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Structural Analysis and Design

Cold Formed Steel Frame Design Manual

Eurocode 3 1-3:2006

A PRODUCT OF COMPUTERS & STRUCTURES, INC.

Proudly developed in the United States of America

Cold Formed Steel Frame Design Manual

EC 3 1-3 2006

for

SAP2000®

ISO SAP121520M49 Rev. 1
Proudly developed in the United States of America February 2021

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Contents

1 Introduction

The design/check of cold-formed steel frames in accordance with the “Eurocode 3 – Design of steel structures – Part 1-3: General rules – Supplementary rules for cold-formed members and sheeting” (EN 1993-1-3, 2006) is seamlessly integrated within the program. Initiation of the design process, along with control of various design parameters, is accomplished using the Design menu. Automated design at the object level is available for any one of a number of user-selected design codes, as long as the structures have first been modeled and analyzed by the program. Model and analysis data, such as material properties and member forces, are recovered directly from the model database, and are used in the design process in accordance with the user defined or default design settings. As with all design applications, the user should carefully review all of the user options and default settings to ensure that the design process is consistent with the user’s expectations. The EN 1993-1-3 cold-formed steel frame design is integrated with the second-order P-Delta and P-δ effects, provided the user specifies that a nonlinear P-Delta analysis be performed.

The default implementation in the software is the CEN version of the code. Additional country specific National Annexes are also included. The Nationally Determined Parameters are noted in this manual with [*NDP*]. Changing the country in the Design Preferences will set the Nationally Determined Parameters for the selected country as defined in Appendix F.

It is important to read this entire manual before using the design algorithms to become familiar with any limitations of the algorithms or assumptions that have been made.

For referring to pertinent sections of the corresponding code, a unique prefix is assigned for each code.

- Reference to the EN 1993-1-1:2005 code is identified with the prefix “EC3-1.”
- Reference to the EN 1993-1-3:2006 code is identified with the prefix “EC3-3.”
- Reference to the EN 1993-1-5:2006 code is identified with the prefix “EC3-5.”
- Reference to the ENV 1993-1-1:1992 code is identified with the prefix “EC3-1992.”

Reference to the Eurocode 1990:2002 code is identified with the prefix “EC0.”

1.1 Units

The EC3-3 design code is based on Newton, millimeter, and second units and, as such, so is this manual, unless noted otherwise. Any units, imperial, metric, or MKS may be used in the software in conjunction with Eurocode 3 design.

1.2 Axes Notation

The software analysis results refer to the member local axes system, which consists of the 2-2 axis that runs parallel to the web and the 3-3 axis that runs parallel to the flanges. Therefore, bending about the 2-2 axis would generate minor axis moment, and bending about the 3-3 axis would generate major axis moment. The Eurocode 3 design code refers to y-y and z-z axes, which are equivalent to the software 3-3 and 2-2 axes, respectively. These notations may be used interchangeably in the design algorithms, although every effort has been made to use the design code convention where possible.

1.3 Stress Check

Cold-formed steel frame design/check consists of calculating the flexural, axial, and shear forces or stresses at several locations along the length of a member, and then comparing those calculated values with acceptable limits. That comparison produces a demand/capacity ratio, which typically should not exceed a value of one if code requirements are to be satisfied. The program does not do the connection design.

Program output can be presented graphically on the model, in tables for both input and output data, or in calculation sheets prepared for each member. For each presentation method, the output is in a format that allows the engineer to quickly study the stress conditions that exist in the structure, and in the event the member is not adequate, aid the engineer in taking appropriate remedial measures.

2 Design Algorithms

This chapter provides an overview of the basic assumptions, design preconditions, and some of the design parameters that affect the design of cold-formed steel frames.

2.1 Check Capability

The program has the ability to check adequacy of a section (shape) in accordance with the requirements of the selected design code. **General sections and sections defined by the Section Designer will not be checked as one of the limitations of the program.** Other limitations include the following calculations not being performed:

- Web crippling strength
- Torsion capacity
- All strengths of welded members

To check adequacy of a section, the program checks the demand/capacity (D/C) ratios at a predefined number of stations for each design load combination. It calculates the envelope of the D/C ratios. It also checks the other requirements on a pass or fail basis. If the capacity ratio remains less than or equal to the D/C ratio limit, which is a number close to 1.0, and if the section passes all the special requirements, the section is considered to be adequate, else the section is considered to be failed. The D/C ratio limit is taken as 1.0 by default. However, this value can be overwritten in the Preferences (Appendix D) and Overwrites (Appendix E).

To check adequacy of an individual section, the user must assign the section using the **Assign** menu. In that case, both the analysis and design sections will be changed.

2.2 Check Stations

For each design combination, cold-formed steel frame members (beams, columns) are checked at a number of locations (stations) along the length of the object. The stations are located at equally spaced segments along the clear length of the object. By default, at least three stations will be located in a column or brace member, and the stations in a beam will be spaced at most 2 feet apart (0.5 m if the model has been created in metric units). The user can overwrite the number of stations in an object before the analysis is run and refine the design along the length of a member by requesting more stations. Refer to the program Help for more information about specifying the number of stations in an object.

2.3 Demand/Capacity Ratios

Determination of the controlling demand/capacity (D/C) ratios for each cold-formed steel frame member indicates the acceptability of the member for the given loading conditions. The steps for calculating the D/C ratios are as follows:

- The factored forces are calculated for axial, flexural, and shear at each defined station for each design combination. The bending moments are calculated about the geometric axes for C, Hat, I-Shape, T, Box, and Pipe sections for which the principal axes coincide with the geometric axes. For Z section, factor moments are determined about the geometric axes. For Single-Angle sections, the design determines the axes of bending provided the lateral-torsional restraint condition, and bending moments are re-calculated according to the axes of bending determined.

  Shear forces are calculated for directions along the geometric axes for all shapes of section.

- The nominal strengths are calculated for compression, tension, bending, and shear based on the equations provided later in this manual. For axial compression, the nominal strengths are determined based on the geometric axes for C, Hat, I-Shape, T, Box, and Pipe sections. For Z and Angles sections, the lateral-torsional restraint condition is checked to determine the buckling axes and all computations related to flexural strength are based on that

  For flexure, the nominal strengths are calculated based on the geometric or principal axes of bending. For the C, Hat, I-Shape, T, Box, and Pipe sections, the principal axes coincide with their geometric axes. For Z section, the nominal flexural strength is calculated based on geometric axes of bending. For Angle sections, the lateral-torsional restraint condition is examined to determine the bending axes and all computations related to flexural strength are based on that.

  The nominal strength for shear is calculated along the geometric axes for all sections.

- Factored forces are compared to nominal strengths to determine D/C ratios. In either case, design codes typically require that the ratios not exceed a value of one. A capacity ratio greater than one indicates a member that has exceeded a limit state.

2.4 Design Load Combinations

The design load combinations are the various combinations of the prescribed load cases for which the structure needs to be checked. The program creates a number of default design load combinations for cold-formed steel frame design. Users can add their own design combinations as well as modify or delete the program default design load combinations. An unlimited number of design load combinations can be specified.

To define a design load combination, simply specify one or more load cases, each with its own scale factor. The scale factors are applied to the forces and moments from the load cases to form the factored design forces and moments for each design load combination.

For normal loading conditions involving static dead load (DL), live load (LL), roof live load (RL), snow load (SL), wind load (WL), earthquake load (EL), notional load (NL), and dynamic response spectrum load (EL), the program has built-in default design combinations for the design code. These are based on the code recommendations.

The default design combinations assume all load cases declared as dead or live to be additive. However, each load case declared as wind, earthquake, or response spectrum cases, is assumed to be non-additive with other loads and produces multiple lateral combinations. Also, static wind, earthquake and notional load responses produce separate design combinations with the sense (positive or negative) reversed. The notional load patterns are added to load combinations involving gravity loads only. The user is free to modify the default design preferences to include the notional loads for combinations involving lateral loads.

For other loading conditions involving moving load, time history, pattern live load, separate consideration of roof live load, snow load, and the like, the user must define the design load combinations in lieu of or in addition to the default design load combinations. If notional loads are to be combined with other load combinations involving wind or earthquake loads, the design load combinations need to be defined in lieu of or in addition to the default design load combinations.

For multi-valued design combinations, such as those involving response spectrum, time history, moving loads and envelopes, where any correspondence between forces is lost, the program automatically produces sub-combinations using the maxima/minima values of the interacting forces. Separate combinations with negative factors for response spectrum load cases are not required because the program automatically takes the minima to be the negative of the maxima response when preparing the sub-combinations described previously.

The program allows live load reduction factors to be applied to the member forces of the reducible live load case on a member-by-member basis to reduce the contribution of the live load to the factored responses.

combinations.

2.5 Member Unsupported Lengths

The column unsupported lengths are required to account for column slenderness effects for flexural buckling and for lateral-torsional buckling. The program automatically determines the unsupported length ratios, which are specified as a fraction of the frame object length. These ratios times the frame object lengths give the unbraced lengths for the member. These ratios can also be overwritten by the user on a member-by-member basis, if desired, using the overwrite option.

Two unsupported lengths, l_{33} and l_{22}, as shown in Figure 2-2 are to be considered for flexural buckling. These are the lengths between support points of the member in the corresponding directions. The length l_{33} corresponds to instability about the 3-3 axis (major axis), and l_{22} corresponds to instability about the 2-2 axis (minor axis). The length l_{LTB} not shown in the figure, is also used for lateral-torsional buckling caused by major direction bending (i.e., about the 3-3 axis).

In determining the values for l_{22} and l_{33} of the members, the program recognizes various aspects of the structure that have an effect on these lengths, such as member connectivity, diaphragm constraints and support points. The program automatically locates the member support points and evaluates the corresponding unsupported length.

It is possible for the unsupported length of a frame object to be evaluated by the program as greater than the corresponding member length. For example, assume a column has a beam framing into it in one direction, but not the other, at a floor level. In this case, the column is assumed to be supported in one direction only at that story level, and its unsupported length in the other direction will exceed the story height.

By default, the unsupported length for lateral-torsional buckling, l_{LTB} is taken to be equal to the l_{22} factor. Similar to l_{22} and l_{33}, l_{LTB} can be overwritten.

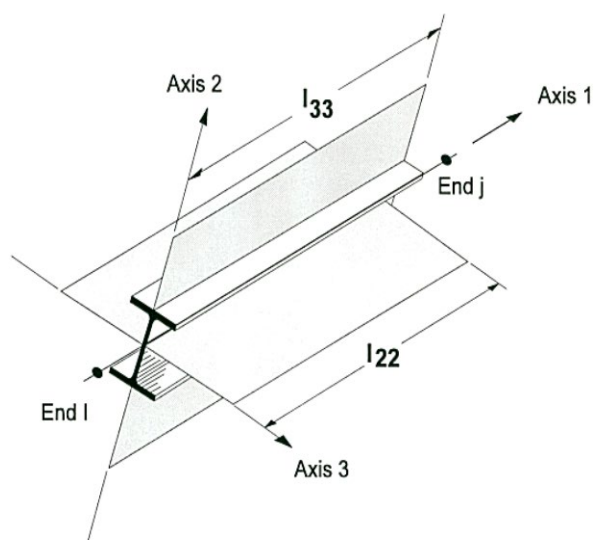


Figure 2-2 Unsupported lengths l_{33} and l_{22}

The unsupported length for minor direction bending for lateral-torsional buckling also can be defined more precisely by using "precise" bracing points in the Lateral Bracing option, which is accessed using the **Design > Lateral Bracing** command. This allows the user to define the lateral bracing of the top, bottom, or both flanges. The bracing can be a point brace or continuous bracing.

The program calculates the unbraced length to determine axial capacity based on the limit state of flexural buckling from this definition. Any bracing at the top or bottom, or both, is considered enough for flexural buckling in the minor direction. While checking moment capacity for the limit state of lateral-torsional buckling (LTB) at a station, the program dynamically calculates the bracing points on the compression flange at the left and at the right of the check station considering the sign of moment diagram. This definition affects only the unbraced lengths for minor direction bending (L_{22}) and lateral-torsional buckling (L_{LTB}). This “exact” method of bracing definition does not allow the user to define unbraced lengths for major direction bending (L_{33}).

