width for the shear lag of a panel may be estimated, in accordance with subsection 21.5, using a linear interpolation of the reduction factors ψ for intermediate curvatures χ, between the elastic χ_{el} and the ultimate elastic-plastic χ_u.

19.5.2. Sections with longitudinal web stiffeners

The same limit strains for tension and compression are adopted as for subsection 19.5.1.

In order to obtain the effective widths for slender webs, it shall be considered that every stiffener divides the web sheet into independent sub-panels. A similar criterion is applied to each sub-panel to the one set out in subsection 19.5.1, and the value ε_{cmax} is considered as the maximum strain on the most highly compressed depth of the panel (see Figure 19.5.2).

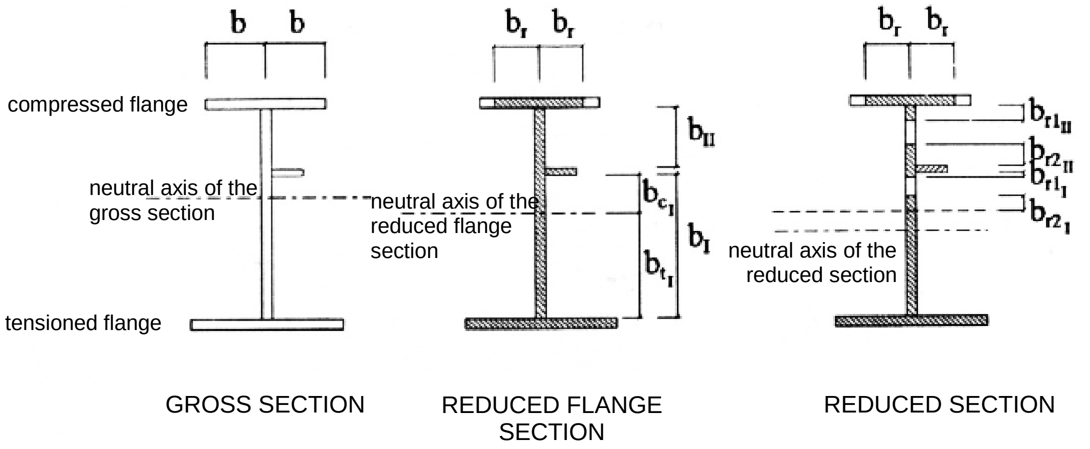


Figure 19.5.2. Effective sections with stiffeners

19.5.3. Sections with longitudinal stiffeners for compressed flanges

The elastic-plastic analysis of cross-sections with compressed, stiffened flanges essentially depends on the latter, the response of which may be assimilated into a series of stiffeners with the associated effective width of compressed flange at each side of their axis. These stiffeners perform as compressed supports resting elastically on transverse stiffeners (or anti-distortion diaphragms of the box girder sections).

Annex 6 discusses possible approximate models for the non-linear elastic-plastic response of compressed, stiffened sheet panels, which may be used to estimate the moment-curvature laws that govern the full cross-section.

19.6. Effects of deformed geometry of the structure

The global analysis of the structure may usually be performed by means of:

a) a first-order analysis, using the initial geometry of the structure;

b) a non-linear second-order analysis, taking into account the effects of the deformed geometry of the structure.

The second-order effects due to deformation of the structure, must be considered if they increase the action effects (forces and deformation) significantly in the structural response.

Geometric and mechanical imperfections must be considered in order to evaluate this, in accordance with Section 22. The principle of superposition cannot apply as it is a non-linear analysis.

The influence of second-order effects on the reduction of resistance capacity of certain individual members, such as wholly or partially compressed supports or constant section beams, is taken into account under this Code by means of reduction factors that are included in their resistance formulae, such as those stated in subsections 35.1, 35.2 and 35.3.

Section 23 describes the methods to evaluate whether the second-order effects have a significant impact on the global response of the structure.

Section 24 discusses the general analysis methodology that allows such effects to be taken into account if need be.

Section 20. Classification of cross-sections

20.1. Bases

Grouping cross-sections into four classes allows to identify the influence of local instability phenomenon of sheets (local buckling) in the compressed areas on:

– resistance, identifying the section capacity for achieving, or not, their elastic or plastic resistance moments (see Figure 20.1.a);

– rotation capacity, identifying their ability to develop, or not, the ultimate curvatures required for a global analysis of forces using elastic or plastic methods (see Figure 20.1.b).

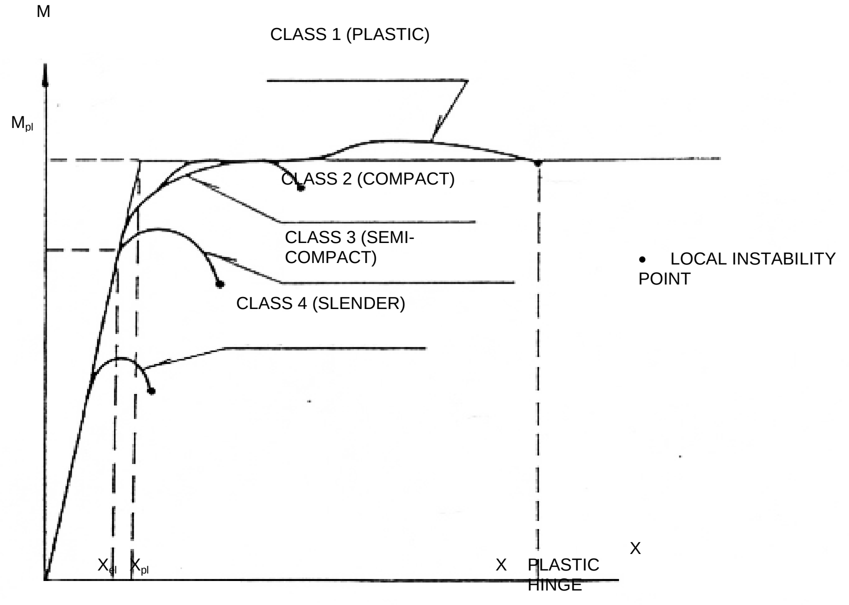


Figure 20.1.a. Moment-curvature (M-χ) laws for cross-sections of classes 1 to 4

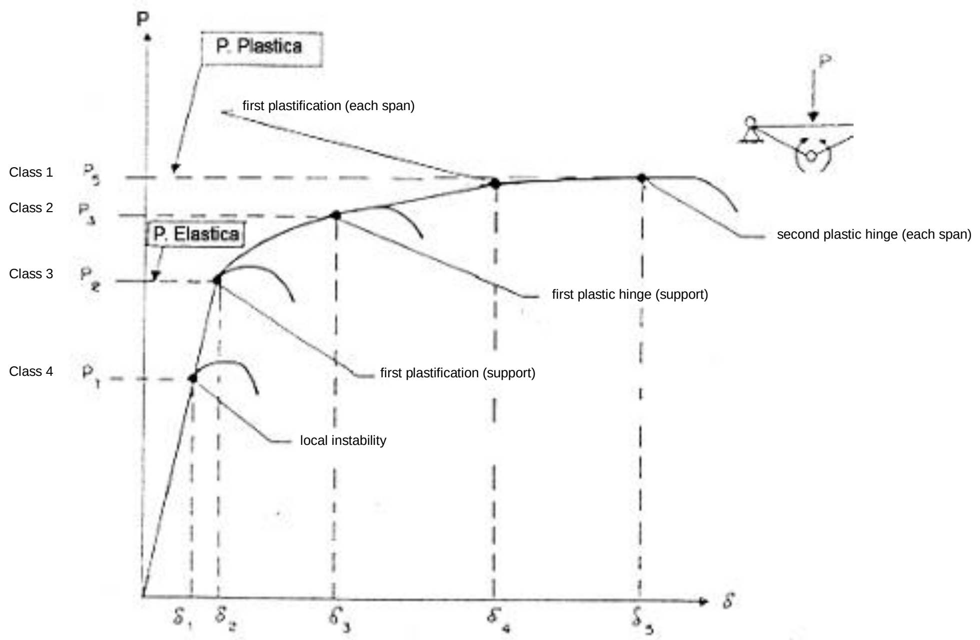
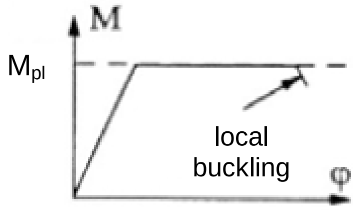
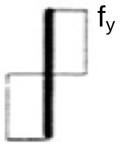
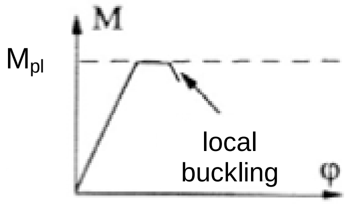
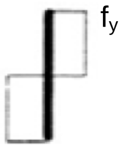
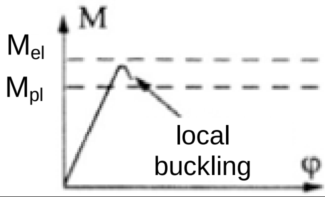
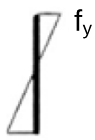
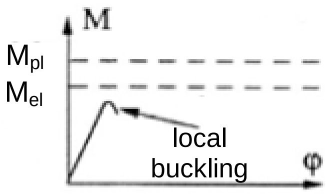
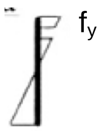


Figure 20.1.b. Elastic-plastic diagram up to fracture of a continuous lintel, depending on the class of the cross-sections.

The assignment of a cross-section class only applies in relation to sheet instability phenomenon under the action of direct stresses. The problems associated with local buckling on sheets subjected to shear stresses are discussed in subsection 35.5 and Section 40.

Table 20.1. Classification of cross-sections in relation to the checking ultimate limit states

Class	Performance model	Design resistance	Rotation capacity of the plastic hinge	Global analysis of the structure
1		PLASTIC on the whole section 	significant	elastic or plastic
2		PLASTIC on the whole section 	limited	elastic
3		ELASTIC on the whole section 	none	elastic
4		ELASTIC on the effective section 	none	elastic

20.2. Classification of cross-sections

Depending on the influence of sheet instability problems on its resistance response, four classes of cross-section are defined (see Figures 20.1.a and 20.1.b):

– sections of class 1 (plastic) develop their plastic resistance capacity without being affected by local buckling in their compressed areas, and can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance;

– sections of class 2 (compact) may develop their plastic moment resistance, but the local buckling limit their rotation capacity to a level below that required for the plastic global analysis to be applied;

- sections of class 3 (semi-compact) are those in which the stress in the extreme compression fibre, assuming an elastic distribution of stresses, can reach the yield strength, but local buckling is liable to prevent the development of the plastic moment resistance;

- sections of class 4 (slender) are those in which local buckling also inhibit the development of their elastic resistance capacity, and the yield strength cannot be reached in the extreme compression fibre.

The classification of a cross-section depends on the following:

a) the yield strength of the steel in the section;

b) the geometry of the section and, in particular, the slenderness (dimension/thickness relationship) of its wholly or partially compressed sheets;

c) any lateral connections in the compressed areas;

d) sign of bending moment in the case of asymmetrical sections, in relation to their neutral fibre;

e) the bending/axial forces relationship in sections subjected to combined bending or compression, which determines the position of the neutral fibre and thus the geometry and extension of the compressed areas of the sheet;

f) the direction of the bending moment axis in cases of biaxial bending, which determines the orientation of the neutral fibre and thus the geometry and extension of the compressed areas of the sheet.

The different compressed sheets in a cross-section, for example the flanges or webs, may be assigned to different classes, depending on the slenderness and extension of their compressed areas.

The highest class is usually assigned to a cross-section, i.e. the least favourable one, from among those relating to each of the section's compressed parts. Alternatively, the classification of a cross-section may distinguish between the assignation of its web class and that of its compressed flanges, for the purposes set out explicitly in certain Sections of this Code.

For slender sections of class 4, the reduction in their resistance capacity in ultimate limit states, as a result of local buckling, may be estimated using ideal effective sections in accordance with subsection 20.7.

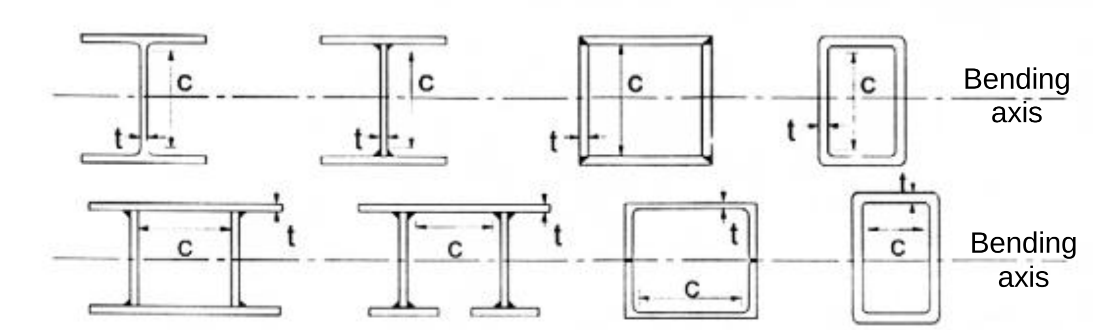
20.3. Criteria for assigning classes to unstiffened sections

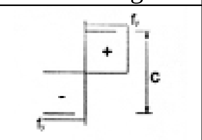
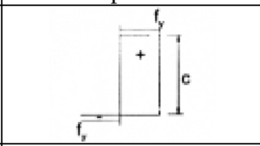
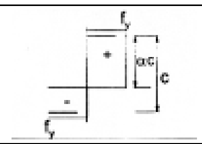
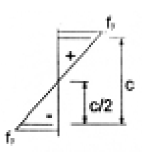
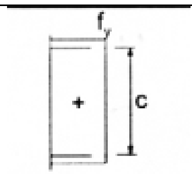
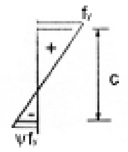
For cross-sections that do not have longitudinal stiffeners, the different parts, whether wholly or partially compressed, may be classified on the basis of the slenderness limit ratios contained in Tables 20.3.a to 20.3.c.

In general, any compressed part that does not satisfy the limits for class 3 set out in those Tables must be assigned class 4.

The plastic distribution of stresses shall initially be used to classify cross-sections, except on the threshold between classes 3 and 4, which shall be established on the basis of the elastic analysis (or the elastic-plastic analysis with plastification in the tensioned area, as discussed below).

Table 20.3.a. Maximum slenderness for internal compression parts (flanges and webs)



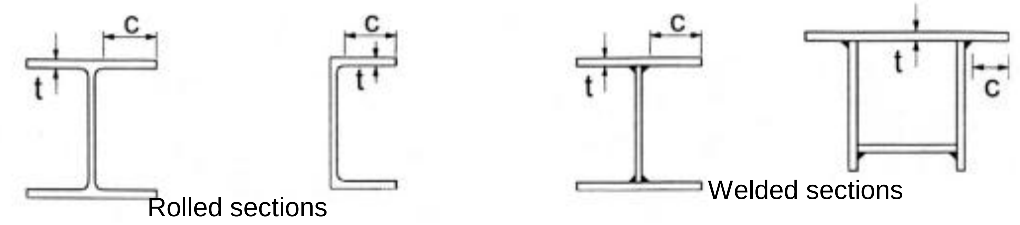
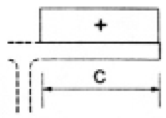
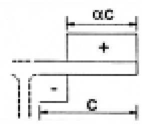
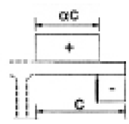
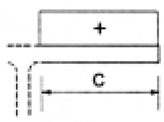
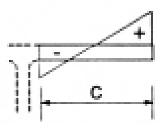
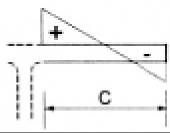
Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression+)			
1	c/t≤72ε	c/t≤33ε	where α>0.5: c/t≤ $\frac{396\varepsilon}{13\alpha - 1}$ where α≤0.5: c/t≤ $\frac{36\varepsilon}{\alpha}$
2	c/t≤83ε	c/t≤38ε	where α>0.5: c/t≤ $\frac{456\varepsilon}{13\alpha - 1}$ where α≤0.5: c/t≤ $\frac{41.5\varepsilon}{\alpha}$
Stress distribution in parts (compression+)			
3	c/t≤124ε	c/t≤42ε	where ψ>-1: c/t≤ $\frac{42\varepsilon}{0.72 + 0.33\psi}$ where ψ≤-1 *): c/t≤62 ε(1- ψ) $\sqrt{(-\psi)}$

$\varepsilon = \sqrt{235/f_y}$	$f_y (N/mm^2)$	235	275	355	420	460
	ε	1.00	0.92	0.81	0.75	0.71

*) ψ ≤ -1 applies where the deformation in the compressed fibre is less than the deformation in the tensioned fibre, which may be partially plastified. In such case, ψ is the algebraic ratio between the plastic deformation in the tensioned fibre (> f_y/E) and the elastic deformation in the compressed fibre (< f_y/E).

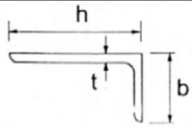
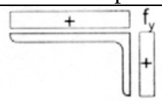
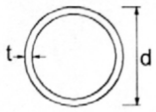
Table 20.3.b

Maximum slenderness for compression parts in outstand flanges

<table>
<tr><td colspan="7">
</td></tr>
<tr><td rowspan="2">Class</td><td rowspan="2" colspan="2">Part subject to compression</td><td colspan="4">Part subject to bending-compression</td></tr>
<tr><td colspan="2">Tip in compression</td><td colspan="2">Tip in tension</td></tr>
<tr><td>Stress distribution in parts (compression+)</td><td colspan="2">
</td><td colspan="2">
</td><td colspan="2">
</td></tr>
<tr><td>1</td><td colspan="2">$c/t \le 9\varepsilon$</td><td colspan="2">$c/t \le \frac{9\varepsilon}{\alpha}$</td><td colspan="2">$c/t \le \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$</td></tr>
<tr><td>2</td><td colspan="2">$c/t \le 10\varepsilon$</td><td colspan="2">$c/t \le \frac{10\varepsilon}{\alpha}$</td><td colspan="2">$c/t \le \frac{10\varepsilon}{\alpha - \sqrt{\alpha}}$</td></tr>
<tr><td>Stress distribution in parts (compression+)</td><td colspan="2">
</td><td colspan="2">
</td><td colspan="2">
</td></tr>
<tr><td>3</td><td colspan="3">$c/t \le 14\varepsilon$</td><td colspan="3">$c/t \le 21\varepsilon\sqrt{k_Q}$</td></tr>
<tr><td rowspan="2">$\varepsilon = \sqrt{235/f_y}$</td><td>$f_y (N/mm^2)$</td><td>235</td><td>275</td><td>355</td><td>420</td><td>460</td></tr>
<tr><td>ε</td><td>1.00</td><td>0.92</td><td>0.81</td><td>0.75</td><td>0.71</td></tr>
</table>

The value of the local buckling factor k_σ may be obtained from Tables 20.7.a and 20.7.b.

Table 20.3.c. Maximum slenderness for compression parts in special cases

<table>
<tr><td colspan="7">Angles</td></tr>
<tr><td colspan="2">See also “outstand flanges” (Table 20.3.b)</td><td colspan="3">
</td><td colspan="2">This does not apply to angles in continuous contact with other components</td></tr>
<tr><td>Class</td><td colspan="6">Section in compression</td></tr>
<tr><td>Stress distribution in parts (compression+)</td><td colspan="6">
</td></tr>
<tr><td>3</td><td colspan="6">$h/t \le 15\varepsilon \; : \; \frac{b+h}{2t} \le 11.5\varepsilon$</td></tr>
<tr><td colspan="7">Tubular sections</td></tr>
<tr><td colspan="7">
</td></tr>
<tr><td></td><td colspan="6">Section in bending and/or compression</td></tr>
<tr><td>1</td><td colspan="6">$d/t \le 50\varepsilon^2$</td></tr>
<tr><td>2</td><td colspan="6">$d/t \le 70\varepsilon^2$</td></tr>
<tr><td>3</td><td colspan="6">$d/t \le 90\varepsilon^2$</td></tr>
<tr><td rowspan="3">$\varepsilon = \sqrt{235/f_y}$</td><td>$f_y (N/mm^2)$</td><td>235</td><td>275</td><td>355</td><td>420</td><td>460</td></tr>
<tr><td>ε</td><td>1.00</td><td>0.92</td><td>0.81</td><td>0.75</td><td>0.71</td></tr>
<tr><td>ε^2</td><td>1.00</td><td>0.85</td><td>0.66</td><td>0.56</td><td>0.51</td></tr>
</table>

The following situations may also be considered:

a) compression parts shall be assigned to class 1 if their local buckling may effectively be prevented through connectors or other fixing components to a concrete slab or other stiff system.

In such cases, the maximum distance between connector axes in the direction of compression shall not exceed:

$22\ t_f,\ \sqrt{235/f_y}$, if the slab is in continuous contact with the part;

$15\ t_f\ \sqrt{235/f_y}$, if it is not.

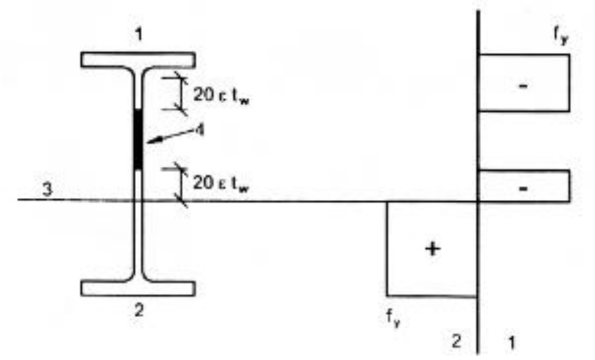
Furthermore, the maximum distance from the extreme of the part to the nearest connector line shall be less than:

$9\ t_f\ \sqrt{235/f_y}$, where t_f is the thickness of the compressed part;

b) With the exception of bridges or components of particular relevance, cross-sections with class 1 or 2 flanges and class 3 webs may be classified as effective class 2 cross-section, replacing the compressed area of the web with two sub-parts of dimension:

$20\,\varepsilon\, t_w = 20\, t_w \sqrt{235/f_y}$, where tw is the thickness of the web.

Both sub-parts shall be located adjacent to the compressed flange and the plastic neutral fibre of the new ideal effective cross-section (see Figure 20.3.a);

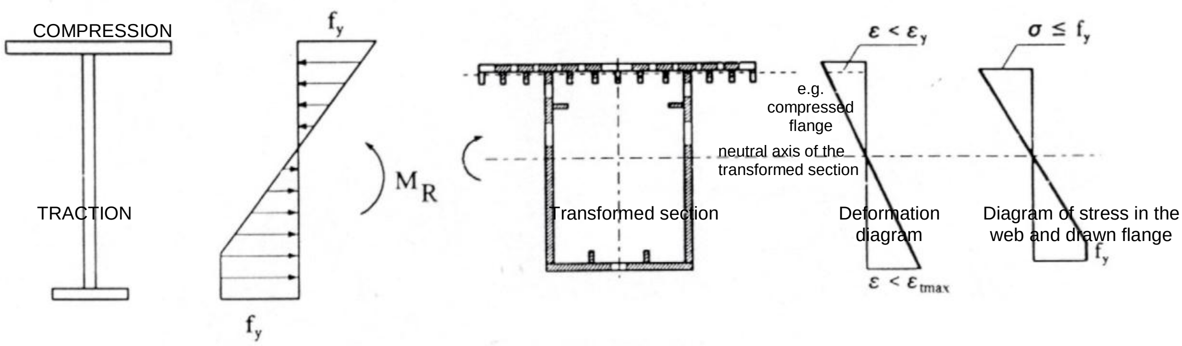


1 compression
2 traction
3 neutral plastic fibre
4 not considered

Figure 20.3.a. Class 3 web equivalent to a class 2 web where the flanges are of class 1 or 2

c) For cross-sections of class 3 or 4 that are asymmetrical in relation to the neutral bending fibre, and where plastification first occurs in the tensioned area of the cross-section, it may be permitted, in order to assign both the web class (see Table 20.3.a) and the ultimate resistance moment of the section, the plastic analysis of the tensioned fibres of the section (see Figure 20.3.b).