There are three sources of unbraced length ratio: (1) “automatic” calculation, (2) “precise” bracing definition, (3) overwrites, with increasing priority in considerations. “Automatic” calculation of the unbraced length is based on member connectivity considering only the members that have been entered into the model. This misses the tiny bracing members. However, such automatically calculated bracing lengths are load combo (moment diagram) independent. This can be reported easily. Similarly, the overwritten values are load combo independent. This allows the program to report the overwritten unbraced length easily. However, if the member has a “precise” bracing definition, the unbraced length can be different at different stations of the member along the length. Also, it can be load combo dependent. Thus, when the unbraced length is reported in the detailed design info, it is reported perfectly considering all three sources as needed. However, when reporting unbraced length on the model shown in the active window, the program-reported value comes from “automatic” calculation or from the overwrites if the user has overwritten it.

2.6 Effects of Breaking a Member into Multiple Elements

The preferred method is to model a beam, column or brace member as one single element. However, the user can request that the program break a member internally at framing intersections and at specified intervals. In this way, accuracy in modeling can be maintained, at the same time design/check specifications can be applied accurately. There is special emphasis on the end forces (moments in particular) for many different aspects of beam, column and brace design. If the member is manually meshed (broken) into segments, maintaining the integrity of the design algorithm becomes difficult.

Manually, breaking a column member into several elements can affect many things during design in the program.

1. The unbraced length: The unbraced length is really the unsupported length between braces. If there is no intermediate brace in the member, the unbraced length is typically calculated automatically by the program from the top of the flange of the beam framing the column at bottom to the bottom of the flange of the beam framing the column at the top. The automatically calculated length factor typically becomes less than 1. If there are intermediate bracing points, the user should overwrite the unbraced length factor in the program. The user should choose the critical (larger) one. Even if the user breaks the element, the program typically picks up the unbraced length correctly, provided that there is no intermediate bracing point.

2. K-factor: Even if the user breaks the member into pieces, the program typically can pick up the K-factor correctly. However, sometimes it cannot. The user should note the K-factors. All segments of the member should have the same K-factors and it should be calculated based

on the entire member. If the calculated K-factor is not reasonable, the user can overwrite the K-factors for all the segments.

3. C_m factor: The C_m factor should be based on the end moments of unbraced lengths of each segment and should not be based on the end moments of the member. The program already calculates the C_m factors based on the end moments of unbraced lengths of each segment. If the break-up points are the brace points, no action is required by the user. If the broken segments do not represent the brace-to-brace unsupported length, the program calculated C_m factor is conservative. If this conservative value is acceptable, no action is required by the user. If it is not acceptable, the user can calculate the C_m factor manually for the critical combination and overwrite its value for that segment.

If the user models a column with a single element and makes sure that the l-factors and K-factors are correct, the factor C_m will be picked up correctly provided no intermediate bracing point. The calculated C_m factor will be slightly conservative if there are intermediate bracing points.

If the user models a column with multiple elements and makes sure that l-factors and K-factors are correct, the factor C_m will be picked up correctly given that the member is broken at the bracing points. The calculated C_m factor will be conservative if the member is not broken at the bracing points.

2.7 Effective Length Factor (*K*)

The effective length method for calculating member axial compressive strength has been used in various forms in several stability-based design codes. The method originates from calculating effective buckling lengths, KL, and is based on elastic/inelastic stability theory. The effective buckling length is used to calculate an axial compressive strength, $N_{b,Rd}$, through an empirical column curve that accounts for geometric imperfections, distributed yielding, and residual stresses present in the cross-section.

There are two types of K-factors in the “Eurocode 3-2005” code. The first type of K-factor is used for calculating the Euler axial capacity assuming that all of the member joints are held in place, i.e., no lateral translation is allowed. The resulting axial capacity is used in calculation of the k factors. This K-factor is named as K_1 in this document. The program calculates the K_1 factor automatically based on nonsway condition. This K_1 factor is always less than 1. The program allows the user to overwrite K_1 on a member-by-member basis.

The other K-factor is used for calculating the Euler axial capacity assuming that all the member joints are free to sway, i.e., lateral translation is allowed. The resulting axial capacity is used in calculating $N_{b,Rd}$. This K-factor is named as K_2 in this document. This K_2 is always greater than 1 if the frame is a sway frame. The program calculates the K_2 factor automatically based on sway condition. The program also allows the user to overwrite K_2 factors on a member-by-member basis. If the frame is not really a sway frame, the user should overwrite the K_2 factors.

The automated K_2-factor calculation is turned off if the user sets the “Consider P-Delta Done?” to be “Yes” in the preferences. In this case, all the columns, beams, and braces are assigned K_2-factors of unity.

Both K_1 and K_2 have two values: one for major direction and the other for minor direction, K_{1major}, K_{1minor}, K_{2major}, K_{2minor}.

There is another K-factor. K_{LTB} for lateral-torsional buckling. By default, K_{LTB} is taken as equal to K_{2minor}. However, the user can overwrite this on a member-by-member basis.

Determination K_2 Factors:

The K-factor algorithm has been developed for building-type structures, where the columns are vertical and the beams are horizontal, and the behavior is basically that of a moment-resisting frame for which the K-factor calculation is relatively complex. For the purpose of calculating K-factor, the objects are identified as columns, beam and braces. All frame objects parallel to the Z-axis are classified as columns. All objects parallel to the X - Y plane are classified as beams. The remainders are considered to be braces.

The beams and braces are assigned K-factor of unity. In the calculation of the K-factor for a column object, the program first makes the following four stiffness summations for each joint in the structural model:

$$S_{ex} = \sum \left(\frac{E_c I_c}{L_c}\right)_x \qquad S_{bx} = \sum \left(\frac{E_b I_b}{L_b}\right)_x$$

$$S_{cy} = \sum \left(\frac{E_c I_c}{L_c}\right)_y \qquad S_{by} = \sum \left(\frac{E_b I_b}{L_b}\right)_y$$

where the *x* and *y* subscripts correspond to the global *X* and *Y* directions and the *c* and *b* subscripts refer to column and beam. The local 2-2 and 3-3 terms EI_{22}/L_{22} and EI_{33}/L_{33} are rotated to give components along the global *X* and *Y* directions to form the $(EI/L)_x$ and $(EI/L)_y$ values. Then for each column, the joint summations at END-I and the END-J of the member are transformed back to the column local 1-2-3 coordinate system, and the G-values for END-I and the END-J of the member are calculated about the 2-2 and 3-3 directions as follows:

$$G_{22}^I = \frac{S_{c22}^I}{S_{b22}^I} \qquad G_{22}^J = \frac{S_{c22}^J}{S_{b22}^J}$$

$$G_{33}^I = \frac{S_{c33}^I}{S_{b33}^I} \qquad G_{33}^J = \frac{S_{c33}^J}{S_{b33}^J}$$

If a rotational release exists at a particular end (and direction) of an object, the corresponding value of G is set to 10.0. If all degrees of freedom for a particular joint are deleted, the G-values for all members connecting to that joint will be set to 1.0 for the end of the member connecting to that joint. Finally, if G^I and G^J are known for a particular direction, the column K-factors for the corresponding direction is calculated by solving the following relationship for α:

$$\frac{\alpha^2 G^I G^J - 36}{6(G^I + G^J)} = \frac{\alpha}{tan\alpha}$$

from which $K = \pi/\alpha$. This relationship is the mathematical formulation for the evaluation of K-factor for moment-resisting frames assuming sidesway to be uninhibited. For other structures, such as braced frame structures, the K-factor for all members are usually unity and should be set so by the user. The following are some important aspects associated with the column K-factor algorithm:

- An object that has a pin at the joint under consideration will not enter the stiffness summations calculated above. An object that has a pin at the far end from the joint under consideration will contribute only 50% of the calculated EI value. Also, beam members that have no column member at the far end from the joint under consideration, such as cantilevers, will not enter the stiffness summation.
- If there are no beams framing into a particular direction of a column member, the associated G-value will be infinity. If the G-values at both ends of a column for a particular direction are infinity, the K-factor corresponding to that direction is set equal to unity.
- If rotational releases exist at both ends of an object for a particular direction, the corresponding K-factor is set to unity.
- The automated K-factor calculation procedure can occasionally generate artificially high K-factor, specifically under circumstances involving skewed beams, fixed support conditions, and under other conditions where the program may have difficulty recognizing that the members are laterally supported and K-factors of unity are to be used.
- The automated K-factor calculation is turned off if the user sets the “Consider P-Delta Done?” to be “Yes” in the preferences. In this case, all the columns, beams, and braces are assigned K-factors of unity.
- All K-factor produced by the program can be overwritten by the user. These values should be reviewed and any unacceptable values should be replaced.
- The beams and braces are assigned K-factor of unity.

Determination K_1 Factors:

If G^I and G^J are known for a particular direction, the column K_1-factor for the corresponding direction is calculated by solving the following relationship for α:

$$\frac{G^I G^J}{4}\alpha^2 + \left(\frac{G^I + G^J}{2}\right)\left(1 - \frac{\alpha}{tan\alpha}\right) + \left(\frac{tan(\alpha/2)}{(\alpha/2)} - 1\right) = 0 \qquad \text{(non-sway)}$$

from which $K_1 = \pi/\alpha$. This relationship is the mathematical formulation for the evaluation of K_1-factor for moment-resisting frames assuming sidesway to be inhibited. The calculation of G^I and G^J follows the same procedure as that for K_2-factor which is already described in this section.

3 Design Process

This chapter provides a detailed description of the algorithms used by the programs in the design/check of structures in accordance with "Cold-Formed Steel Design Manual 2016." The implementation covers load combinations from Eurocode 1990:2002 [EN 1990:2002], which are described in the section "Design Loading Combinations" in this chapter.

3.1 Notations

The various notations used in this chapter are described herein.

A_{eff}	Effective cross-sectional area, mm^2
A_g	Gross cross-sectional area, mm^2
A_{net}	Net area of the cross-section, mm^2
A_s	Area of the stiffener, mm^2
A_w	Area of the web, mm^2
b_{eff}	Effective width of element, mm
b_{e1}, b_{e2}	Effective width of element, mm
b_p	Appropriate width of the element, mm
C_1, C_2, C_3	Bending coefficient dependent on moment gradient used in calculation of lateral-torsional buckling moment
D_o	Outside diameter of pipes, mm
D_i	Inside diameter of pipes, mm
$e_{N,y}, e_{N,z}$	Shift of the centroid of the effective area relative to the center of gravity of the gross cross-section on y-y and z-z axis, respectively

E	Modulus of elasticity, N/mm^2
f_{bv}	Shear strength considering buckling, N/mm^2
f_u	Ultimate strength, N/mm^2
f_{yb}	Base yield strength, N/mm^2
f_{ya}	Average yield strength, N/mm^2
G	Shear modulus, N/mm^2
i_y, i_z	Radius of gyration of full unreduced cross-section, mm
i_o	Polar radius of gyration of cross-section about shear center, mm
I_s	Effective moment of inertia of the stiffener, mm^4
I_y or I_{22}	Major moment of inertia, mm^4
I_z or I_{33}	Minor moment of inertia, mm^4
I_t	Torsion constant of the gross cross-section, mm^6
I_w	Warping constant of cross-section, mm^6
k_{yy}	Interaction factor
k_{zz}	Interaction factor
k_{yz}	Interaction factor
k_{zy}	Interaction factor
K_y, K_z	Effective length factor for buckling about major and minor axes, respectively
K_t	Effective length *K*-factor for lateral-torsional buckling
K	Spring stiffness of stiffener for distortional buckling, N/mm^2
k_σ	Buckling factor of plate element
k_d	Plate buckling coefficient for distortional buckling
k_v	Shear buckling coefficient
L_t	Lateral-torsional unbraced length of member, mm