The maximum tensile deformation shall be limited to 6 ε_y, where ε_y is the yield strength of the steel. Continuous elements must also satisfy the ductility requirements set out in subsection 20.5;



Class 3 Class 4

Figure 20.3.b. Elastic-plastic response to tension in class 3 or 4 webs

d) Except in the case of ultimate limit state checking on components that are susceptible to instability problems, which are discussed in Section 35, cross-sections of class 4 may be regarded as sections of class 3 where they are required for design stresses that are lower than the effective design yield strength of the steel, and the slenderness of the wholly or partially compressed parts is lower than the limit values given in Tables 20.3.a to 20.3.c, but adopting a corrected value of ε' such that:

$$\varepsilon' = \varepsilon\sqrt{\frac{f_y/\gamma_{M0}}{\sigma_{c,Ed}}} \geq \varepsilon \text{ , where:}$$

$\varepsilon = \sqrt{235/f_y}$

f_y yield strength of the steel, in N/mm^2;

$\sigma_{c,Ed}$, maximum design compression stress acting on the part to be classified, obtained on the basis of a first-order global analysis or, where relevant, a second-order global analysis, for the design hypothesis being considered;

e) tubular sections of class 4, the analysis of which must be approached using laminate theory, do not fall within the scope of this Code.

20.4. Criteria for assigning classes to sections with longitudinal stiffeners

Compressed parts that have longitudinal stiffeners shall be classified as class 4.

Alternatively, the section may be classified in accordance with subsection 20.3 without taking into account the presence of such stiffeners.

20.5. Cross-sections requirements for plastic global analysis

Using a plastic global analysis means that adequate rotation capacity must be ensured at the plastic hinge location.

In general, rotation requirements may differ depending on the location of the plastic hinge and the load hypothesis being considered.

The rotation requirements for the plastic design of a structure may be assumed if the conditions set out below are met for all members where plastic hinges form, or are likely to form, under the different design hypotheses that are to be considered.

In a uniform member, the following two requirements are satisfied:

– cross-sections at the plastic hinge location shall be of class 1;

– in hinges located on supports or under the action of local transverse forces that exceeds 10 % of the plastic shear resistance of the cross-section, transverse web stiffeners shall be provided within a distance from the hinge location not exceeding half of the height of the cross-section.

Where the cross-section of the member vary along their length, the following additional criteria should be satisfied:

- the web thickness should not be effective for a distance each way along the member from the plastic hinge location of at least double the clear depth of the web;

- the compression flange should be class 1 for a distance each way along the member from the plastic hinge location of not less than double the clear depth of the web at the hinge section, and wherever the bending moment in the section is greater than 80 % of the hinge's plastic resistance moment;

- elsewhere in the member, the compression flange should be class 1 or 2, and the web should be class 1, 2 or 3.

The geometry and connections of steel members must also ensure resistance to lateral buckling. They shall also ensure lateral restraint on compression flanges in the plastic hinge sections.

If the plastic hinge is located in a jointed section, the joint must have adequate ductility to ensure the required hinge rotation or, alternatively, be designed with sufficient resistance to ensure that the plastic hinge develops in the member outside the joint. The resistance and ductility requirements for joints are discussed in Section 57.

20.6. Cross-section requirements for elastic global analysis

An elastic global analysis usually always applies, irrespective of the class of the cross-sections in the members of the structure, without any other restrictions than the subsequent resistance checks, according to their class.

20.7. Properties of the effective section in slender cross-sections

In general, the properties of the effective section in cross-sections of class 4 (slender) are obtained by defining certain effective widths in the compressed areas of the parts, in accordance with the criteria set out in Table 20.7.a for internal compression web parts and in Table 20.7.b for flange parts with a free edge.

The reduction factor ρ of the compression part width may be estimated according to the following expressions:

$$\rho = \frac{\bar{\lambda}_p - 0.055(3+\psi)}{\bar{\lambda}_p^2} \leq 1.0 \text{,} \quad \text{for internal compression part;}$$

$$\rho = \frac{\bar{\lambda}_p - 0.188}{\bar{\lambda}_p^2} \leq 1.0 \text{,} \quad \text{for parts with a free edge,}$$

where:

$$\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \sqrt{\frac{\varepsilon_y}{\varepsilon_{cr}}} = \frac{\bar{b}/t}{28.4\varepsilon\sqrt{k_\sigma}} \text{, and:}$$

ψ ratio between deformations at the extremities of the parts, in accordance with Tables 20.7.a and 20.7.b;

σ_{cr} ideal critical stress for local buckling:

$\sigma_{cr} = E\varepsilon_{cr} = k_\sigma \sigma_E$, with

$$\sigma_E = \frac{\pi^2 E t^2}{12(1-\upsilon^2)\bar{b}^2} = 190000\left(\frac{t}{\bar{b}^2}\right)^2, \text{ in N/mm}^2$$

ν Poisson's ratio of the steel;

t part thickness;

ε_{cr} ideal critical deformation for panel local buckling: $\varepsilon_{cr} = 0.9k_\sigma\left(\frac{t}{\bar{b}}\right)^2$

k_σ part local buckling factor, obtained from Tables 20.7.a and 20.7.b;

$\bar{b}$ part width, obtained in accordance with the figures in Tables 20.3.a, 20.3.b and 20.3.c.

In slender sections with stiffened webs or flanges (Figure 20.7.a), the stiffened part may be treated as a set of sub- part of width b, delimited by the longitudinal stiffeners, thus obtaining the effective width of each sub- part in accordance with the criteria set out above, and depending on the ratio between the deformations on its edges.

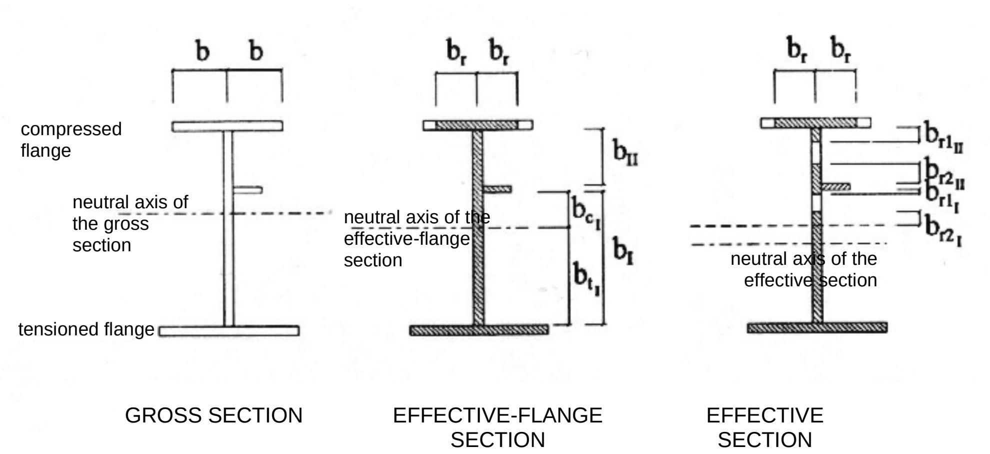
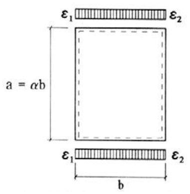
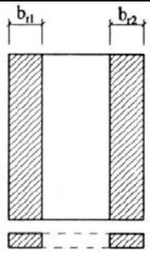
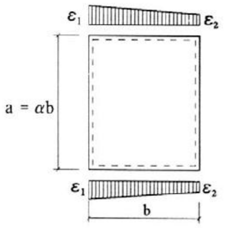
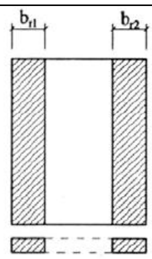
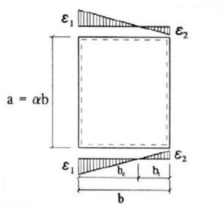
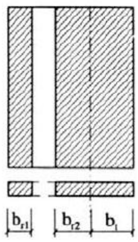


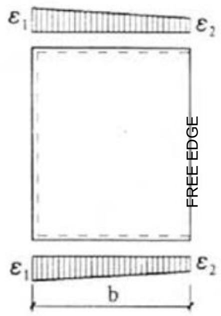
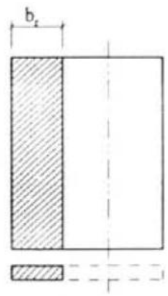
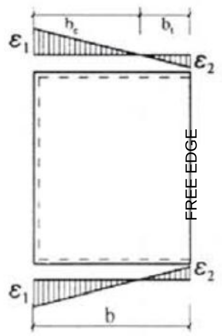
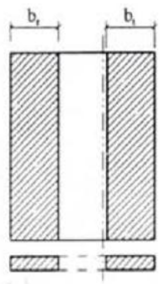
Figure 20.7.a. Effective section in stiffened slender sections

Table 20.7.a. Reduction factor ρ (internal parts)

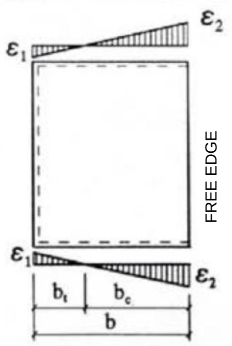
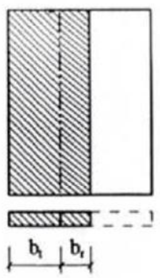
<table>
<tr><th colspan="9">INTERNAL PARTS FOR FLANGES AND WEBS</th></tr>
<tr><th colspan="4">STRAINS</th><th colspan="5">EFFECTIVE WIDTH b_r</th></tr>
<tr><th colspan="9">$\varepsilon_1 > 0$ (compression)</th></tr>
<tr><td colspan="4">
</td><td>
</td><td colspan="4">$b_r = \rho b$
$b_{r1} = 0.5 b_r$
$b_{r2} = 0.5 b_r$
$\psi = 1$</td></tr>
<tr><td colspan="4">
</td><td>
</td><td colspan="4">$b_r = \rho b$
$b_{r1} = \frac{2b_r}{5-\psi}$
$b_{r2} = b_r - b_{r1}$
$o < \psi < 1$</td></tr>
<tr><td colspan="4">
</td><td>
</td><td colspan="4">$b_r = \rho b$
$b_{r1} = 0.4 b_r$
$b_{r2} = 0.6 b_r$
$\psi < 0$</td></tr>
<tr><td colspan="2">$\psi = \frac{\varepsilon_2}{\varepsilon_1}$</td><td>1</td><td>$1 > \psi > 0$</td><td>0</td><td>$0 > \psi > -1$</td><td>$-1$</td><td colspan="2">$-1 > \psi > -5$</td></tr>
<tr><td rowspan="2">k_σ</td><td>$\alpha \geq 1$</td><td>4.0</td><td>$\frac{8.2}{1.05+\psi}$</td><td>7.81</td><td rowspan="2">$7.81 - 6.29\psi + 9.78\psi^2$</td><td rowspan="2">23.9</td><td colspan="2" rowspan="2">$5.98(1-\psi)^2$</td></tr>
<tr><td>$\alpha < 1$</td><td colspan="3">$\left(\alpha + \frac{1}{\alpha}\right)^2 \frac{2.05}{1.05+\psi}$</td></tr>
</table>

k_σ = local buckling factor

Table 20.7.b. Reduction factor ρ (parts with a free edge)

INTERNAL PARTS FOR FLANGES AND WEBS		
STRAINS	**EFFECTIVE WIDTH b_r**	
$\varepsilon_1 > 0$ (compression)		
		$b_r = \rho b$ $1 > \psi \geq 0$
		$b_r = \rho b_c = \rho \frac{b}{1-\psi}$ $\psi < 0$

$\psi = \frac{\varepsilon_2}{\varepsilon_1}$	$\psi > 1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1
k_σ	$0.57 - \frac{0.21}{\psi} + \frac{0.07}{\psi^2}$	0.43	$\frac{0.578}{\psi + 0.34}$	1.70	$1.7 - 5\psi + 17.1\psi^2$	23.8

$\varepsilon_1 \leq 0$ (TENSION)		
		$b_r = \rho b_c$
$\psi = \frac{\varepsilon_1}{\varepsilon_2}$	for	$-3 \leq \psi \leq 0 \Rightarrow k_\sigma = 0.57 - 0.21\psi + 0.07\psi^2$

k_σ = local buckling factor

The effective widths for compression flanges may usually be obtained from the geometry of the gross cross-section. However, in order to obtain the effective widths of the webs, the deformation plane ψ must be used. This is obtained from the effective area of the compression flange parts. It is not usually necessary to repeat the process, and the gross dimensions of the web may be used to calculate ψ (see Figure 20.7.b).