L_x, L_y	Unbraced length of member for buckling about major and minor axes, respectively, mm
$M_{b,Rd}$	Nominal flexural strength, N-mm
$M_{c,Rd}$	Nominal flexural strength due to distortional buckling, N-mm
$M_{y,Ed}, M_{z,Ed}$	Design bending moment about y-y and z-z axis, respectively, N-mm
$\Delta M_{y,Ed}$, $\Delta M_{z,Ed}$	Moment due to the shift of the centroidal y-y and z-z axis, respectively
$M_{y,Rk}, M_{z,Rk}$	Characteristic value of resistance to bending moment about y-y and z-z axis, respectively, N-mm
$M_{f,Rd}$	Plastic moment capacity of the cross-section consisting of the effective area of flanges only, N-mm
$M_{wf,Rd}$	Plastic moment capacity of the cross-section consisting of the effective area of flanges and fully effective area of the web, N-mm
$N_{b,Rd}$	Nominal member (lateral-torsional0 buckling compressive strength, N
$N_{c,Rd}$	Nominal section (local and distortional) buckling compressive strength, N
$N_{cr,y}$, $N_{cr,z}$	Elastic critical force for buckling about major y-y and minor z-z axis, respectively
$N_{cr,T}$, $N_{cr,TF}$	Elastic critical force for torsional and torsional-flexural buckling, respectively
N_{Rd}	Design normal force, N
N_{Rk}	Characteristic value of resistance to compression
$N_{t,Rd}$	Nominal tensile strength, N
s_w	Slant height of the web
t	Thickness of the element of the section, mm
t_r	Reduced thickness for distortional buckling, mm
$V_{b,Rd}$	Nominal shear strength, N
W_{eff}	Effective section modulus of the effective cross-section, mm^3

W_{el}	Elastic section modulus of the gross cross-section, mm^3
W_{pl}	Plastic section modulus of the gross cross-section, mm^3
$W_{pl,f}$	the plastic section modulus of the section under bending consisting of the effective area of the flanges only, mm^3
$W_{pl,wf}$	the plastic section modulus of the section under bending consisting of the effective area of the flanges and the fully effective area of the web, mm^3
y_o	Distance from centroid to shear center in major y-axis direction, mm
z_o	Distance from centroid to shear center in minor z-axis direction, mm
γ_{M0}	Partial factor for resistance of the cross-section
γ_{M1}	Partial factor for resistance of the member to instability
γ_{M2}	Partial factor for resistance of the cross-section in tension to fracture
$\bar{\lambda}$	Nondimensional slenderness
$\bar{\lambda}_d$	Nondimensional slenderness for distortional buckling
$\bar{\lambda}_{LT}$	Nondimensional slenderness for lateral-torsional buckling
$\bar{\lambda}_p$	Plate slenderness of the equivalent plate
$\bar{\lambda}_{p,red}$	Reduced plate slenderness of the equivalent plate
$\bar{\lambda}_w$	Relative web slenderness factor of shear buckling
μ	Poisson's ratio of steel = 0.30
α	Imperfection factor corresponding to the appropriate buckling curve according to the type of cross-section
α_{LT}	Imperfection factor corresponding to the appropriate curve for lateral-torsional buckling
Φ	Value to determine the reduction factor χ
Φ_{LT}	Value to determine the reduction factor χ_{LT}
χ	Reduction factor for buckling
χ_{LT}	Reduction factor for lateral-torsional buckling

χ_d	Reduction factor for distortional buckling
ρ	Reduction factor in calculation of effective width of element
σ_1, σ_1	Stress at the opposite ends of the element computed on basis of effective design width, N/mm^2
ψ	Stress ratio used to determine the effective width

3.2 Design Loading Combinations

The design load combinations are combinations of load cases for which the structure is designed and checked. A default set of automated load combinations is available in the software, as described in this section. These default combinations can be modified or deleted. In addition, manually defined combinations can be added should the default combinations not cover all conditions required for the structure of interest.

The default load combinations considered by the software for the EC3-3 are defined in the following sections and handle dead (D), live (L), wind (W), and earthquake (E) loads. For other load types, combinations should be manually generated.

The following two sections describe the automated load combinations generated by the software for ultimate strength and serviceability, in accordance with EC0.

3.2.1 Ultimate Strength Combinations

The load combinations are defined based on EC0 equation 6.10 or the less favorable EC0 equations 6.10a and 6.10b [*NDP*].

$$\sum_{j\geq 1}\gamma_{G,j}G_{k,j}+\gamma_P P+\gamma_{Q,1}Q_{k,1}+\sum_{i>1}\gamma_{Q,i}\Psi_{0,i}Q_{k,i} \qquad \text{(EC0 Eq. 6.10)}$$

$$\sum_{j\geq 1}\gamma_{G,j}G_{k,j}+\gamma_P P+\gamma_{Q,1}\Psi_{Q,1}Q_{k,1}+\sum_{i>1}\gamma_{Q,i}\Psi_{0,i}Q_{k,i} \qquad \text{(EC0 Eq. 6.10a)}$$

$$\sum_{j\geq 1}\xi_j\gamma_{G,j}G_{k,j}+\gamma_P P+\gamma_{Q,1}Q_{k,1}+\sum_{i>1}\gamma_{Q,i}\Psi_{0,i}Q_{k,i} \qquad \text{(EC0 Eq. 6.10b)}$$

Load combinations including earthquake effects are generated based on:

$$\sum_{j\geq 1}G_{k,j}+P+A_{Ed}+\sum_{i>1}\Psi_{2,i}Q_{k,i} \qquad \text{(EC0 Eq. 6.12b)}$$

The following load combinations are considered if the option is set to generate the combinations based on EC0 equation 6.10.

$$\gamma_{Gj,sup}D \qquad \text{(EC0 Eq. 6.10)}$$

$$\gamma_{Gj,sup}D+\gamma_{Q,1}L \qquad \text{(EC0 Eq. 6.10)}$$

$$\gamma_{Gj,inf}D\pm\gamma_{Q,1}W \qquad \text{(EC0 Eq. 6.10)}$$
$$\gamma_{Gj,sup}D\pm\gamma_{Q,1}W \qquad \text{(EC0 Eq. 6.10)}$$

$$\gamma_{Gj,sup}D+\gamma_{Q,1}L\pm\gamma_{Q,1}\Psi_{0,i}W \qquad \text{(EC0 Eq. 6.10)}$$
$$\gamma_{Gj,sup}D\pm\gamma_{Q,1}W+\gamma_{Q,1}\Psi_{0,i}L \qquad \text{(EC0 Eq. 6.10)}$$

$D \pm 1.0E$ (EC0 Eq. 6.12b)
$D \pm 1.0E + \Psi_{2,i}L$ (EC0 Eq. 6.12b)

The following load combinations are considered if the option is set to generate the combinations based on the maximum of EC0 equations 6.10a and 6.10b.

$\gamma_{Gj,sup}D$ (EC0 Eq. 6.10a)
$\xi\gamma_{Gj,sup}D$ (EC0 Eq. 6.10b)

$\gamma_{Gj,sup}D + \gamma_{Q,1}\Psi_{0,1}L$ (EC0 Eq. 6.10a)
$\xi\gamma_{Gj,sup}D + \gamma_{Q,1}L$ (EC0 Eq. 6.10b)

$\gamma_{Gj,inf}D \pm \gamma_{Q,1}\Psi_{0,1}W$ (EC0 Eq. 6.10a)
$\gamma_{Gj,sup}D \pm \gamma_{Q,1}\Psi_{0,1}W$ (EC0 Eq. 6.10a)
$\gamma_{Gj,inf}D \pm \gamma_{Q,1}W$ (EC0 Eq. 6.10b)
$\xi\gamma_{Gj,sup}D \pm \gamma_{Q,1}W$ (EC0 Eq. 6.10b)

$\gamma_{Gj,sup}D + \gamma_{Q,1}\Psi_{0,1}L \pm \gamma_{Q,i}\Psi_{0,i}W$ (EC0 Eq. 6.10a)
$\gamma_{Gj,sup}D \pm \gamma_{Q,1}\Psi_{0,1}W + \gamma_{Q,i}\Psi_{0,i}L$ (EC0 Eq. 6.10a)

$\xi\gamma_{Gj,sup}D \pm \gamma_{Q,1}L + \gamma_{Q,i}\Psi_{0,i}W$ (EC0 Eq. 6.10b)
$\xi\gamma_{Gj,sup}D + \gamma_{Q,1}W \pm \gamma_{Q,i}\Psi_{0,i}L$ (EC0 Eq. 6.10b)

$D \pm 1.0E$ (EC0 Eq. 6.12b)
$D \pm 1.0E + \Psi_{2,i}L$ (EC0 Eq. 6.12b)

The variable values and factors used in the load combinations are defined as:

$\gamma_{Gj,sup}$ = 1.35 [*NDP*] (EC0 Table A1.2(B))
$\gamma_{Gj,inf}$ = 1.00 [*NDP*] (EC0 Table A1.2(B))
$\gamma_{G,1}$ = 1.5 [*NDP*] (EC0 Table A1.2(B))
$\Psi_{0,i} = \begin{cases} 0.7 & (live\ load, not\ storage) \\ 0.6 & (wind\ load) \end{cases}$ [*NDP*] (EC0 Table A1.1)
ξ = 0.85 [*NDP*] (EC0 Table A1.2(B))
$\Psi_{2,i}$ = 0.3 (assumed office/residential) [*NDP*] (EC0 Table A1.1)

3.2.2 Serviceability Combinations

The following characteristic load combinations are considered for the deflection checks:

D (EC0 Eq. 6.10b)
$D + L$ (EC0 Eq. 6.10b)

3.3 Calculation of Nominal Strengths

In calculation of nominal strengths, all cross-sections are treated as thin-walled section and classified as Class 4 slenderness. Additionally, the section properties such as neutral axis, moment of inertia and section modulus are measured and calculated to the centerline of the elements of the cross-section. The program performs several checks of width-to-thickness ratio of each of the elements of the cross-section for the applicability limits of the design provisions as described in Section 5.2 in the EC3-3 (Table 3-1). Warning messages are provided in the design report as any of these limits are not satisfied. It is worth noting that as the width-to-thickness ratios are not satisfied, the provisions and equations to calculate the nominal strengths provided by EC3-3 are technically no longer valid and the user should use the limits established by custom experimental testing.

The effects of rounded corners of the cross-section are also checked to determine whether they can be ignored. As $r \leq 5t$ and $r \leq 0.1b_p$ where r is the inside radius of the corner, t is the thickness, and b_p is the flat width of the flange measured to the points of intersection of the midlines of the elements, the effects of rounded corners are ignored, and the properties of the cross-section are calculated assuming that the section consists of plane elements with sharp corners. Otherwise, the section properties are determined using the formulations as described in Part I in Vol. 1 of the AISI 2016 (AISI, 2016.)

The nominal strengths in compression, tension, bending, and shear are computed for cold-formed steel members in accordance with the subsequent sections. The nominal compression strengths for all shapes of sections are calculated based on their geometric (or principal) axes of buckling. For C, Hat, I-Shape, T, Box, and Pipe sections, the principal axes coincide with their geometric axes. For Z and Single Angle sections, the lateral-torsional restraint condition is examined to determine the buckling axes and all computations are based on those.

The nominal flexural strengths for all shapes of sections are calculated based on their geometric (or principal) axes of bending. For the C-Section, Hat, I-Shape, T, Box, and Pipe sections, the principal axes coincide with their geometric axes. For Z section, nominal flexural strength is calculated based on geometric axes. For Single Angle sections, the lateral-torsional restraint condition is examined to determine the bending axes and all computations are based on those.

The nominal strengths in compression and flexure due to global buckling depend heavily on the unbraced length about the axis of bending, and for lateral-torsional and distortional buckling. These unbraced lengths can be specified as a fraction of the member length in the Overwrites. By specifying a ratio for an unbraced length type, the number of brace points will be internally determined, and the braced point locations will be arranged such that they are symmetric over the center line of the member, and the unbraced length at the ends of the member will always be less than or equal to the specified unbraced length $L_{end} \leq L$ (Figure 3-1). By default, the unbraced length ratios about major and minor axes of bending are determined by the analysis of structures, and the unbraced length ratios for lateral-torsional buckling is taken as unity.