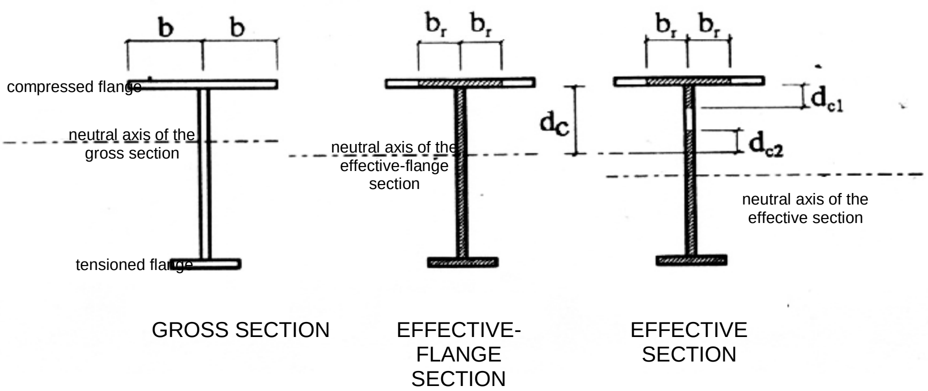
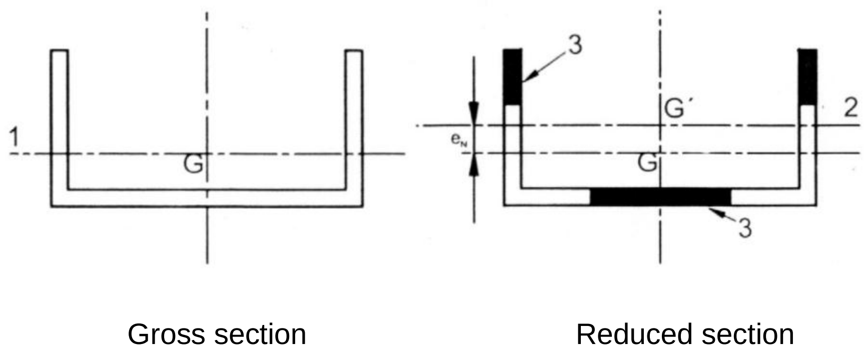


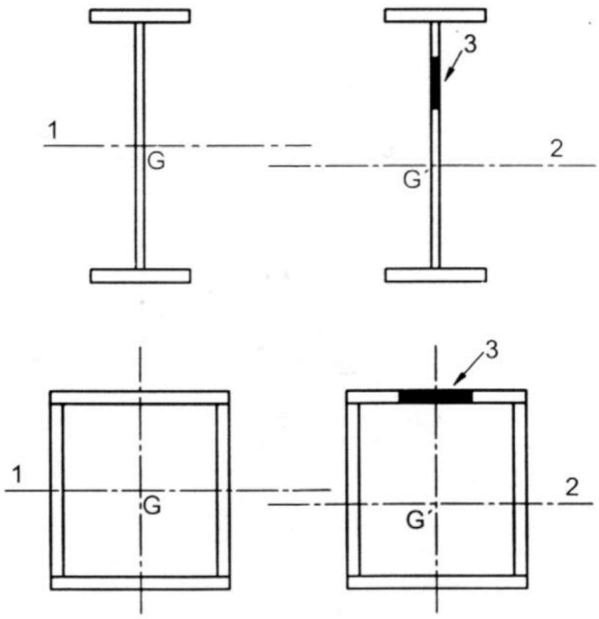
Figure 20.7.b. Definition of the effective section

The neutral fibre of the effective section will usually experience a displacement of e_N value in relation to the neutral fibre of the gross section (see Figures 20.7.c and d). Such displacement must be taken into account in order to obtain the static parameters (I_{ef} and W_{ef}) of the effective section.



G	*centre of gravity of the gross section*
G'	*centre of gravity of the reduced section*
1	*neutral fibre of the gross section*
2	*neutral fibre of the reduced section*
3	*ineffective areas*

Figure 20.7.c. Reduced section under axial force



Gross section Effective section

G *centre of gravity of the gross section*
G' *centre of gravity of the effective section*
1 *neutral fibre of the gross section*
2 *neutral fibre of the effective section*
3 *ineffective areas*

Figure 20.7.d. Effective section under bending

When cross-sections of class 4 are stressed by an axial force acting on the centre of gravity of the gross section, the effect of displacement of the neutral fibre between the effective section and the gross section must be considered, in order to obtain the increase in bending forces on the neutral fibre of the effective section. In order to avoid iterative processes, such additional moment may be approximately estimated using the displacement e_N of the neutral fibre of the effective section, assuming that it is subjected only to a centred compression (Figure 20.7.c):

$$\Delta M = N e_N$$

Apart from checks on the ultimate limit states of steel members that are liable to instability problems, which are discussed in subsections 35.1, 35.2 and 35.3, the effective widths of the compressed parts of cross-sections in class 4 may be estimated more precisely using a value of $\bar{\lambda}_p$, calculated form the values of the maximum stress or deformation in the compressed part. These values are obtained using the effective widths of all fully or partially compressed parts in the section:

$$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{\frac{\sigma_{c,Ed}}{f_y/\gamma_{M0}}} = \bar{\lambda}_p \sqrt{\frac{\varepsilon_{c,Ed}}{\varepsilon_y/\gamma_{M0}}} \leq \bar{\lambda}_p$$

where:

$\sigma_{c,Ed}$ maximum design compression stress that loads this part, obtained from the static parameters of the effective section for the load hypothesis being considered;

$\varepsilon_{c,Ed} = \sigma_{c,Ed}/E$ maximum design compression deformation, estimated in the same way.

This procedure requires an iterative calculation in which both the values for $\sigma_{c,Ed}$ and $\varepsilon_{c,Ed}$ and the ratio between the deformations at the extremities of the part ψ, and the effective widths for the various parts and sub-parts, are obtained at each stage from the stress and deformations laws of the total effective section calculated in the previous iteration.

Section 21. Consideration of the effects of shear lag

21.1. Bases

The content of this Section does not apply to rolled sections or trussed sections with reduced-dimension flanges.

The diffusion of the shear force from the edges of contact between webs and flanges, compressed or tensioned ,in linear members with open or closed sections, leads to non-linear distribution of the direct stresses in the flange panels of such sections (see subsection 21.3.5).

It may be assumed for practical purposes, when checking sections and estimating the bending stiffness incorporated into the global models for the structural analysis, that direct stresses are distributed uniformly at a certain effective width of the flange, which is known as the effective width.

The effective width depends on the member type (isostatic or continuous), the action type (local or distributed), the length of the member between points of zero bending moment, the presence of stiffeners in the flanges, the span of flanges with free edges, and finally the number of webs in the section and the distance between those webs.

The effective width varies throughout the member's directrix. It may also vary depending on the materials' plastification state or possible local buckling in the compressed flange parts, which is different in serviceability and failure situations.

The effects of shear lag may be neglected when:

$$b_0 \leq L / n$$

where:

b_0 flange outstand (b_1), for external semi-flanges, or half the width of an internal element (b), for internal semi-flanges (see Figure 21.3.a);

L span of isostatic members or approximate length between adjacent points of zero bending moment in continuous members (see subsection 21.3.1);

n= 20 for conventional building members;

50 for bridges or one-off building members with slender sections or where the stress or deformation control requires great precision;

20 for ultimate limit state checks on sections of class 1 or 2 (see Section 20), in all cases.

21.2. Effective width depending on analysis type

The effects of shear lag need only be considered for the global analysis of the structure when their impact is significant, for example:

– where there are significant reductions in the effective width of flanges;

– where the Designer feels that great precision is needed for the stress or deformation checks;

– in lattice, arched or stay bridges;

– in deformation checks on assemblies of cantilevered parts with significant lengths between webs.

It is not necessary to consider the effects of shear lag in the global analysis for structures composed of double T sections or beams, mainly in building.

In all cases, and unless greater precision is required, a uniform effective width may be assumed over the length of the span for the structural analysis, using the effective width corresponding to the centre of span section, as defined in subsection 21.3.

Where the limits in subsection 21.1 are exceeded, the effects of shear lag should be considered using the effective widths defined in subsections 21.3 and 21.4 at serviceability and fatigue limit state verifications, and those defined in subsection 21.5 for ultimate limit state verification.

The elastic distribution of direct stresses due to the diffusion of patch loading in the web applied at the flange level may be estimated in accordance with subsection 21.6.

21.3. Effective widths of unstiffened flanges for serviceability and fatigue limit states

The effects of shear lag on the elastic phase may be estimated using an effective flange width obtained by (see Figure 21.3.a):

$b_e = \psi_{el}\, b$, for internal flanges;

$b_{1e} = \psi_{el}\, b1$, for outstand flanges,

where ψ_{el} (≤ 1) are the reduction factors specified below.

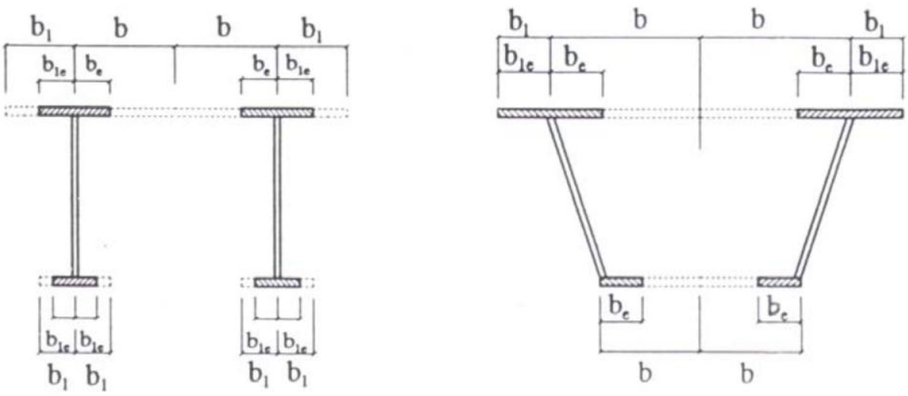


Figure 21.3.a. Effective widths for open and closed sections

The distribution of effective widths throughout a continuous beam may be assumed in accordance with the diagram in Figure 21.3.b.

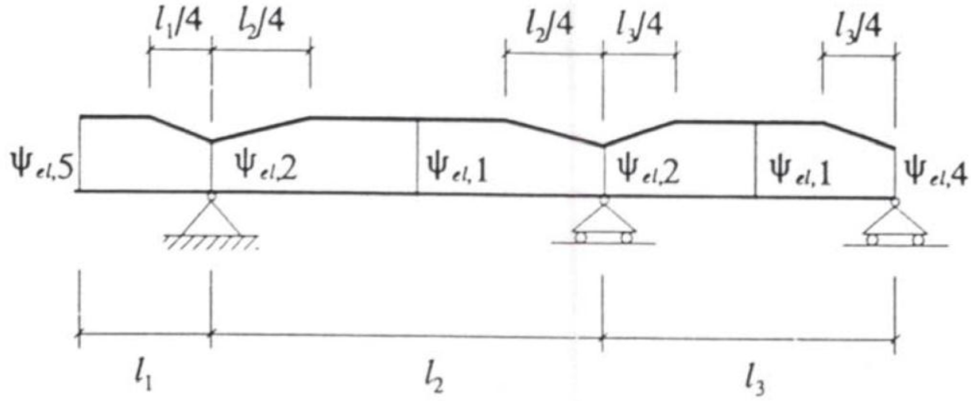


Figure 21.3.b. Distribution of effective widths for continuous beams

21.3.1. Effective lengths

In order to estimate ψ_{el}, an effective length L may be used, where L is the distance between points of zero bending moment. To put it simply, the approximate effective lengths given in Figure 21.3.1 may be used for continuous beams provided adjacent spans do not differ more than 50% and any cantilever span is not larger than half the adjacent span.