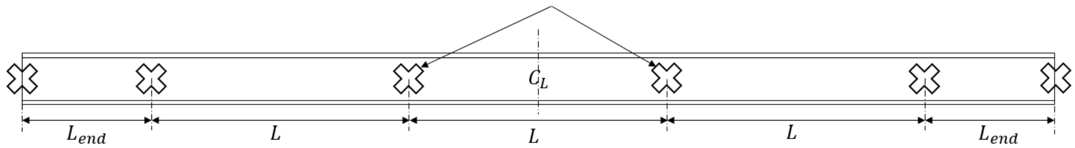


Figure 3-1: Braced point locations by Overwrites

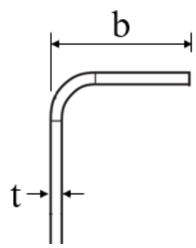
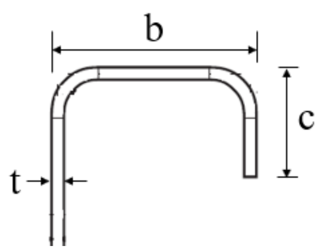
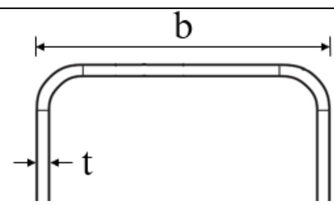
The lateral-torsional bracing condition can also be specified by point and/or uniform bracing in the Lateral Bracing option under the Design menu. However, the bracing condition assigned by this option will be overwritten by the user-specified unbraced length ratio for lateral-torsional buckling in the Overwrites.

Both nominal compression and flexural strengths in consideration of the limit state of local and distortional buckling are calculated using the Effective Width Method (Appendix B).

For all sections, the nominal shear strengths are calculated for directions aligned with the geometric axes.

The calculations of the nominal strengths are not performed for General and Section Designer sections.

Table 3-1 Width-to-thickness ratios for applicability of Effective Width Method

Element of cross-section	Limiting value
	$\frac{b}{t} \leq 50$
	$\frac{b}{t} \leq 60$ $\frac{c}{t} \leq 50$ $0.2 \leq \frac{c}{b} \leq 0.6$
	$\frac{b}{t} \leq 500$

3.3.1 Nominal Tensile Strength

This section applies to the members subject to axial tension. The design tensile strength is taken as the lower value obtained according to the limit states of yielding of gross section under tension and tensile rupture in the net section.

$$N_{t,Rd} = \frac{f_{ya}A_g}{\gamma_{M0}} \leq F_{n,Rd} = \frac{f_u A_{net}}{\gamma_{M2}} \qquad \text{(EC3-3 Eq. 6.1 \& 8.4)}$$

where

A_g is the gross area of the cross-section

A_{net} is the net area of the cross-section

f_u is the ultimate strength

γ_{M0} is the partial factor for resistance of the cross-section

γ_{M2} is the partial factor for resistance of the cross-section in tension to fracture

f_{ya} is the average yield strength and determined by:

$$f_{ya} = f_{yb} + \left(f_u - f_{yb}\right)\frac{knt^2}{A_g} \leq \frac{f_u + f_{yb}}{2} \qquad \text{(EC3-3 3.2.2 Eq. 3.1)}$$

$k = 7$ is the numerical coefficient for roll forming

n is the number of 90° bends in the cross-section. Fractions of 90° bends are counted as fractions of n

t is the thickness of the section

3.3.2 Nominal Compressive Strength

The design compression strength is taken to be the least compression capacity of the members in consideration of member buckling including flexural and torsional and flexural-torsional buckling, and local and distortional buckling of the cross-section. The limit states of torsional and flexural-torsional buckling are ignored for closed sections (Box and Pipe sections.) The summary of limit states considered for each type of section is displayed in Table 3-2 below.

Table 3-2 Limit States Considered for the Sections Subjected to Compression

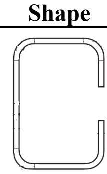
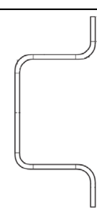
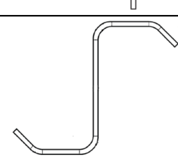
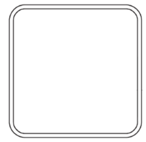
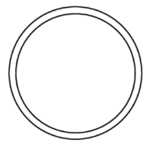
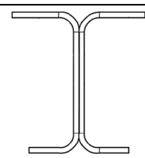
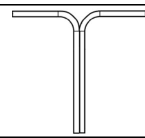
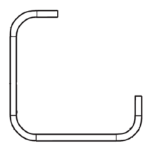
Section	Shape	Limit States
C		With lips: FB, TFB, LDB Without lips: FB, FTB, LB
Hat		With lips: FB, TFB, LDB Without lips: FB, FTB, LB
Z		With lips: FB, TFB, LDB Without lips: FB, FTB, LB

Table 3-2 Limit States Considered for the Sections Subjected to Compression

Box		FB, LB
Pipe		FB, LB
I-Wide Flange		FB, TFB, LB
Tee		FB, TFB, LB
Angle		With lips: FB, TFB, LDB Without lips: FB, FTB, LB

FB = flexural buckling
TFB = torsional-flexural buckling
LB = Local buckling
LDB = Local and distortional buckling

3.3.2.1 Member Buckling

The nominal member buckling compressive strength is the minimum value obtained according to the limit states of flexural buckling and torsional-flexural buckling:

$$N_{b,Rd} = \frac{\chi A_{eff} f_{yb}}{\gamma_{M1}} \quad \text{(EC3-1 Eq. 6.48)}$$

where

A_{eff} is the effective area of the cross-section calculated using the Effective Width Method (Appendix B) and the uniform compressive stress equal to f_{yb}

f_{yb} is the basic yield strength

γ_{M1} is the partial factor for resistance of the member to instability

In above equation, χ is the reduction factor for the relevant buckling curve calculated as follows:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \leq 1.0 \quad \text{(EC3-1 Eq. 6.49)}$$

where

$$\Phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right]$$

$$\bar{\lambda} = \sqrt{\frac{A_{eff} f_{yb}}{N_{cr}}}$$

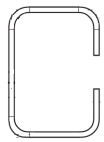
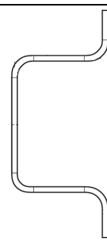
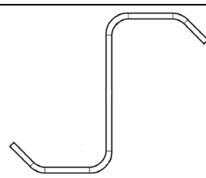
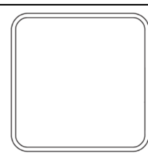
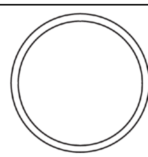
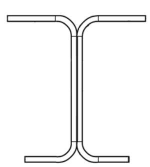
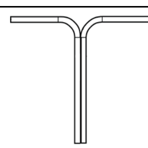
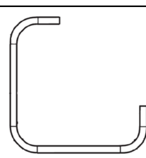
α is an imperfection factor corresponding to the appropriate buckling curve as shown in Table 3-3. And the buckling curve is determined according to the type of cross-section as illustrated in Table 3-4.

N_{cr} is the elastic critical force for the relevant buckling mode based on the gross cross-sectional properties and calculated as described in subsequent sections.

Table 3-3 Imperfection factors for buckling curves

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0.13	0.21	0.34	0.49	0.76

Table 3-4 Buckling curve for various types of cross-sections

Section	**Shape**	**Axis of buckling**	**Buckling curve**
C		With Lips: Any Without Lips: Any	b c
Hat		With Lips: Any Without Lips: Any	b c
Z		With Lips: Any Without Lips: Any	b c
Box		Any	c
Pipe		Any	c
I-Wide Flange		Equal flanges: y-y z-z	 a b
		Unequal flanges: Any	 c
Tee		Any	c
Angle		With Lips: Any Without Lips: Any	c c

3.3.2.1.1 Flexural Buckling

For flexural buckling, the elastic critical force N_{cr} is determined by:

$$N_{cr,y} = \frac{\pi^2 EI_y}{\left(K_y L_y\right)^2} \qquad \text{(AISI E2.1-1 \& Eq. 2.3.2.1.1-1)}$$

$$N_{cr,z} = \frac{\pi^2 EI_z}{(K_z L_z)^2} \qquad \text{(AISI E2.1-1 \& Eq. 2.3.2.1.1-1)}$$

For Z and Single-Angle sections, the program checks for buckling axes. As the frame member is attached to deck with through-fasteners or fully restrained against lateral-torsional buckling, the buckling axes are the geometric axes, and N_{cr} is calculated with K, L, and I based on the geometric axes. Otherwise, the buckling axes are the principal axes and and N_{cr} is calculated with K, L, and I based on the principal axes.

3.3.2.1.2 Torsional and Torsional-Flexural Buckling

For torsional and torsional-flexural buckling, the buckling curve used is the buckling curve about the z-z axis from Table 3-4.

3.3.2.1.2.1 Box and Pipe Sections

The limit states of torsional and torsional-flexural buckling are not considered for members with closed sections, such as Box and Pipe sections.

3.3.2.1.2.2 I-Shapes with Equal Flanges

$$N_{cr,T} = \frac{1}{i_0^2}\left[GI_t + \frac{\pi^2 EI_w}{L_T^2}\right] \qquad \text{(EC3-3 6.2.3(5) Eq. 6.33a)}$$

$$N_{cr,TF} = N_{cr,T} \qquad \text{(EC3-3 6.2.3(5) Eq. 6.34)}$$

3.3.2.1.2.3 C-Section with or without Lips, Hat-Section with or without Lips

$$N_{cr,TF} = \frac{N_{cr,y}}{2\beta}\left[1 + \frac{N_{cr,T}}{N_{cr,y}} - \sqrt{\left(1 - \frac{N_{cr,T}}{N_{cr,y}}\right)^2 + 4\left(\frac{y_0}{i_0}\right)^2 \frac{N_{cr,T}}{N_{cr,y}}}\right] \qquad \text{(EC3-3 6.2.3(7) Eq. 6.35)}$$

3.3.2.1.2.4 I-Shapes with Unequal Flanges, Tee Sections

$$N_{cr,TF} = \frac{N_{cr,z}}{2\beta}\left[1 + \frac{N_{cr,T}}{N_{cr,z}} - \sqrt{\left(1 - \frac{N_{cr,T}}{N_{cr,z}}\right)^2 + 4\left(\frac{z_0}{i_0}\right)^2 \frac{N_{cr,T}}{N_{cr,z}}}\right] \qquad \text{(EC3-3 6.2.3(7) Eq. 6.35)}$$

3.3.2.1.2.5 Z Sections

Z section is considered as point-symmetric section and $N_{cr,TF}$ is taken as the lesser of $N_{cr,T}$ as calculated in Section 3.3.2.1.2.2 and $N_{cr,z}$ as determined in Section 3.3.2.1.1 using minor principal axis of the section.

3.3.2.1.2.6 Single Angle Sections with Equal Legs

For angle section with equal legs, a check for buckling axes is performed. As the frame member is fully restrained against lateral-torsional buckling, the buckling axes are the geometric axes, the section is considered non-symmetric, and F_{cre} is calculated as the lowest root of the cubic equation:

$$\left(N_{cr,TF}-N_{cr,y}\right)\left(N_{cr,TF}-N_{cr,z}\right)\left(N_{cr,TF}-N_{cr,T}\right)-N_{cr,TF}^{2}\left(N_{cr,TF}-N_{cr,z}\right)\left(\frac{y_0}{r_0}\right)^2-N_{cr,TF}^{2}\left(N_{cr,TF}-N_{cr,y}\right)\left(\frac{z_0}{r_0}\right)^2=0$$

where y- and z-axes are the major and minor geometric axes, respectively.

Otherwise, the bending axes are principal, the section is singly-symmetric about the major principal axis, and $N_{cr,TF}$ is determined as follows:

$$N_{cr,TF}=\frac{N_{cr,y}}{2\beta}\left[1+\frac{N_{cr,T}}{N_{cr,y}}-\sqrt{\left(1-\frac{N_{cr,T}}{N_{cr,y}}\right)^2+4\left(\frac{y_0}{i_0}\right)^2\frac{N_{cr,T}}{N_{cr,y}}}\right] \quad \text{(EC3-3 Eq. 6.35)}$$

where y-axis is the major principal.