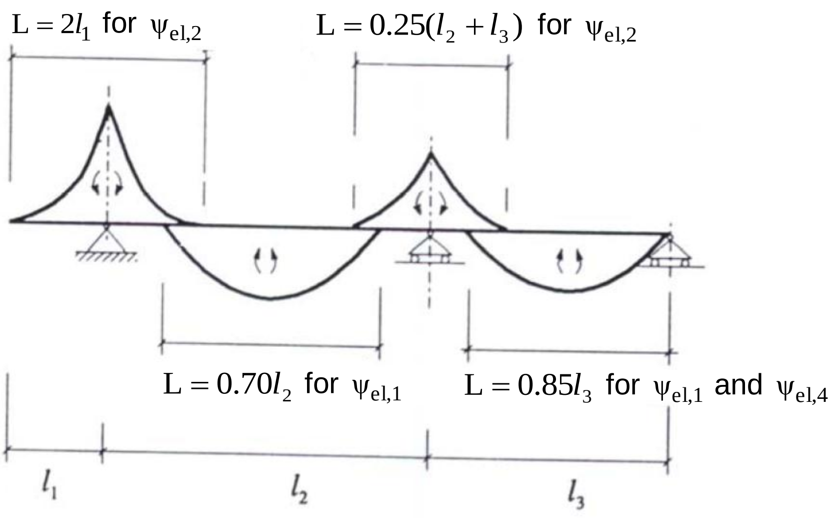


Figure 21.3.1. Effective lengths for continuous beams

21.3.2. Elastic ψ_{el} factors. Uniformly-distributed loads in continuous beams with compensated spans

The elastic reduction factors for effective flange width, ψ_{el}, use the following values, depending on the parameter $\beta = b_0/L$ (where b_0 is defined in subsection 21.1):

– in all cases:

$\psi_{el,i} = 1$ $\quad\quad$ $\beta \leq 0.02$

– in the centre of isostatic or continuous spans (positive bending):

$\psi_{el,1} = 1$ $\quad\quad$ $\beta \leq 0.05$

$$\psi_{el,1} = \frac{1}{1 + 6.4\beta^2} \qquad 0.05 < \beta < 0.70$$

$$\psi_{el,1} = \frac{1}{5.9\beta} \qquad 0.70 \leq \beta$$

– in areas around the supports for continuous beams or cantilevers (negative bending):

$$\psi_{el,2} = \frac{1}{1 + 6.0\left(\beta - \frac{1}{2500\beta}\right) + 1.6\beta^2} \qquad 0.02 < \beta < 0.70$$

$$\psi_{el,2} = \frac{1}{8.6\beta} \qquad 0.70 \leq \beta$$

– at extreme spans of continuous beams (positive bending):

$\psi_{el,4} = (0.55 + 0.025 / \beta)\ \psi_{el,1} \leq \psi_{el,1}$

– in cantilevered areas (negative bending):

$\psi_{el,5} = \psi_{el,2}$

The expressions above are assumed to apply to uniform loadings (parabolic laws of bending moment).

21.3.3. *Elastic ψ_{el} factors. Special cases*

The existence of significant concentrated or local loads may reduce the effective width significantly in relation to that obtained where there are only uniformly-distributed loads.

In central span areas subjected to local loads (linear laws of bending moment), the reduction factor is expressed as follows:

– if the concentrated load is applied at L/2:

$$\psi_{el,3} \cong (1.115 - 5.74\beta. \qquad 0.02 < \beta \leq 0.05$$

$$\psi_{el,3} = \frac{1}{1 + 4.0\beta + 3.2\beta^2} \qquad 0.05 \leq \beta$$

– if the concentrated load is applied at x < L/2:

$\psi_{el,3} = 0.33\ (2\ \psi_{el,3}\ (\beta^*_x) + \psi_{el,3}\ (\beta^*_{L-x}))$, where:

$\psi_{el,3}\ (\beta^*_x)$ the value of $\psi_{el,3}$ obtained for a $\beta^*_x = 0.5\ b_0/x$;

$\psi_{el,3}\ (\beta^*_{L-x})$ the value of $\psi_{el,3}$ obtained for a $\beta^*_{L-x} = 0.5\ b_0 / (L-x)$;

– for cantilevers subjected to concentrated loads at their extremes, the following may also be used:

$\psi_{el,5} = \psi_{el,2}$

The ψ_{el} factors in subsection 21.3.2 may be used where bending is mainly due to uniform loading and the members are isostatic or continuous and fulfil the geometric span limits in subsection 21.3.1.

Where the effect of concentrated or local loads is sufficiently significant in relation to that of distributed loads and overloads, or where the conditions for using the effective lengths L given in Figure 21.3.1 are not met, a single global reduction factor may be used for the section, obtained by the following expression:

$$\psi_{el} = \frac{\Sigma M_i}{\Sigma \frac{M_i}{\psi_{el,i}}} \text{ where:}$$

Mi bending moment in the section for the isolated load 'i', with its corresponding algebraic sign;

ΣM_i total bending moment loading the section;

$\psi_{el,\,I}$ reduction factor for the effective width corresponding to the isolated load 'i', and obtained using the approximate expressions defined above. Effective length L_i is the distance between points of zero bending moment for the load 'i'.

21.3.4. Members under combined local and global loads

Certain structural members are loaded by a combination of the effects of local bending that result from the action of the direct loads that act on them, and the effects, usually axial forces, of the member's involvement in the global response of the structure.

For example, this is the case for the upper chords in lattice structures, in decks of stay systems, stay in lower deck arches, etc.

The structural analysis (local and global) and section resistance checks shall consider the different effective widths of such members in order to reflect adequately the effects of local bending under direct loads and the diffusion on its plane of axial loads from the overall work.

21.3.5. Approximate distribution of direct stresses in flanges

Once ψ_{el} has been found for a section, the transverse distribution of direct stresses over the width of the flange may be estimated in a suitably approximate way, as shown in Figure 21.3.5.

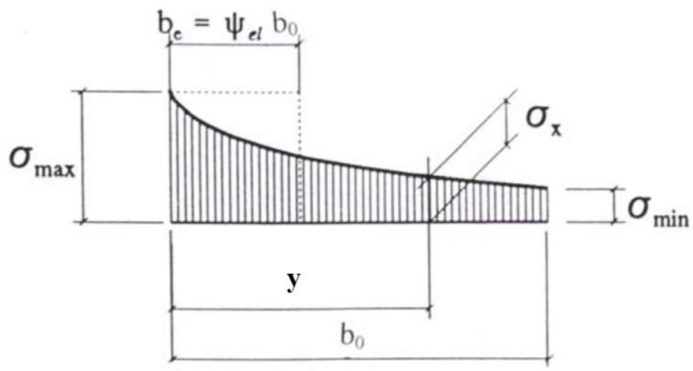


Figure 21.3.5. Approximate law of direct stresses in flanges

- If $\psi_{el} > 0.20$:

  $\sigma_{min} = \sigma_{max} (1.25\ \psi_{el} - 0.25)$;

  $\sigma_x = \sigma_{min} + (\sigma_{max}-\sigma_{min}) (1 - y/b_0)^4$;

- If $\psi_{el} \leq 0.20$:

  $\sigma_x = 0$ for $y \geq 5\ \psi_{el}\ b_0$;

  $\sigma_x = \sigma_{max} (1-y / (5\ \psi_{el}\ b_0))^4$ for $y < 5\ \psi_{el}\ b_0$.

21.4. Effective widths of stiffened flanges for serviceability and fatigue limit states

The presence of stiffeners in flanges on steel beams or box girders (Figure 21.4) increases the effects of shear lag, reducing the effective widths that are to be considered, which may be estimated in a similar way to unstiffened flanges:

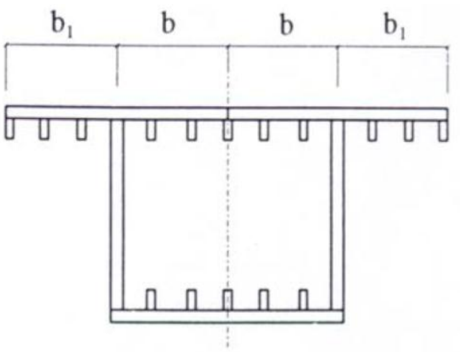


Figure 21.4. Effective widths on stiffened flanges.

$b_e = \psi'_{el}\ b$, for internal flanges;

$b_{1e} = \psi'_{el}\ b_1$, for outstand flanges.

The reduction factors ψ'_{el} are obtained using the expressions set out in the preceding subsection, but replacing the parameter β with:

$\beta' = \alpha\ \beta = \alpha\ b_0/L$

where:

$$\alpha = \sqrt{1 + \frac{A_{sl}}{b_0 t}}$$

$b_0 =$ b for internal, stiffened flanges;

$b_0 =$ b_1 for outstand, stiffened flanges;

$A_{sl} =$ area of longitudinal stiffeners situated within the flange width b_0;

$t =$ flange thickness.

21.5. Effective width of flanges in ultimate limit states

The effects of shear lag on steel section resistance checks may be estimated conservatively using the same elastic reduction factors for the effective flange width, ψ_{el}, defined in subsections 21.3 and 21.4. Alternatively, more precise criteria may be applied as follows:

Where the resistance checks for ultimate limit states consider plastified flanges, the reduction factors for effective flange width in the elastic-plastic range, ψ_{ult}, apply more favourable values than these ψ_{el} values.

This may be assumed for tensioned flanges and compressed flanges in cross-sections of class 1 or 2, in accordance with Section 20.

$\psi_{ult} = \psi_{el}^{\beta} \geq \psi_{el}$ for unstiffened flanges;

$\psi'_{ult} = \psi'^{\beta'}_{el} \geq \psi'_{el}$ for tensioned flanges with longitudinal stiffeners.

For slender cross-sections of class 4, it is necessary to consider the effects of shear lag and the local buckling of compressed parts together when checking ultimate limit state. A reduced effective area for compressed flanges, A_{ef}, must therefore be used, and it is estimated using the expression:

$A_{ef} = A_{c,ef}\,\psi_{ult}$ where:

$A_{c,ef}$ area of the effective section of the compressed slender flange, with or without stiffeners, related to local buckling (see Section 20);

ψ_{ult} reduction factor for the effective width of the compressed flange, related to shear lag, estimated in the elastic range using the expressions for ψ_{el} (see subsections 21.3 and 21.4), but replacing the parameter β with:

$\beta' = \alpha\beta = \alpha\, b_0/L$, where:

$$\alpha = \sqrt{\frac{A_{c,ef}}{b_0 t}}$$

For compressed flanges of class 3 (see Section 20), where there are practically no signs of local buckling or deformation outside the elastic range, the following shall be used for checking ultimate limit state:

$$\psi_{ult} = \psi_{el}$$

21.6. Effective width for local loads applied in the web plane

The application of local loads in the web plane of a section through the flange cover plate causes a distribution of direct stresses in the transverse direction of the member's directrix, and the elastic diffusion over that web plane obeys a non-linear law (see Figure 21.6) which may be calculated approximately using the following expression:

$$\sigma_{z,Ed} = \frac{F_{Ed}}{b_e(t_w + a_{st})}$$ where:

$\sigma_{z,Ed}$ design value of direct stress in the transverse direction to the directrix at the point of the web that is under consideration;

F_{Ed} design value of the transverse force applied;

t_w web thickness;

a_{st} area of the gross cross-section, per unit of length, of any transverse stiffeners situated in the area immediately affected by the load under the cover plate, assuming diffusion at 45º through its thickness. The value of the area of a stiffener divided by the distance between stiffener axes is used.

The effective width, b_e, is obtained using the following expression:

$$b_e = s_e \sqrt{1 + \left(\frac{z}{s_e n}\right)^2}$$ where:

$$n = 0.636 \sqrt{1 + \frac{0.878\, a_{st}}{t_w}}$$

$s_e = s_s + 2\, t_f$ where:

t_f flange thickness;

s_s length of the area where the local load is applied to the flange cover plate;

s_e length of the diffusion area of the local load in the contact section flange-web, assuming load diffusion of 45º in the flange cover plate;

z transverse distance between the section studied and the contact section flange-web, next to the load application area (see Figure 21.6).