In the preceding equations,

- I_w is the warping constant of the gross cross-section, mm[6]
- I_t is the torsion constant of the gross cross-section, mm[6]
- x_0, y_0 are the coordinates of the shear center with respect to the centroid
- $i_0=\sqrt{i_y^2+i_z^2+y_0^2+z_0^2}$ = polar radius of gyration about the shear center (EC3-3 Eq. 6.33b)
- $\beta=1-\frac{y_o^2+z_o^2}{i_o^2}$ (EC3-3 6.2.3(7))
- I_y, I_z are the moment of inertia about the major and minor directions, respectively
- K_y, K_z are effective length factors in the major and minor directions, respectively
- K_T is the effective length factor for torsional buckling, and it is taken equal to K_{LTB} in this program; it can be overwritten
- L_y, L_z are effective lengths in the major and minor directions, respectively, mm
- L_T is the effective length for torsional buckling and it is taken equal the unbraced length for lateral-torsional buckling L_{LTB}. L_{LTB} can be overwritten.
- i_y, i_z are the radii of gyration about the major and minor directions, respectively

3.3.2.2 Local and Distortional Buckling

The nominal compressive strength of members in consideration of local and distortional buckling, $N_{c,Rd}$, is determined using the Effective Width Method as described in Appendix B:

$$N_{c,Rd}=\begin{cases}\dfrac{A_{eff}f_{yb}}{\gamma_{M0}} & A_{eff}<A_g\\ \sum_{i=1}^{n}\dfrac{A_i\left[f_{yb}+4\left(f_{ya}-f_{yb}\right)\left(1-\frac{\bar{\lambda}_{e,i}}{\bar{\lambda}_{e0,i}}\right)\right]}{\gamma_{M0}}\le\dfrac{A_g f_{ya}}{\gamma_{M0}} & A_{eff}=A_g\end{cases} \quad \text{(EC3-3 Eq. 6.2 \& 6.3)}$$

where

- A_{eff} is the effective area of the cross-section obtained using the Effective Width Method by assuming a uniform compressive stress equal to f_{yb}
- A_i is the area of element i of the cross-section
- f_{yb} is the basic yield strength
- f_{ya} is the average yield strength
- γ_{M0} is the partial factor for resistance of the cross-section

For plane elements, $\bar{\lambda}_{e,i}=\bar{\lambda}_p$ and $\bar{\lambda}_{e0,i}=0.673$

For stiffened elements, $\bar{\lambda}_{e,i}=\bar{\lambda}_d$ and $\bar{\lambda}_{e0,i}=0.65$. Both $\bar{\lambda}_p$ and $\bar{\lambda}_d$ are calculated as described in Appendix B

3.3.3 Nominal Flexure Strength

This section applies to members subject to simple bending about one geometric or principal axis. For the C, Hat, I-Shape, T, Box, and Pipe sections, the principal axes coincide with their geometric axes. For Z section, the nominal flexural strength is calculated based on geometric axes. For the Single Angle sections, the lateral-torsional restraint is examined to determine the bending axes according to the EN 1993-1-3 and all computations are based on that.

The nominal bending strength is the minimum value obtained considering the limit states of lateral-torsional buckling, and local and distortional buckling as appropriate for different structural shapes.

For members with box or pipe section, lateral-torsional buckling is not considered.

3.3.3.1 Lateral-Torsional Buckling

The nominal flexural strength, $M_{b,Rd}$, in consideration of lateral-torsional buckling is calculated as follows:

$$M_{b,Rd} = \chi_{LT} W_{eff,y} \frac{f_{yb}}{\gamma_{M1}} \quad \text{(EC3-1 Eq. 6.55)}$$

where

$W_{eff,y}$ = effective section modulus of the effective cross-section subjected only to bending moment with a maximum stress $\sigma_{com,Ed} = f_{yb}/\gamma_{M0}$

f_{yb} = the basic yield strength

γ_{M1} = partial factor for resistance of members to instability

χ_{LT} = the reduction factor for lateral-torsional buckling and calculated as follows:

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \leq 1.0 \quad \text{(EC3-1 Eq. 6.49)}$$

where

$\Phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0.2\right) + \bar{\lambda}_{LT}^2\right]$

$\bar{\lambda}_{LT} = \sqrt{\frac{W_{eff,y} f_{yb}}{M_{cr}}}$

$\alpha_{LT} = 0.34$ is an imperfection factor corresponding to the buckling curve b, which is used for all cross-sections.

M_{cr} is the elastic critical moment based on the gross cross-sectional properties and calculated as follows:

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{L_{cr}^2} \left\{ \left[\left(\frac{K_{LTB}}{k_w} \right) \frac{I_w}{I_z} + \frac{L_{cr}^2 G I_T}{\pi^2 E I_z} + \left(C_2 z_g - C_3 z_j \right)^2 \right]^{0.5} - \left(C_2 z_g - C_3 z_j \right) \right\} \quad \text{(EC3-1993 F.2)}$$

where I_z, I_w, and I_T are the minor axis moment of inertia, warping constant, and torsion constant, respectively; L_{cr} is the effective unbraced length for the lateral-torsional buckling mode and is defined as:

$$L_{cr} = K_{LTB} L_{LTB}$$

where K_{LTB} is the effective length factor for the lateral-torsional buckling mode, and L_{LTB} is the unbraced length for the lateral-torsional buckling mode. For more details on these two factors, please refer to Sections 5.5 and 5.6 in Chapter 5 of this manual.

k_w refers to end warping. It is defaulted to 1.0 and can be modified to have value ranging between

0.5 and 1.0 in the Overwrite, and z_g and z_j are calculated as:

$$z_g = z_a - z_s$$

$$z_j = z_s - \frac{0.5}{I_y}\int_A z(y^2 + z^2)dA$$

where z_a is the coordinate of the point of load application defaulted to be the coordinate on top of the section, and z_s is the coordinate of the shear center. Both z_a and z_s are measured with respect to the coordinate of the centroid of the section along the minor principle axis and can be overwritten in the Overwrites.

The value of z_j is calculated using the formula shown in Part I in Vol. 1 of the AISI 2016.

C_1, C_2, and C_3 are taken from Table F.1.1 and F.1.2 in EC3-1992 (Tables 3-5 and 3-6, respectively.) For the case of linear bending moment diagram as shown in Table 3-5, regression analyses have been performed to determine the relationships of C_1 and C_3 as functions of ψ and K_{LTB}, and C_2 is taken as zero:

$$C_1 = \begin{cases} 0.9(-1.338K_{LTB}^2 + 1.140K_{LTB} + 3.210) & -1.0 \le \psi < -0.5 \\ (0.176\psi^2 - 0.461\psi + 0.625)(-1.338K_{LTB}^2 + 1.140K_{LTB} + 3.210) & -0.5 \le \psi \le 0.75 \\ C - (C-1)\dfrac{\psi - 0.75}{0.25} & 0.75 < \psi \le 1.0 \end{cases}$$

where $C = 0.378(-1.338K_{LTB}^2 + 1.140K_{LTB} + 3.210)$

$$C_3 = \begin{cases} (2.01\psi^3 - 3.647\psi^2 + 2.2\psi + 7.783)(0.412K_{LTB}^2 - 0.929K_{LTB} + 0.639) & -1.0 \le \psi \le 0.75 \\ C - (C-1)\dfrac{\psi - 0.75}{B - 0.75} & 0.75 < \psi \le 1.0 \end{cases}$$

where $C = 8.230(0.412K_{LTB}^2 - 0.929K_{LTB} + 0.639)$ and $B = -0.443K_{LTB}^2 + 0.377K_{LTB} + 1.066$

$\psi = (M_1/M_2)$

M_1 and M_2 are the smaller and larger bending moment, respectively, at the ends of the segment between lateral restraints in the plane of bending.

For the cases in Table 3-6, C_1, C_2, and C_3 are taken as shown in the table for K_{LTB} having value of 0.5 and 1.0, and interpolated for other value of K_{LTB} within the range of 0.5 and 1.0. In Table 3-6 for the cases with simply-supported conditions, the values of C_1, C_2, and C_3 are exactly taken from Table F.1.2 of the EC3-1992. The other two cases with fixed-end support conditions, C_1, C_2, and C_3 are taken conservatively equal to those in the case with simply-supported conditions and similar loading.

For any other cases, C_2 and $C_3$are taken as zero, and C_1 is calculated as follows:

$$C_1 = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \le 0.9(-1.338K_{LTB}^2 + 1.140K_{LTB} + 3.210)$$

where,

M_{max} = absolute value of maximum moment in unbraced segment, N-mm.

M_A = absolute value of moment at quarter point of the unbraced segment, N-mm.

M_B = absolute value of moment at the middle of the unbraced segment, N-mm.

M_C = absolute value of moment at three-quarter point of the unbraced segment, N-mm.

For the purpose of determining C_1, C_2, and C_3, the program limits the value of K_{LTB} to be within the range between 0.5 and 1.0. If K_{LTB} is input to have the value less than 0.5 in the Overwrites, it will be taken to be 0.5. Likewise, if it is input to have the value greater than 1.0, it will be taken to be 1.0. If it is program determined, it will be defaulted to be 1.0. This condition of K_{LTB} is not imposed for any other calculations elsewhere.

C_1 should be taken as 1.0 for cantilevers. However, the program is unable to detect whether the member is a cantilever. **The user should overwrite C_1 for cantilevers.** The program also defaults C_1 to 1.0 if the minor unbraced length, L_z, is redefined to be more than the length of the member by the user or the program, i.e., if the unbraced length is longer than the member length. The Overwrites can be used to change the value of C_1, C_2, and C_3 for any member.

Table 3-5 Values of C_1, C_2, and C_3 for end moment loading. (Source: EC3-1992)

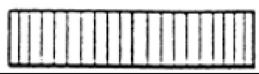
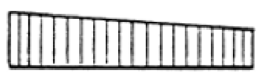
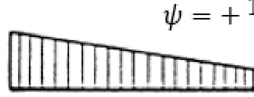
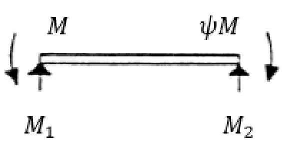
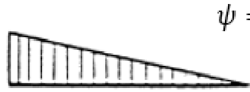
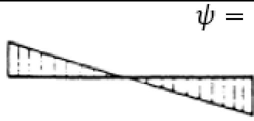
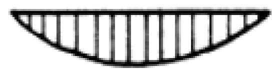
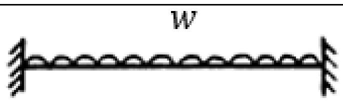
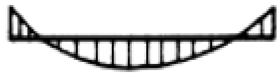
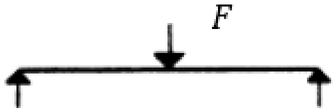
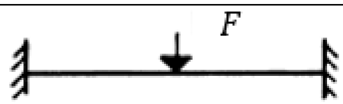
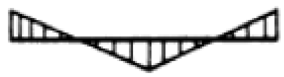
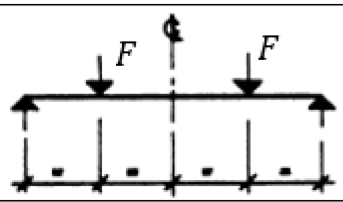
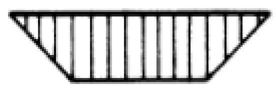
Loading and support conditions	Bending moment diagram	Value of K_{LTB}	Values of factors C_1	C_2	C_3
	$\psi = +1$ 	1.0 0.7 0.5	1.000 1.000 1.000	----	1.000 1.113 1.144
	$\psi = +3/4$ 	1.0 0.7 0.5	1.141 1.270 1.305	----	0.998 1.565 2.283
	$\psi = +1/2$ 	1.0 0.7 0.5	1.323 1.473 1.514	----	0.992 1.556 2.271
	$\psi = +1/4$ 	1.0 0.7 0.5	1.563 1.739 1.788	----	0.977 1.531 2.235
	$\psi = 0$ 	1.0 0.7 0.5	1.879 2.092 2.150	----	0.939 1.473 2.150
	$\psi = -1/4$ 	1.0 0.7 0.5	2.281 2.538 2.609	----	0.855 1.340 1.957
	$\psi = -1/2$ 	1.0 0.7 0.5	2.704 3.009 3.093	----	0.676 1.059 1.546
	$\psi = -3/4$ 	1.0 0.7 0.5	2.927 3.009 3.093	----	0.366 0.575 0.837
	$\psi = -1$ 	1.0 0.7 0.5	2.752 3.063 3.149	----	0.000 0.000 0.000

Table 3-6 Values of C_1, C_2, and C_3 for transverse loading cases. (Source: EC3-1992)

Loading and support conditions	Bending moment diagram	Value of K_{LTB}	Values of factors		
			C_1	C_2	C_3
		1.0	1.132	0.459	0.525
		0.5	0.972	0.304	0.980
		1.0	1.365	0.553	1.730
		0.5	1.070	0.432	3.050
		1.0 0.5	1.046 1.010	0.430 0.410	1.120 1.890

The elastic critical moment, M_{cr}, of C, Hat, Z, I, T, and Angle sections is calculated as described previously. The equation to calculate M_{cr} is only applicable to cross-sections symmetrical about the minor axis of bending, which does not apply to Z and Angle sections. However, it is still calculated using this equation given the available information. For Angle section, it is assumed that the shear center coordinate z_s is the projection on the axis along the direction of load application z_a in calculation of z_g.