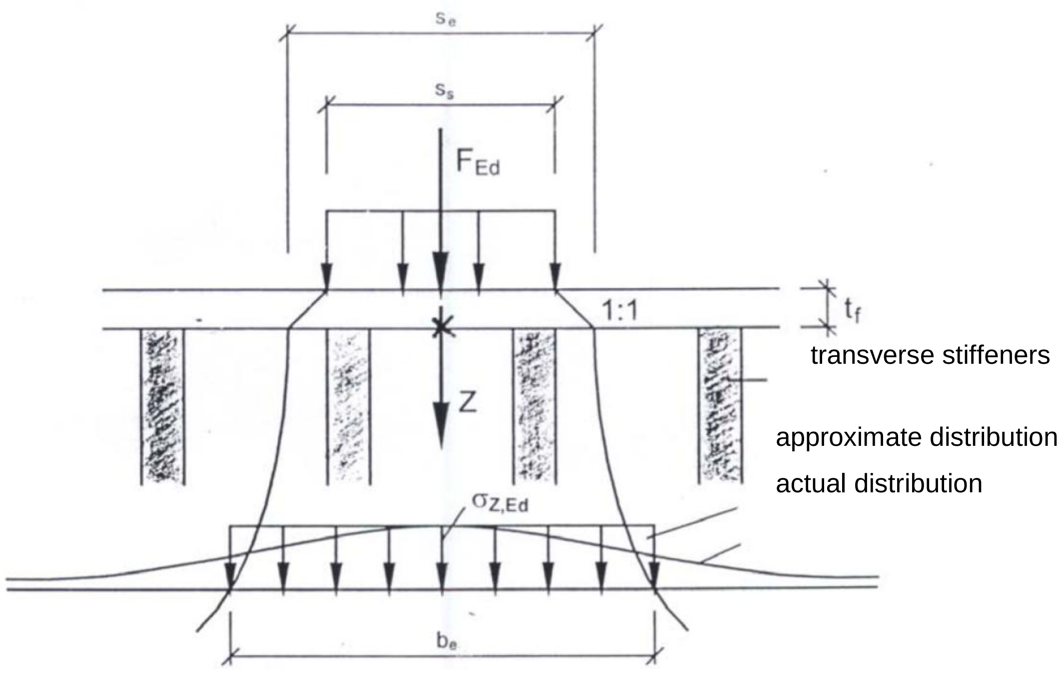


Figure 21.6. Diffusion of local loads on the web plane

Section 22. Consideration of imperfections

22.1. Bases

The second-order analysis of structures the response of which is sensitive to deformation of their initial geometry must take sufficient account of the effects of residual stresses on the steel's non-linear response, as well as inevitable geometric imperfections such as lack of verticality, straightness, flatness or fit and eccentricity of joints, and other execution and assembly tolerances.

These effects may usually be incorporated into the structural analyses by using certain equivalent geometric imperfections.

The effects of imperfections must be considered in the following cases:

a) the effect of imperfections on the global analysis of the structure;

b) the effect of imperfections on the analysis of lateral bracing systems for bending or compressed members;

c) the effect of imperfections on the local analysis for individual members.

Imperfections must be included in the structural analyses for ultimate limit state checks wherever they have a significant effect. It is not usually necessary to consider them for checking serviceability limit state.

22.2. Application method

The effects of equivalent geometric imperfections, which are defined in subsection 22.3, must be included in the global analysis of translational structures that are susceptible to lateral instability phenomenon (Sections 23 and 24). The forces resulting from the analysis must be considered in subsequent resistance checks on the structure's various members.

In the case of braced structures (see subsection 23.3), the equivalent geometric imperfections defined in subsection 22.3 are also used to check the resistance of stabilisation systems for lateral bracing (cores, diaphragm walls, lattices, etc.), in accordance with subsection 23.4.

The effects of the imperfections set out in subsection 22.4 shall also be incorporated into the structural analysis for any lateral bracing systems for bending or compressed members. The forces resulting from the analysis shall be taken into account when designing such bracing systems.

In the case of resistance checking for individual members that are sensitive to instability, using the methods or formulae in subsections 35.1, 35.2 and 35.3 of this Code, the effects of equivalent geometric imperfections in individual members are already implicitly included in such checks.

According to subsection 22.5, in the case of unconventional individual members the resistance checking for which is not explicitly covered by the methods set out in Section 35, and in the case of global instability of structures mentioned in subsection

22.3.2, the local geometric imperfections of the individual members set out in subsections 22.3.2 and 22.3.5 must be incorporated into the second-order analysis of such members or structures, respectively.

If so desired, equivalent geometric imperfections may be replaced by equivalent forces transversal to the directrix of the compressed members, in accordance with subsections 22.3.3 and 22.4.1.

22.3. Imperfections in the global analysis of the structure

Equivalent geometric imperfections must be included in the global analysis of all structures for which second-order effects cannot be discounted. Section 23 set out the conditions of non-translational for structures that allow such effects not to be considered.

The geometry of the design model is the result of incorporating equivalent geometric imperfections into the ideal, theoretical geometry in such a way as to produce the most unfavourable effects.

The imperfections to be considered may therefore be obtained from the modes of global buckling of the structure in the plane of buckling considered.

It is usually necessary to examine the possibility of structural buckling on and outside its plane, but not at the same time (Figure 22.3).

In some structures with low global torsional stiffness, it will also be necessary to consider the possibility of generalised torsional anti-symmetric buckling, by applying imperfections at opposite direction at the two opposite faces of the structure (Figure 22.3).

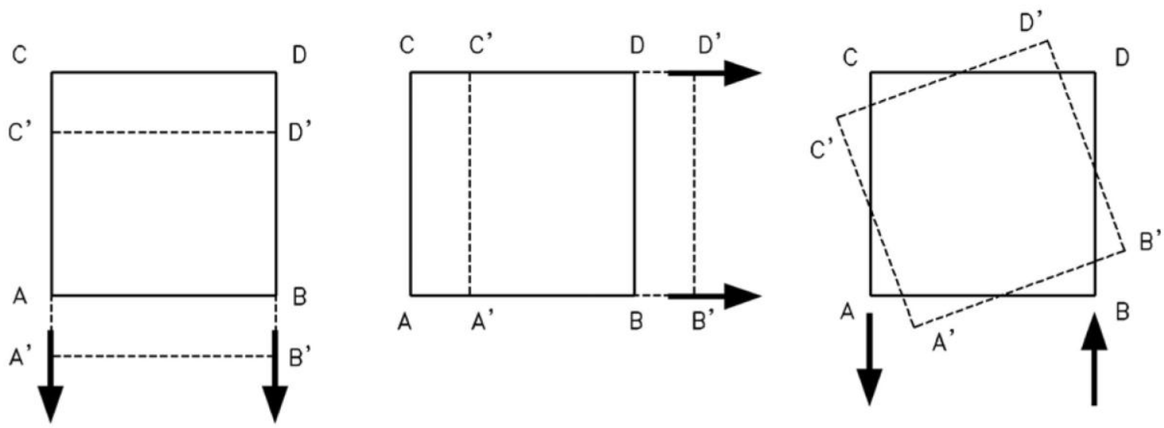


Figure 22.3. Possible forms of structural instability, due to translational or torsional effects

The effect of imperfections on the global analysis of translational structures is the sum of a global verticality defect in the structure and some initial curvatures in all its compressed members, as a second-degree parabola.

If so desired, geometric imperfections may be replaced by a self-balancing system of equivalent transverse forces, in accordance with subsection 22.3.3.

In general, any instability under symmetrical and asymmetrical buckling modes must always be analysed, as well as instability in each combination of actions in which the lowest load amplification factor that produces elastic instability in the system is obtained.

22.3.1. *Equivalent global sway imperfections*

An initial verticality defect shall be considered, such as (see Figure 22.3.1):

$\phi = k_h \cdot k_m \cdot \phi_0$ where:

ϕ_0 basic value of lateral imperfection: $\phi_0 = 1/200$;

k_h reduction factor for height 'h' (in metres) of the structure:

$$k_h = \frac{2}{\sqrt{h}} \text{ with } \frac{2}{3} \le k_h \le 1.0 \;;$$

k_m reduction factor for the number of alignments, 'm', for compressed members (bridge piles or columns in buildings) on the buckling plane being considered:

$$k_m = \sqrt{0.5\left(1+\frac{1}{m}\right)}$$

The expression of 'm' only accounts for members loaded by compression with a design value, N_{Ed}, equal to or greater than 50 % of the mean compression per member, for the buckling plane and combination of actions considered.

In principle, 'm' need only take account of compressed members that extend throughout the total height 'h' of the structure used to obtain k_h.

For building frames, sway imperfections may be disregarded for a certain combination of actions, where:

$H_{Ed} \geq 0.15\, V_{Ed}$ where:

H_{Ed} design value of the result of the total horizontal actions at the base of the building, corresponding to the combination of actions considered;

V_{Ed} design value of the result of the total vertical actions at the base of the building, for that combination of actions.

The structural effects caused by equivalent global sway imperfections are less significant than those caused by horizontal actions that act on the structure.

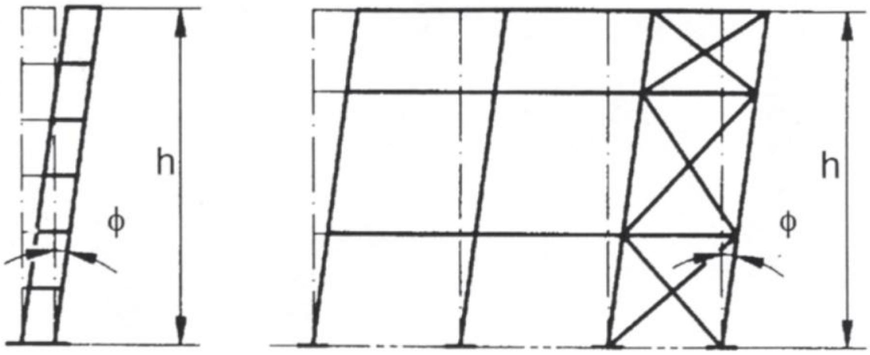


Figure 22.3.1. Global sway imperfections

22.3.2. Equivalent initial curvatures in compressed members

In addition to the initial global verticality defect in the structure, and with the exception of the circumstances set out below, the effect on the global instability of translational structures of local imperfections of all compressed members must be considered where such members fulfil the following two conditions:

a) at least one of the two nodes at the ends of the member may not be regarded as hinged;

b) the non-dimensional slenderness of the member (see subsection 35.1.2) on the buckling plane that is considered, calculated as a double-hinged bar at its ends, is such that:

$$\bar{\lambda} > 0.5\sqrt{\frac{A \cdot f_y}{N_{Ed}}}$$

where:

A area of the member’s cross-section;

N_{Ed} design value of the compression force in the member, for the combination of actions analysed.

This condition is equivalent to the design compression axil of the member, N_{Ed}, being more than 25 % of its critical load according to Euler’s formula, N_{cr}.

In such cases, an equivalent initial bow may be used for the compressed members affected, as a second-degree parabola and maximum amplitude of member imperfection e_0, such that:

Type of buckling curve (see subsection 35.1.2)	Global analysis method for the structure	
	Elastic global analysis	Plastic global analysis
	e_0	e_0
a_0	L/350	L/300
A	L/300	L/250
B	L/250	L/200
C	L/200	L/150
D	L/150	L/100

where L is the member length.

If a more precise analysis is desired, the expressions set out in subsection 22.3.5 may be used alternatively.

22.3.3. Horizontal forces equivalent to imperfections

The effects of global sway imperfections and initial bow imperfections on compressed members may be replaced by systems of equivalent, self-balanced, horizontal forces, proportionate to the vertical loads applied in the relevant combination of actions. This is estimated as follows for each member (see Figure 22.3.3):

a) in the case of initial verticality defects in compressed members:

$$H_{td} = \phi\, N_{Ed}$$

b) in the case of initial bow in compressed members, where it is necessary to consider them in accordance with subsection 22.3.2:

$$q_{td} = \frac{8\, N_{Ed} \cdot e_0}{L^2}$$

$$H_{td} = \frac{4\, N_{Ed} \cdot e_0}{L}$$

where L and NEd are respectively the length and the compression force design value of the member.

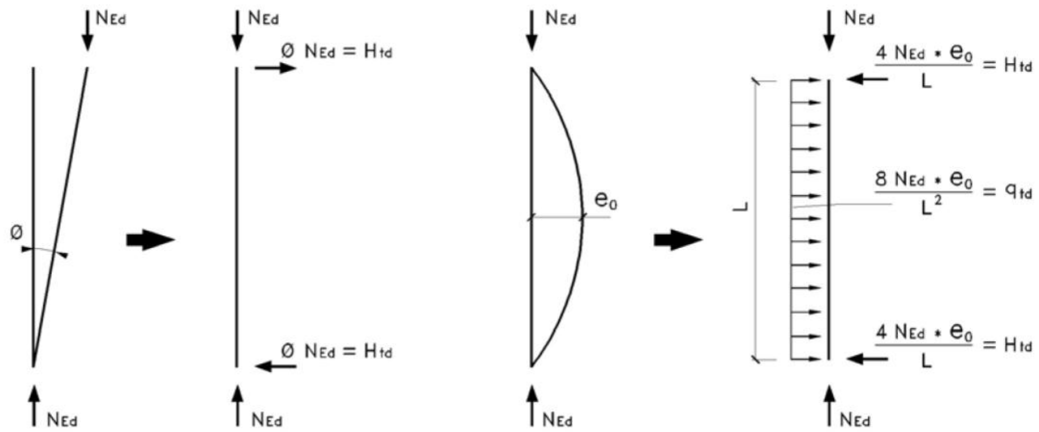
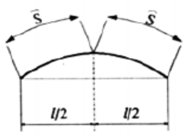
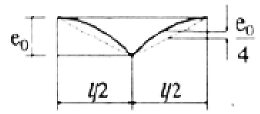
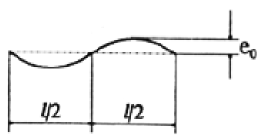


Figure 22.3.3. Horizontal forces equivalent to imperfections

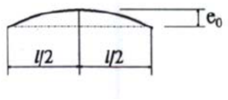
22.3.4. Imperfections for the global analysis of arches

Unless the general method described in subsection 22.3.5 is used in the global instability analysis for arches under buckling on or outside their plane, the geometric imperfections defined below may be used.