For Pipe and Box sections, the reduction factor for buckling χ_{LT} is taken as unity in the calculation of flexural strength due to lateral-torsional buckling.

For Non-prismatic element with all sections along the element having similar shape, all properties required for calculation of M_{cr} is linearly interpolated from those properties of the two end sections of the segment which the design section falls in. This procedure is also applied to Non-prismatic element with sections having different shapes and may produce unexpected design results.

The value of M_{cr} can be overwritten in the Overwrites.

3.3.3.2 Local and Distortional Buckling

The Effective Width Method described in Section 5 of the EC3-3 is adopted to calculate the flexural strength for the limit state of local and distortional buckling. The stress condition and capacity of each element of the section are determined as described in Appendix B Effective Width of Elements and Tables B-1 and B-2 of this manual. And the nominal flexural strength is computed as:

$$M_{c,Rd} = \begin{cases} \frac{W_{eff} f_{yb}}{\gamma_{M0}}, & W_{eff} < W_{el} \\ \frac{f_{yb}\left[W_{el}+4\left(W_{pl}-W_{el}\right)\left(1-\frac{\bar{\lambda}_{emax}}{\bar{\lambda}_{e0}}\right)\right]}{\gamma_{M0}} \le \frac{W_{pl} f_{ya}}{\gamma_{M0}}, & W_{eff} = W_{el} \end{cases}$$ (EC3-3 Eq. 6.4 & 6.5)

where

$W_{eff,y}$ = effective section modulus of the effective cross-section subjected only to bending moment with a maximum stress $\sigma_{com,Ed} = f_{yb}/\gamma_{M0}$

W_{el} = elastic section modulus of the gross cross-section

W_{pl} = plastic section modulus of the gross cross-section

f_{yb} = the basic yield strength

f_{ya} = the average yield strength

γ_{M0} = the partial factor for resistance of the cross-section

$\bar{\lambda}_{emax}/\bar{\lambda}_{e0}$ is taken as the greatest ratio of $\bar{\lambda}_e/\bar{\lambda}_{e0}$ for all elements of the cross-section.

For doubly supported elements, $\bar{\lambda}_e = \bar{\lambda}_p$ and $\bar{\lambda}_{e0} = 0.5 + \sqrt{0.25 - 0.055(3+\psi)}$

For outstand elements, $\bar{\lambda}_e = \bar{\lambda}_p$ and $\bar{\lambda}_{e0} = 0.673$. Both $\bar{\lambda}_p$ and ψ are calculated as described in Appendix B

For Z section, because the nominal flexural strength due to lateral-torsional buckling is only considered about the geometric axes, the effective section modulus is also calculated based on bending about the geometric axes even though the section is point symmetric.

For angle section, when bending axes as determined in Section 3.4.4.1.6 are principal axes, the calculation of effective width of elements and effective section modulus as described in Appendix B is also applicable. The corresponding element stresses and effective width are calculated accounting for the angle between principal and geometric axes.

3.3.4 Nominal Shear Strength

The nominal shear strengths are calculated for shears along the geometric axes for all sections. In calculating nominal strength for shear, $V_{b,Rd}$, it is assumed that there are no intermediate stiffeners used to enhance shear strength of a section. The program calculates shear strengths considering the limit state of shear buckling.

The nominal shear strength $V_{b,Rd}$ is computed as:

$$V_{b,Rd} = \frac{\frac{h_w}{\sin\phi} t f_{bv}}{\gamma_{M0}} = \frac{h_w t f_{bv}}{\gamma_{M0}}$$ (EC3-3 Eq. 6.8)

where

h_w = height of the web measured between points of intersection of flange and web midlines.

t = thickness of the web

ϕ = the slope of the web relative to the flanges. For all sections available for EC3-3 coldformed steel design, $\phi = 90°$

f_{bv} = the shear strength considering buckling and computed as follows:

$$f_{bv} = \begin{cases} 0.58 f_{yb} & \bar{\lambda}_w \le 0.83 \\ 0.48 f_{yb}/\bar{\lambda}_w & 0.83 < \bar{\lambda}_w < 1.40 \\ 0.67 f_{yb}/\bar{\lambda}_w^2 & \bar{\lambda}_w \ge 1.40 \end{cases} \quad \text{(EC3-3 Table 6.1)}$$

where

$$\bar{\lambda}_w = 0.346 \frac{s_w}{t} \sqrt{\frac{f_{yb}}{E}} \quad \text{(EC3-3 Eq. 6.10a)}$$

s_w = slant height of the web. If the effects of rounded corners can be neglected, it is equal to h_w as defined above. Otherwise, it is the height measured between the midpoints of the adjacent rounded corner elements as shown in Figure 3-2.

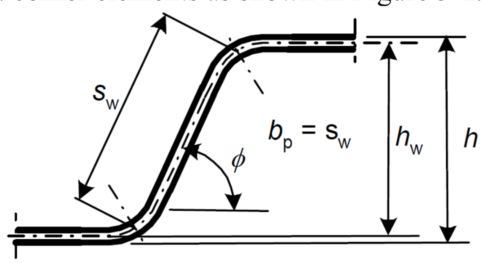


Figure 3-2: Notional height of the web

In minor direction of shear, the notional width of the flanges is used in place of the height of the web for calculation of nominal shear strength. For I-section with different flanges, the width of the larger flange is used to determine the slenderness $\bar{\lambda}_w$ and the shear stress f_{bv}, which is applied with total area of both top and bottom flanges to calculate $V_{b,Rd}$.

For pipe section, the EC3-3 is silent on shear capacity. Therefore, the provisions by AISC 360-16 are applied to calculate shear capacity for pipe section as follows:

$$V_n = F_{cr} A_g / 2 \quad \text{(AISC G5-1)}$$

where

$$F_{cr} = max \left\{ \frac{1.60E}{\sqrt{\frac{L_v}{D}} \left(\frac{D}{t}\right)^{\frac{5}{4}}}, \frac{0.78E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}} \right\} \le 0.6 F_y \quad \text{(AISC G5-2)}$$

A_g = gross cross-sectional area of the pipe section

D = outside diameter of the pipe section

t = thickness of the pipe section

L_v = length of the member. The AISC 360-16 defines L_v as the distance from maximum to zero shear force, but the program uses the length of the member resulting in more conservative design for shear.

3.4 Design of Members for Combined Forces

Previous sections of this design manual address members subject to only one type of force, namely axial tension, axial compression, flexure or shear. This section addresses the design of members subject to a combination of two or more of the individual forces.

In the calculation of the demand/capacity (D/C) ratios, first, for each station along the length of the member, the actual member force/moment components are calculated for each design combination. Then, the corresponding capacities are calculated. The D/C ratios are calculated at each station for each member under the influence of each of the design combinations. The controlling D/C ratio is then obtained, along with the associated station and design combination. A D/C ratio greater than the D/C ratio limit (whose default value is 1.0) indicates exceeding a limit state. At each station for each load combination, the governing D/C ratio is taken as the largest calculated from the subsections below.

3.4.1 Section Subjected to Tension and Bending

The D/C ratio for section subjected to tension and bending is

$$\frac{N_{Ed}}{N_{t,Rd}} + \frac{M_{y,Ed}}{M_{cy.Rd,ten}} + \frac{M_{z,Ed}}{M_{cz.Rd,ten}} \quad \text{(EC3-3 Eq. 6.23)}$$

$$\frac{M_{y,Ed}}{M_{cy.Rd,com}} + \frac{M_{z,Ed}}{M_{cz.Rd,com}} - \frac{N_{Ed}}{N_{t,Rd}} \quad \text{(EC3-3 Eq. 6.24)}$$

where

$M_{y,Ed}$, $M_{z,Ed}$	applied moments about major and minor axis, respectively
N_{Ed}	applied tension
$M_{cy.Rd,ten}$, $M_{cz.Rd,ten}$	moment capacity about major and minor axis, respectively, of the cross-section for maximum tensile stress
$M_{cy.Rd,com}$, $M_{cz.Rd,com}$	moment capacity about major and minor axis, respectively, of the cross-section for maximum compressive stress
$N_{t,Rd}$	tension capacity of the cross-section

3.4.2 Section Subjected to Compression and Bending

The D/C ratio for section subjected to compression and bending is

$$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{y,Ed} + \Delta M_{y,Ed}}{M_{cy.Rd,com}} + \frac{M_{z,Ed} + \Delta M_{z,Ed}}{M_{cz.Rd,com}} \quad \text{(EC3-3 Eq. 6.25)}$$

$$\frac{M_{y,Ed} + \Delta M_{y,Ed}}{M_{cy.Rd,ten}} + \frac{M_{z,Ed} + \Delta M_{z,Ed}}{M_{cz.Rd,ten}} - \frac{N_{Ed}}{N_{c,Rd}} \quad \text{(EC3-3 Eq. 6.26)}$$

where

$M_{y,Ed}$, $M_{z,Ed}$	applied moments about major and minor axis, respectively
$\Delta M_{y,Ed}$, $\Delta M_{z,Ed}$	additional moments about major and minor axis, respectively, due to the shifts of centroidal axes and taken as:

$\Delta M_{y,Ed} = N_{Ed} e_{Ny}$ and $\Delta M_{z,Ed} = N_{Ed} e_{Nz}$

e_{Ny}, e_{Nz}	the shifts of major y-y and minor z-z centroidal axis of the effective cross-section with respect to the gross cross-section
$M_{cy.Rd,ten}$, $M_{cz.Rd,ten}$	moment capacity about major and minor axis, respectively, of the cross-section for maximum tensile stress
$M_{cy.Rd,com}$, $M_{cz.Rd,com}$	moment capacity about major and minor axis, respectively, of the cross-section for maximum compressive stress
$N_{c,Rd}$	compression capacity of the cross-section

3.4.3 Section Subjected to Shear Force, Axial Force, and Bending Moment

The D/C ratio for section subjected to an axial force N_{Ed}, bending moments $M_{y,Ed}$ and $M_{z,Ed}$ about major and minor axes, respectively, and shear force $V_{y,Ed} > 0.5V_{wy,Ed}$ and $V_{z,Ed} > 0.5V_{wz,Ed}$ is:

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y.Rd}} + \left(1 - \frac{M_{f,Rd}}{M_{wf.Rd}}\right)\left(\frac{2V_{y,Ed}}{V_{wy.Rd}} - 1\right)^2 + \frac{M_{z,Ed}}{M_{z.Rd}} + \left(\frac{2V_{z,Ed}}{V_{wz.Rd}} - 1\right)^2$$

where

N_{Rd}	axial capacity of the cross-section
$M_{y,Rd}, M_{z,Rd}$	moment capacity about major and minor axis, respectively, of the cross-section
$V_{wy,Rd}, V_{wz,Ed}$	shear capacity along major and minor axis, respectively
$M_{f,Rd} = \frac{W_{pl,f} f_{yb}}{\gamma_{M0}}$	plastic moment capacity of the cross-section consisting of the effective area of flanges only
$W_{pl,f}$	the plastic section modulus of the section under bending consisting of the effective area of the flanges only (Figure 3-3 Right.) The plastic neutral axis is taken as that of the section having effective area of the flanges and fully effective area of the web.
$M_{wf,Rd} = \frac{W_{pl,wf} f_{yb}}{\gamma_{M0}}$	plastic moment capacity of the cross-section consisting of the effective area of the flanges and the fully effective area of the web
$W_{pl,wf}$	the plastic section modulus of the section under bending consisting of the effective area of the flanges and the fully effective area of the web (Figure 3-3 Left.)