22.3.4.1. Buckling on the arc plane

		Form of equivalent geometric imperfections on the arch plane (parabola or sine function)	Value of e_0 for sections corresponding to the various buckling curves			
			a	b	c	d
1	Triple-hinged arch with symmetrical buckling		$\frac{s}{300}$	$\frac{s}{250}$	$\frac{s}{200}$	$\frac{s}{150}$
2	Double-hinged, fixed-ended or triple-hinged arch with antimetric buckling		$\frac{1}{600}$	$\frac{1}{500}$	$\frac{1}{400}$	$\frac{1}{300}$

22.3.4.2. Buckling outside the arch plane

	Form of equivalent geometric imperfections outside the arch plane (parabola or sine function)	Value of e_0 for sections corresponding to the various buckling curves			
		a	b	c	d
Triple-hinged arch Double-hinged arch Fixed-ended arch		$\frac{l_0}{300}$	$\frac{l_0}{250}$	$\frac{l_0}{200}$	$\frac{l_0}{150}$

$l_0 = l$ for $l \leq 20$ m

$l_0 = \sqrt{20 \cdot l}$ for $l \leq 20$ m

22.3.5. Geometric imperfections relating to forms of buckling in complex structures

As an alternative to the equivalent global and local geometric imperfections set out in subsections 22.3.1 and 22.3.2 respectively, a single system of initial geometric imperfections may be defined, similar to the shape of the elastic critical buckling mode of the structure, for the combination of actions and the buckling plane under consideration, and with an amplitude given by:

$$\eta_{init} = e'_o \, \eta_{cr}$$

$$e'_o = e_o \left(\frac{N_{cr}}{EI\eta''_{cr,max}} \right) = e_o \left(\frac{1}{\bar{\lambda}^2} \frac{N_{Rk}}{EI\eta''_{cr,max}} \right)$$

where:

η_{cr} Shape of the elastic critical buckling mode of the structure, where $EI\eta''_{cr,max}$ is the bending moment at the critical cross-section due to η_{cr}.

$$e_0 = \alpha \left(\bar{\lambda} - 0.2 \right) \frac{M_{Rk}}{N_{Rk}} k_\gamma$$

where:

$$k_\gamma = \frac{1 - \chi \bar{\lambda}^2 / \gamma_{M1}}{1 - \chi \bar{\lambda}^2} \geq 1.0$$

α imperfection factor for the relevant buckling curve for the critical cross-section, in accordance with Table 35.1.2.a;

χ reduction factor for the relevant buckling mode, in accordance with subsection 35.1.2;

$\bar{\lambda}$ relative non-dimensional slenderness of the structure, obtained in accordance with the following;

– It is assumed that some forces are applied to the nodes of the structure, so all elements are loaded by design axial forces N_{Ed}, resulting from a first-order global analysis for the combination of actions considered. Bending moment at the elements may be disregarded.

– For that combination of actions, the critical elastic instability mode of the structure and the critical minimum amplifier coefficient α_{cr}, are obtained for the above distribution of design axial forces N_{Ed}, when starting the elastic instability.

– In first-order analysis, it is also obtained the minimum amplifier coefficient α_{uk}, of such distribution of design axial forces N_{Ed}, when the characteristic resistance N_{Rk}, is reached in the cross-section of the weakest element, without considering the buckling effects.

– the relative non-dimensional slenderness of the structure, for such combination of actions, is:

$$\bar{\lambda} = \sqrt{\frac{\alpha_{uk}}{\alpha_{cr}}}$$

MRk, NRk characteristic moment and axial resistance, respectively, of the critical cross-section, in accordance with subsections 34.3 and 34.4. The quotient $\frac{M_{Rk}}{N_{Rk}}$ will thus be:

$\frac{W_{pl}}{A}$ for sections of class 1 or 2;

$\frac{W_{el,min}}{A}$ for sections of class 3;

$\frac{W_{ef,min}}{A_{ef}}$ for sections of class 4.

22.4. Imperfections for analysis of bracing systems

The effects of equivalent geometric imperfections must be included into the analysis of bracing systems which are required to provide lateral stability of bending or compressed members.

An equivalent initial bow will be considered in the members that are to be stabilised, such that:

$e_0 = k_m L/500$ where:

L span of the bracing system;

k_m reduction factor for the number of members to be considered, which may be estimated in accordance with the following:

$$k_m = \sqrt{0.5\left(1+\frac{1}{m}\right)}$$ where 'm' is the number of members to be restrained by the bracing system under consideration.

22.4.1. Equivalent transverse forces on the bracing

For convenience, the effects of geometric imperfections deriving from the initial bow imperfections of the members that are to be restrained may be replaced by a system of equivalent forces of value (see Figures 22.4.1.a and 22.4.1.b):

$$q = \Sigma N_{ed} \cdot 8 \cdot \frac{e_0 + \delta_q}{L^2},$$ where:

δ_q in plane deflection of the bracing system, estimated on the basis of a first-order elastic design under the action of forces 'q' and of any external action that places a load on the bracing system.

It is therefore necessary to use an iterative process.

If a second-order analysis is used, δ_q may be taken as equal to zero, but such an analysis shall include all the forces that load the stabilising system;

N_{Ed} maximum value of axial force loading each member that is to be stabilised, and assumed to be uniform throughout the length L of the bracing system. For non-uniform forces, this hypothesis is slightly conservative.

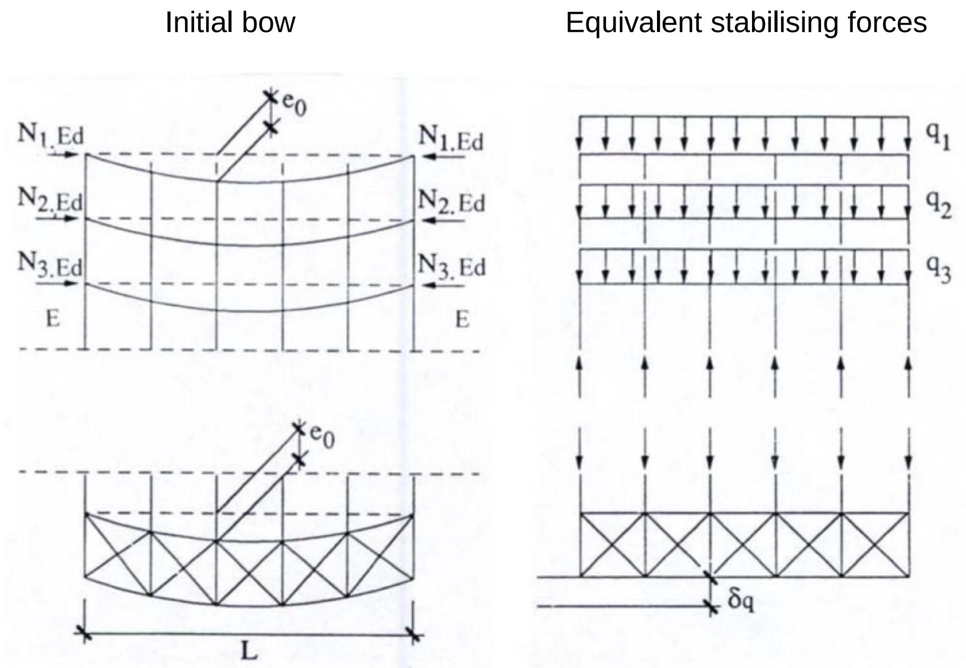


Figure 22.4.1.a. Imperfections in the bracing system

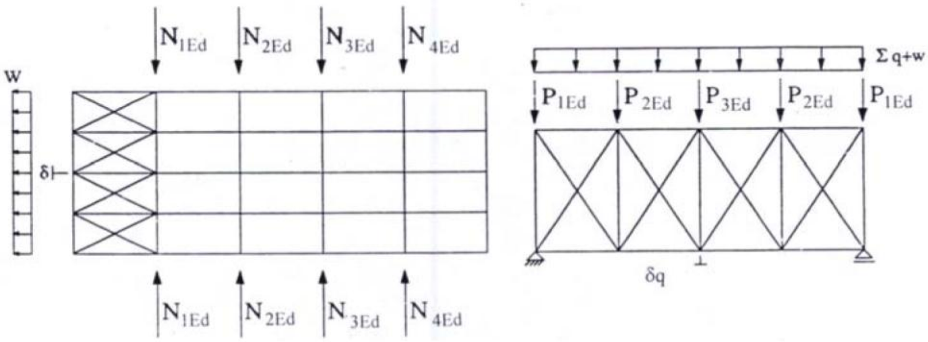
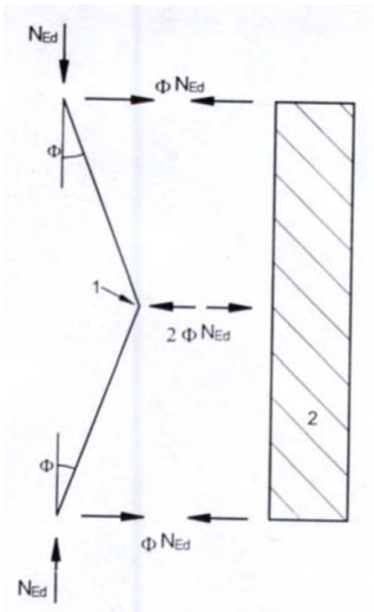


Figure 22.4.1.b. Equivalent forces, including external action

Where a bracing system restrains bending or compressed members that are spliced and does not transmit bending moment, it must also be verified that the bracing system is capable of resisting a local transverse force equal to k_m N_{Ed}/100, transmitted by each compressed member in the joint section, and of transmitting it to the bracing points adjacent to that member (see Figure 22.4.1.c). In such a case, any external loads acting on the bracing systems shall also be included, but any such forces arising from the imperfections defined above are not to be added.



$\Phi = k_m \Phi_0$ $\qquad \Phi_0 = 1/200$

$2\Phi N_{Ed} = k_m N_{Ed}/100$

1. Splice.

2. Bracing system.

Figure 22.4.1.c. Local forces on the bracing system for members with a continuous join.

Each member of lateral support and connections to the bracing system must also be capable of resisting a force equal to k_m $\Sigma N_{ed}/100$ of the members that are to be restrained.

22.5. Imperfections in the local analysis of individual members

The effects of local imperfections in individual compressed or bending members are usually considered implicitly in the formulae for verifying instability limit states in Section 35.

Alternatively, or in cases where such formulae do not apply (for example, in certain non-uniform section members, or those that have variable compression levels throughout their length, or in the presence of transverse loads or complex connection conditions at the ends, etc), the resistance of compressed or bending members to instability, either on its plane or laterally, may be justified by a second-order analysis using certain local initial imperfections, as equivalent parabolic curvatures with the maximum amplitude defined in subsection 22.3.2 or, in order to be more precise, in subsection 22.3.5.

In the second-order analysis of lateral buckling problems on bending members, a value of 0.5 e_0 may be used for sway imperfection, where e_0 is the amplitude of the equivalent initial imperfection for buckling on a plane perpendicular to bending plane (usually in relation to the section’s axis of smaller inertia). It is not usually necessary to incorporate an additional torsional imperfection.

Section 23. Lateral stability of structures

23.1. Lateral stiffness

The influence of second-order effects on the resistance of a structure basically depends on its lateral stiffness.

The lateral stiffness of a structure is usually ensured by means of:

a) its own stiffness for systems with rigid nodes;

b) lateral, triangular bracing systems;

c) lateral bracing systems using diaphragm walls or rigid cores;

d) a combination of the aforementioned structural schemes.

When designing semi-rigid joints (see subsection 57.4) between structural members, their moment-rotation diagrams (see subsection 57.2) must be taken into account when evaluating lateral stiffness.

Foundations must usually be designed so that the effects of lateral displacement and rotation at the base are discounted.

For asymmetrical structures in plan, the effects of interaction between torsion and bending must be considered when checking lateral stability.