The above equation has the form of equation 6.27 in the EC3-3 but is slightly different as it includes the ratio contribution from the minor-axis loading and capacity of both moment and shear.

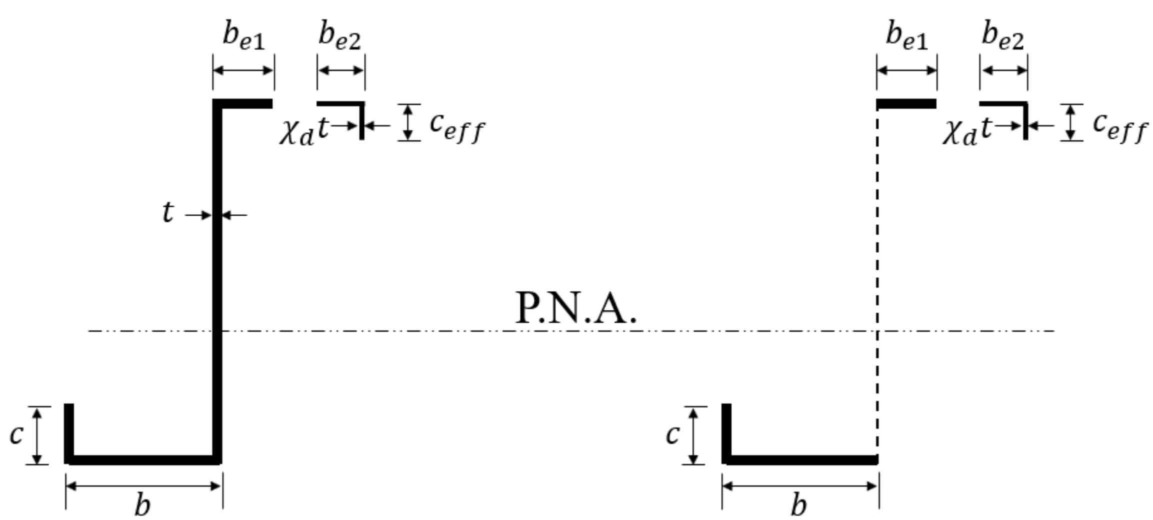


Figure 3-3: Effective sections to compute plastic section moduli – Left: $W_{pl,wf}$ *– Right:* $W_{pl,f}$

3.4.4 Members Subjected to Bending and Axial Compression

The D/C ratio for member subjected to bending and compression is:

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \quad \text{(EC3-1 Eq. 6.61)}$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\frac{\chi_{LT} M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \quad \text{(EC3-1 Eq. 6.62)}$$

where

$N_{Ed}, M_{y,Ed}, M_{z,Ed}$	applied compression and moments about major and minor axis, respectively
$\Delta M_{y,Ed}, \Delta M_{z,Ed}$	additional moments about major and minor axis, respectively, due to the shifts of centroidal axes and taken as:

$\Delta M_{y,Ed} = N_{Ed} e_{Ny}$ and $\Delta M_{z,Ed} = N_{Ed} e_{Nz}$

e_{Ny}, e_{Nz}	the shifts of major y-y and minor z-z centroidal axis of the effective cross-section with respect to the gross cross-section
$N_{Rk} = A_{eff} f_{yb}$	characteristic value of resistance to compression
$M_{y.Rk} = W_{eff,y} f_{yb}$	characteristic value of resistance to bending moment about y-y axis
$M_{z.Rk} = W_{eff,z} f_{yb}$	characteristic value of resistance to bending moment about z-z axis
χ_y, χ_z	the reduction factors due to flexural buckling from Section 3.3.2.1
χ_{LT}	the reduction factors due to lateral-torsional buckling from Section 3.3.3.1
$k_{yy}, k_{yz}, k_{zy}, k_{zz}$	interaction factors and calculated according to Annex A or Annex B of the EC3-1 for class 4 cross-sections

The D/C ratio for member subjected to bending and compression is also computed by an alternative formula:

$$\left(\frac{N_{Ed}}{N_{b,Rd}}\right)^{0.8} + \left(\frac{M_{y,Ed}+\Delta M_{y,Ed}}{M_{by,Rd}}\right)^{0.8} + \left(\frac{M_{z,Ed}+\Delta M_{z,Ed}}{M_{bz,Rd}}\right)^{0.8}$$

The above equation has the form of equation 6.36 in the EC3-3 but is slightly different as it includes the ratio contribution from the minor-axis loading and capacity of moment.

APPENDICES

Appendix A
P-Delta Effects

Modern design provisions are based on the principle that the member forces are calculated by a second-order elastic analysis, where the equilibrium is satisfied on the deformed geometry of the structure. The effects of the loads acting on the deformed geometry of the structure are known as the second-order or the P-Delta effects.

The P-Delta effects come from two sources: global lateral translation of the frame and the local deformation of members within the frame.

Consider the frame object shown in Figure A-1, which is extracted from a story level of a larger structure. The overall global translation of this frame object is indicated by Δ. The local deformation of the member is shown as δ. The total second order P-Delta effects on this frame object are those caused by both Δ and δ.

The program has an option to consider P-Delta effects in the analysis. When you consider P-Delta effects in the analysis, the program does a good job of capturing the effect due to the Δ deformation (P-Δ effect) shown in Figure B-1, but it does not typically capture the effect of the δ deformation (P-δ effect), unless, in the model, the frame object is broken into multiple elements over its length.

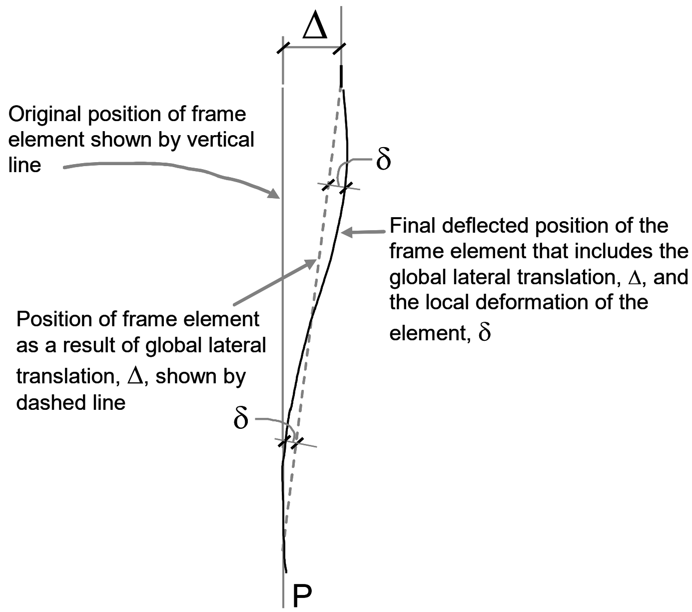


Figure A-1 ***P-Δ and P-δ effects***

In design codes, required strengths are usually required to be determined using a second-order analysis that considers both P-Δ and P-δ effects. Approximate second-order analysis procedures based on amplification of responses from first-order analysis for calculating the required flexural and axial strengths are common in current design codes but are not specified in the Cold-formed steel Design Manual 2015. Therefore, second-order analysis considering both P-Δ and P-δ effects should be run (the program has the capability of performing this rigorous second-order analysis) before the cold-formed steel frame design is carried out. In this case, the required strengths are determined directly from the analysis results.

To properly model the P-δ effect in a finite element analysis, each element, especially column elements, must be broken into multiple finite elements. Although a single element per member can capture the P-δ effect to some extent, the program considers that inadequate. For practical reasons, the software internally divides the column elements into two members. The user must provide additional subdivisions where a column is expected to have multiple inflection points.

In general, cold-formed steel frame design requires consideration of P-Delta effects in the analysis before the check/design is performed. Although two elements per line object are generally adequate to model the P-Δ effect, it is recommended to use more than two elements per line object for the cases where both P-Δ and P-δ effects are to be considered for a member having multiple points of inflection. However, explicit manual breaking of the member into elements has other consequences related to member end moments and unbraced segment end moment. It is recommended that the members be broken internally by the program. In this way, the member is recognized as one unit, ends of the members are identified properly, and P-Δ and P-δ effects are modeled better.

Appendix B
Effective Width of Elements

The nominal strengths for compression and flexure of a section are dependent on the slenderness and stress capacity of individual elements of the section. In consideration of local buckling, each element of the section is categorized into loading conditions of Uniform Compression (UC) or Stress Gradient (SG) and stiffening conditions of Stiffened (S) for internal element, Partially Stiffened (PS) with a simple lip edge stiffener, or Unstiffened (US) for outstand element, and its slenderness and stress capacity are calculated correspondingly.

Under axial compression load, all elements of the section are subjected to Uniform Compression (UC) stress condition. When the section is subjected to flexural bending about major axis, for example, the flanges of C section is assumed to be in Uniform Compression while the web is in Stress Gradient. If bending is about minor axis, the web is in Uniform Compression and flanges are in Stress Gradient.

Tables B-1 summarizes the loading and stiffening conditions of each element of the sections in the program and its stress capacity. Both the width and thickness of each element are also used to compute its area, which is utilized in calculation of the total compression and flexure capacity due to the condition of local buckling. Based on the loading and stiffening conditions, the buckling factor is calculated as summarized in Tables B-2 and B-3.

The notional flat widths of the plane elements used in calculation of the effective width are measured to the points of intersection of their centerlines. The effects of rounded corners of the cross-section are checked to determine whether they can be ignored as described previously in Section 3.3. In case they cannot be ignored, the effective section properties are further reduced to account for the influence of rounded corners as illustrated at the end of this appendix.

Table B-1 Effective Width of Elements for Stress Capacity – Members Subjected to Compression or Flexure

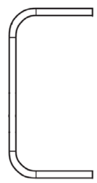
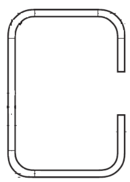
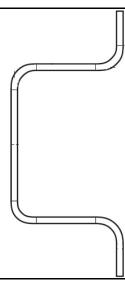
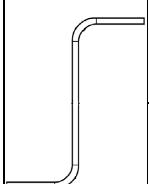
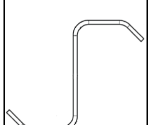
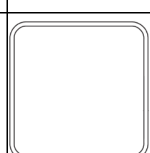
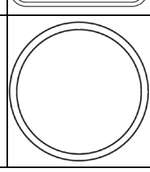
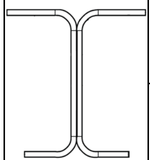
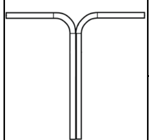
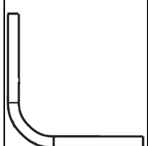
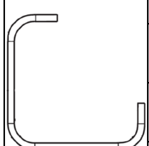
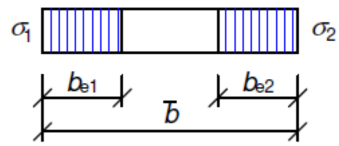
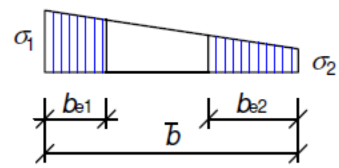
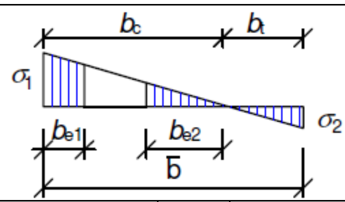
Section Type	Example	Element	Compression		Flexure			
					Major		Minor	
			Stiffening	Stress	Stiffening	Stress	Stiffening	Stress
C/Hat Section without Lips		Flange	US	UC	US	UC	US	SG
		Web	S	UC	S	SG	S	UC
C-Section with Lips		Flange	PS	UC	PS	UC	S	SG
		Lip	US	UC	US	SG	US	UC
		Web	S	UC	S	SG	S	UC
Hat Section with Lips		Flange	PS	UC	PS	UC	S	SG
		Lip	US	UC	US	SG	US	UC
		Web	S	UC	S	SG	S	UC
Z Section without Lips		Flange	US	UC	US	UC	US	SG
		Web	S	UC	S	SG	S	UC
Z Section with Lips		Flange	PS	UC	PS	UC	S	SG
		Lip	US	UC	US	SG	US	S
		Web	S	UC	S	SG	S	UC
Box		Flange	S	UC	S	UC	S	SG
		Web	S	UC	S	SG	S	UC
Pipe		———	———	———	———	———	———	———

Table B-1 Effective Width of Elements for Stress Capacity – Members Subjected to Compression or Flexure

I Shape		Flange	US	UC	US	UC	US	SG
		Web	S	UC	S	SG	S	UC
T Shape/Double Angle		Flange	US	UC	US	UC	US	SG
		Web	US	UC	US	SG	US	UC
Angle without Lips		Flange	US	UC	US	UC	US	SG
		Web	US	UC	US	SG	US	UC
Angle with Lips		Flange	PS	UC	PS	UC	S	SG
		Flange Lip	US	UC	US	SG	US	UC
		Web	S	UC	S	SG	PS	UC
		Web Lip	US	UC	US	UC	US	SG

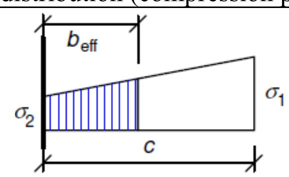
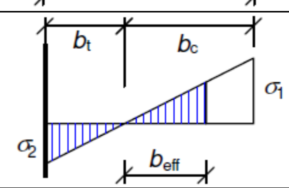
S: Stiffened US: Unstiffened PS: Partially stiffened with a simple lip edge stiffener
UC: Uniform Compression SG: Stress Gradient
γ = lip angle measured from the horizontal line parallel to the flange.