Lateral stability must be ensured both for the structure in service and during its various phases of construction.

23.2. Classification of non-translational and translational structures

A structure may be classified as non-translational when its lateral stiffness allows disregarding the influence of second-order effects on its resistance. The global analysis of non-translational structures may be performed in accordance with the theory for the first-order analysis.

A structure may be regarded as non-translational related to a given mode of lateral instability and a determined combination of actions, if it fulfils the following criterion:

$$\alpha_{cr} = \frac{F_{cr}}{F_{Ed}} \geq 10$$ where an elastic global analysis is performed;

$$\alpha_{cr} = \frac{F_{cr}}{F_{Ed}} \geq 15$$ where a plastic or elastic-plastic global analysis is performed,

where:

F_{cr} critical elastic instability load for the mode of global buckling considered, under the combination of actions to be considered;

F_{Ed} design load that acts on the structure for the said combination of actions;

α_{cr} amplification factor for which the configuration of design loads must be multiplied to cause elastic lateral instability, in accordance with the mode of global buckling under consideration.

All load combinations for which α_{cr} does not satisfy this non-translational criterion must be analysed.

23.2.1. Non-translational criterion for conventional building structures

A structure may be classified as non-translational when its lateral stiffness allows disregarding the influence of second-order effects of the forces or on its global structural performance. The global analysis of non-translational structures may be performed in accordance with the theory for the first-order analysis. Second-order effects must only be considered for resistance checks on individual compressed members, in accordance with subsection 22.5 and Section 35.

In simple frames with flat-cover or gently sloping lintels, as well as in flat building frames, with rigid nodes, the non-translational criterion in subsection 23.2 may be assumed to be met if, on each storey and for the combination of actions under consideration, the criterion is met for:

$$\alpha_{cr} = \left(\frac{F_{H,Ed}}{F_{v,Ed}}\right) \times \left(\frac{h_p}{\delta_{H,Ed}}\right)$$ where:

$F_{H,Ed}$ design value of the horizontal force, estimated at the lower level of each storey, and resulting from the horizontal loads that act above that level, including the effects of the imperfections given in Section 22;

$F_{v,Ed}$ design value of the vertical force, estimated at the lower level of each storey, and resulting from the vertical loads that act above that level;

h_p height of the storey under consideration;

$\delta_{H,Ed}$ relative horizontal displacement between the upper and lower levels of the storey under consideration, under the action of external, horizontal and vertical design actions and transverse forces equivalent to the imperfections set out in Section 22, for the combination of actions considered.

23.3. Classification of braced and unbraced structures

A structure may be classified as braced when its lateral stiffness is ensured by means of a brace stabilising system that allows the influence of second-order effects on its structural response to be discounted. The global analysis may therefore be performed in accordance with the theory for the first-order analysis.

In order for the structure to be considered braced, the stiffness of the brace system must be verified using the non-translational criteria set out in subsection 23.2, applied to the whole of the structure that is to be classified, including the brace system to which it is connected.

The brace system must also satisfy the requirements set out in subsection 23.4.

23.4. Analysis of brace systems

The brace system must be designed to withstand:

– the effects of the imperfections set out in Section 22, both for the brace system itself and for all the structures that it braces;

– all horizontal forces that may load the structures that it braces;

– all horizontal and vertical forces that act directly on the brace system.

All these actions together might be assured to load just the brace system, without having a significant impact on the response of the structures that the system braces.

Section 24. Methods for analysing global stability of structures

24.1. Basic principles

In all structures without sufficient lateral stiffness to be considered non-translational or braced in accordance with the criteria set out in subsections 23.2 and 23.3 respectively, their global lateral stability must be checked in accordance with the methods described in this Section, which consider the second-order effects as well as the equivalent geometric imperfections defined in Section 22.

Depending on the structure type and method for the global analysis to be performed, the second-order effects and equivalent geometric imperfections may be considered according to one of the following methods:

a) through a global translational analysis that includes all such effects, i.e. the equivalent global sway imperfections of the translational structure as defined in subsection 22.3.1, and the equivalent initial bow for imperfections of the individual compressed members defined in subsection 22.3.2. In both cases, the equivalent transverse forces set out in subsection 22.3.3 may be used alternatively. For complex structures, it is worth using a single system of geometric imperfections similar to the modes of buckling, in accordance with subsection 22.3.5;

b) through a global translational analysis of the structure that only considers, apart from the items mentioned in subsection 22.3.2, the effects of equivalent global sway imperfections, followed by a check on the effects of instability on individual compressed members.

   Subsection 22.3.2 sets out the conditions under which the global mobility analysis must also incorporate equivalent imperfections for linear bows in certain compressed members;

c) in certain basic cases discussed in subsection 35.1 and Annex 5, it may be sufficient to verify the stability on individual compressed members in accordance with Section 35, using appropriate ‘buckling lengths’ (see subsection 35.1 and Annex 5), based on the structure’s global instability mode and using the loads obtained according to the first-order theory, without considering the equivalent geometric imperfections.

If method a) is used, it is sufficient to verify the stability of individual compressed members using the structure's second-order global analysis, and no additional verification is needed for the resistance check on the various sections under the forces resulting from the design.

If method b) is used, the stability of the individual compressed members must subsequently be checked, including second-order effects and local imperfections in such members that are not previously considered in the second-order global analysis of the structure (for example, buckling due to combined bending and compression or lateral buckling of the member).

The methods set out in subsection 35.3 may therefore be used where applicable, otherwise the more general methods in subsection 22.5 may be used, assuming that the individual member and its buckling length (less than or equal to the distance between adjacent points with transverse displacement prevented) are subjected to bending and compression loads at the extremities of the member, these being obtained from the global translational analysis. In general, such individual members may also be analysed, with their actual lengths, using the general non-linear elastic-plastic method discussed in subsection 24.4, and subjected to the aforementioned loads at both ends.

Methods a) and b) require second-order effects under the external loads and equivalent effects of the imperfections to be taken into account using an adequate structural analysis:

– general, elastic-plastic, non-linear analysis method using the second-order theory, in accordance with subsection 24.4;

– elastic methods using the second-order theory;

– where applicable (see subsection 24.2), using an approximate method consisting of performing a first-order elastic analysis, followed by amplification of the results of that analysis (bending forces, shearing forces and lateral displacement, for example) using adequate factors in accordance with subsection 24.2;

– in certain specific cases relating to building structures, as discussed in subsection 24.3, elastic methods may be applied to frames with suitably local plastic hinges, giving due consideration to the reduction in the structure's lateral stiffness owing to the presence of such plastic hinges.

If non-linear analysis methods are used, the principle of superposition shall not apply. Independent checks must therefore be carried out on all combinations of actions and associated modes of instability that may be relevant.

24.2. Elastic analysis of translational structures

Second-order elastic analyses under the action of external loads and equivalent geometric imperfections apply to all types of translational structure.

Alternatively, it may be sufficient to perform a first-order elastic analysis within the scope given below and under the external actions and effects of equivalent initial geometric imperfections, and to amplify the bending moment, shearing forces and other effects caused strictly by lateral deformation by the following factor:

$$\left(\frac{1}{1-\frac{1}{\alpha_{cr}}}\right) \qquad \text{where:} \qquad \alpha_{cr} \geq 3.0$$

α_{cr} amplification factor for which the configuration of design loads must be multiplied to achieve elastic instability, in accordance with the mode of global buckling under consideration, and as defined in subsection 23.2.

This simplified method only applies to:

a) building frames on a single storey;

b) normal building frames with various storeys, provided that all the storeys have similar conditions in respect of:

- vertical load distribution;
- horizontal load distribution; and
- lateral frame stiffness in relation to horizontal actions.

The conditions relating to floor or roof lintels set out in subsection 23.2.1 shall also be met;

c) bridges or other types of structure, or any members, where α_{cr} > 3.0 and it may be considered that the sections subjected to maximum bending in the first-order analysis (including the effects of imperfections) are basically the same as those that are amplified by the second-order effects (PΔ effects).

In other cases, a general, second-order, elastic analysis method must be used.

24.3. Plastic analysis of translational structures

In general, performing plastic analyses on translational structures is not-allowed, but in strictly defined cases under this paragraph, or where the general elastic-plastic method is applied using the second-order theory described in subsection 24.4 it may be allowed.

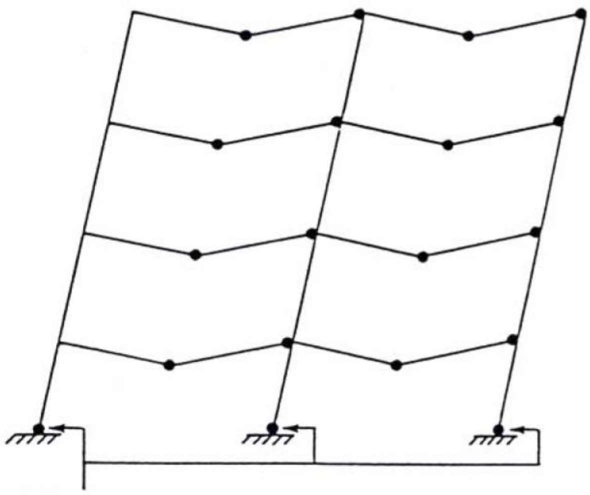
A rigid-plastic analysis of a translational structure, which indirectly considers the second-order effects due to global lateral instability, is only permitted for building structures that meet the following conditions:

- the cross-sections of members (lintels, supports) liable to develop a plastic hinge must satisfy the ductility requirements set out in subparagraph 20.5;
- sections where plastic hinges form must be symmetrical and be suitably braced to withstand lateral buckling and buckling on the frame's perpendicular plane;
- the amplification factor α_{cr} (see subsection 23.2) shall be greater than or equal to 5.0;
- even if the preceding limitations are fulfilled, the analysis is still restricted to the following conventional building structures:

a) orthogonal frames of one or two storeys where one of the following two conditions is met:

- plastic hinges may not develop in the supports;
- plastic hinges may appear at the top or bottom of supports, but not in between, and they also fulfil the requirements set out in subsection 24.3.1;

b) orthogonal frames with several storeys where the translational breakage mechanism is an incomplete mechanism where hinges are only permitted in the lower sections of supports at the bottom storeys. The critical sections must also be designed so that such possible hinges in support bases are the last ones that occur in the structure, and remaining all the support sections throughout the height of the structure in the elastic range throughout the development of successive hinges in lintels (see Figure 24.3).



plastic hinges only at the bottom of supports on the bottom storey

Figure 24.3. Incomplete plastic mechanism for orthogonal frames with various storeys

Second-order forces may be considered indirectly for translational structures with rigid/plastic hinges, where applicable, using the elastic second-order analysis models for translational structures described in subsections 24.1 and 24.2, provided that they adequately consider the plastic hinges in the stiffness conditions for the corresponding elastic models.

24.3.1. Requirements for supports for the plastic analysis

For orthogonal building frames of one or two storeys where the requirements set out in subsection 24.3 are verified so as to allow a simplified, plastic, rigid translational analysis that involves plastic hinges at the ends of all or any supports, it is necessary to ensure that such sections have adequate capacity to rotate under the simultaneous action of the compression forces that load them.

This requirement may be considered to have been satisfied if, under the axial forces obtained by a first-order plastic, rigid analysis, it is verified that:

$$\bar{\lambda} \leq 0.3\sqrt{A \cdot f_y / N_{Ed}}$$ or its equivalent:

$$N_{cr} \geq 11.11\ N_{Ed}$$

where:

A area of the support, assuming a constant section;

f_y yield strength of the steel;

N_{Ed} design value of the axial compression force in the support;

N_{cr} critical axial force of the support according to Euler's formula, assuming it to be double-jointed;

$\bar{\lambda}$ non-dimensional slenderness, corresponding to the ideal critical axial force for buckling in the support, and using the height of the support to give a conservative buckling length.

24.4. General, non-linear analysis method using second-order theory

The effects of non-linear performance of materials and the equilibrium of the structure on its deformed geometric configuration are considered simultaneously in the general, non-linear analysis method using second-order theory.

Furthermore, geometric imperfections equivalent to construction and material (residual stress) imperfections, as set out in Section 22, must also be taken into account.

The elastic-plastic effects of non-linear material shall be considered in accordance with subsection 19.5, for sections with and without stiffeners.

This method ensures that the structure neither has any global or local instability conditions at the level of its constituent members, nor exceeds the resistance capacity of the various sections of those members, in respect of the various combinations of actions, together with the corresponding partial factors for safety and the associated instability modes.