Table B-2 Internal (stiffened) elements

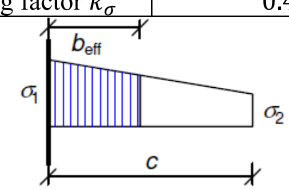
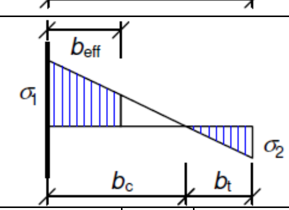
Stress distribution (compression positive)	Effective width b_{eff}
	$\underline{\psi = 1:}$ $b_{eff} = \rho\bar{b}$ $b_{e1} = 0.5b_{eff}$ $b_{e2} = 0.5b_{eff}$
	$\underline{1 > \psi \geq 0:}$ $b_{eff} = \rho\bar{b}$ $b_{e1} = \frac{2}{5-\psi}b_{eff}$ $b_{e2} = b_{eff} - b_{e1}$
	$\underline{\psi < 0:}$ $b_{eff} = \rho b_c = \rho\bar{b}/(1-\psi)$ $b_{e1} = 0.4b_{eff}$ $b_{e2} = 0.6b_{eff}$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi \geq 0$	$0 > \psi \geq -1$	$-1 > \psi \geq -3$	$-3 > \psi$
Buckling factor k_σ	4.0	$8.2/(1.05+\psi)$	$7.81 - 6.29\psi + 9.78\psi^2$	$5.98(1-\psi)^2$	95.68

Table B-3 Outstand (unstiffened) elements

Stress distribution (compression positive)	Effective width b_{eff}
	$\underline{1 > \psi \geq 0:}$ $b_{eff} = \rho c$
	$\underline{\psi < 0:}$ $b_{eff} = \rho b_c = \rho\bar{b}/(1-\psi)$

$\psi = \sigma_2/\sigma_1$	1	$1 \geq \psi \geq -3$	$-3 > \psi$
Buckling factor k_σ	0.43	$0.57 - 0.21\psi + 0.07\psi^2$	1.83

Stress distribution (compression positive)	Effective width b_{eff}
	$\underline{1 > \psi \geq 0:}$ $b_{eff} = \rho c$
	$\underline{\psi < 0:}$ $b_{eff} = \rho b_c = \rho\bar{b}/(1-\psi)$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi \geq 0$	$0 > \psi \geq -1$	$-1 > \psi$
Buckling factor k_σ	0.43	$0.578/(\psi + 0.34)$	$1.7 - 5\psi + 17.1\psi^2$	23.8

In general case, the reduction factor, ρ, is determined as follows:

For internal (stiffened) elements:

$$\rho = \begin{cases} 1.0 & \bar{\lambda}_p \leq 0.673 \\ \begin{cases} \frac{\bar{\lambda}_p - 0.055(3+\psi)}{\bar{\lambda}_p^2} \leq 1.0 & \psi \geq -3 \\ \frac{1}{\bar{\lambda}_p} \leq 1.0 & \psi < -3 \end{cases} & \bar{\lambda}_p > 0.673 \end{cases}$$ (EC3-5 4.4(2) Eq. 4.2)

For outstand (unstiffened) elements:

$$\rho = \begin{cases} 1.0 & \bar{\lambda}_p \leq 0.748 \\ \frac{\bar{\lambda}_p - 0.188}{\bar{\lambda}_p^2} \leq 1.0 & \bar{\lambda}_p > 0.738 \end{cases}$$ (EC3-5 4.4(2) Eq. 4.3)

where

$\bar{\lambda}_p = \frac{b_p/t}{28.4\varepsilon\sqrt{k_\sigma}}$

ψ = stress ratio determined as shown in Tables B-2 and B-3

b_p = appropriate width of the element

k_σ = buckling factor determined as shown in Tables B-2 and B-3

t = thickness of the element

$\varepsilon = \sqrt{\frac{235}{f_{yb}[N/mm^2]}}$

If the maximum stress in the element $\sigma_{com,Ed} < f_{yb}/\gamma_{M0}$, the reduced slenderness $\bar{\lambda}_{p,red}$ is used in place of $\bar{\lambda}_p$:

$$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{\frac{\sigma_{com,Ed}}{f_{yb}/\gamma_{M0}}}$$ (EC3-5 4.4(2) Eq. 4.3)

For section under bending about major axis, the stress ratio ψ and effective width of the flange subject to stress gradient are determined based on gross section properties. For the web, they are calculated using the effective area of the compression flange and the gross area of the web.

Under bending about minor axis, for C, Hat, Box and Angle sections, the effective width of the web (vertical flange for Angle section) is calculated first based on gross section and that of the flanges (horizontal flange for Angle section) are obtained using the section consisting of the effective area of the web and gross area of the flanges. For I, T, and Z sections, the effective width of the flange under compression or stress gradient is determined first based on the gross section and that of the web is computed using the effective area of the flanges and the gross area of the web.

i. <u>Partially stiffened element with simple lip edge stiffener under uniform compression:</u>

For the element partially stiffened with a simple lip edge stiffener under uniform compression, the effective widths, b_{e2} and c_{eff} (Figure B-1,) and the reduced thickness, t_{red}, of the flange and the lip stiffener (Figure B-2) are determined using an iteration procedure as follows:

1. Calculate the lip slenderness following the general case above for outstand element but with k_σ taken as:

$$k_\sigma = \begin{cases} 0.5 & b_{p,c}/b_p \leq 0.35 \\ 0.5 + 0.83\sqrt[3]{\left(\frac{b_{p,c}}{b_p} - 0.35\right)^2} & 0.35 < b_{p,c}/b_p \leq 0.6 \\ 0.5 + 0.83\sqrt[3]{(0.6-0.35)^2} = 0.83 & 0.6 < b_{p,c}/b_p \end{cases}$$

(EC3-3 5.5.3.2(5) Eq. 5.13b & 5.13c)

2. Assume $\chi_d = 1.0$
3. Calculate the effective width b_{e2} of the flange using a reduced slenderness $\bar{\lambda}_{p,red}$ of the flange in place of $\bar{\lambda}_p$ following the general case above for internal element and Table B-2:
   $\bar{\lambda}_{p,red} = \bar{\lambda}_p\sqrt{\chi_d}$ (EC3-3 5.5.3.3(9) Eq. 5.20)
4. Determine the effective width c_{eff} of the lip also with the reduced slenderness $\bar{\lambda}_{p,red}$ of the lip in place of $\bar{\lambda}_p$ for outstand element as described in the general case:
   $\bar{\lambda}_{p,red} = \bar{\lambda}_p\sqrt{\chi_d}$ (EC3-3 5.5.3.3(9) Eq. 5.20)
   $c_{eff} = \rho b_{p,c}$ (EC3-3 5.5.3.2(5) Eq. 5.13a)
5. Compute reduction factor χ_d for the distortional buckling resistance as follows:

$$\chi_d = \begin{cases} 1.0 & \bar{\lambda}_d \leq 0.65 \\ 1.47 - 0.723\bar{\lambda}_d & 0.65 < \bar{\lambda}_d < 1.38 \\ \frac{0.66}{\bar{\lambda}_d} & \bar{\lambda}_d \geq 1.38 \end{cases}$$ (EC3-3 5.5.3.1(7) Eq. 5.12a, b & c)

$\bar{\lambda}_d = \sqrt{f_{yb}/\sigma_{cr,s}}$ (EC3-3 5.5.3.1(7) Eq. 5.12d)

$\sigma_{cr,s} = \frac{2\sqrt{KEI_s}}{A_s}$ (EC3-3 5.5.3.2(7) Eq. 5.15)

$A_s = t(b_{e2} + c_{eff})$ (EC3-3 5.5.3.2(6) Eq. 5.14a)

I_s = effective moment of inertia of the stiffener including b_{e2} and c_{eff} about the centroid axis a-a (Figure B-1) and generally calculated as:

$$I_s = \frac{t\left(t^2b_{e2}^2 + 4b_{e2}c_{eff}^3 - 4b_{e2}c_{eff}^3cos^2(\theta) + t^2b_{e2}c_{eff} + c_{eff}^4 - c_{eff}^4cos^2(\theta)\right)}{12(b_{e2} + c_{eff})}$$

$K = \frac{Et^3}{4(1-\upsilon^2)}\frac{1}{(b_1^2h_w + b_1^3 + 0.5b_1b_2h_wk_f)}$ (EC3-3 5.5.3.1(5) Eq. 5.10b)

where

b_1	is the distance from the web-to-flange junction to the gravity center of the effective area of the edge stiffener (Figure B-1) of flange 1 (or top flange)
b_2	is the distance from the web-to-flange junction to the gravity center of the effective area of the edge stiffener of flange 2 (or bottom flange)
h_w	is the web depth
$k_f = 0$	if flange 2 is in tension (e.g. for beam bending about y-y axis)
$k_f = \frac{A_{s2}}{A_{s1}}$	if flange 2 is in compression (e.g. for beam in compression)
$k_f = 1$	for section symmetric about y-y axis in compression
A_{s1}	is the effective area of stiffener 1 (or top stiffener)

A_{s2}	is the effective area of stiffener 2 (or bottom stiffener)
E	is the modulus of elasticity of steel material
υ	is the Poisson's ratio of steel material
t	is the thickness of the element
θ	is the angle of the lip below the flange

6. Repeat steps 3 through 5 iteratively until χ_d converges.
7. The reduced thickness of b_{e2} of the flange and c_{eff} of the lip is $t_{red} = t\chi_d$

Note that b_{e1} of the flange is not affected by distortional buckling and not included in the iteration procedure described above.

For the determination of effective section properties under uniform axial compression, the iterative procedure is carried out for both flanges 1 & 2 (or top and bottom flanges) simultaneously as their properties are required to compute the parameters k_f and K. Under bending about major axis, however, the iterative procedure can be carried out separately for each flange at a time as only one flange is in compression while the other is in tension.

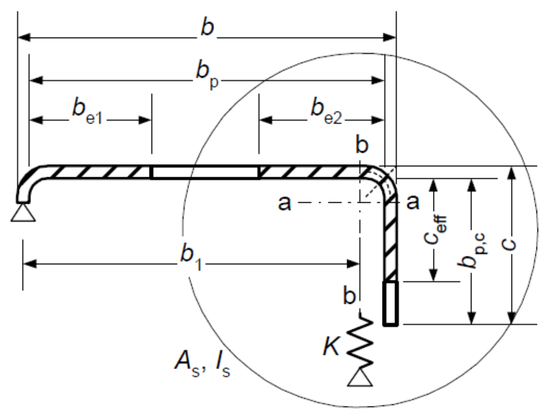


Figure B-1 Edge stiffener (Dubina et al., 2012)

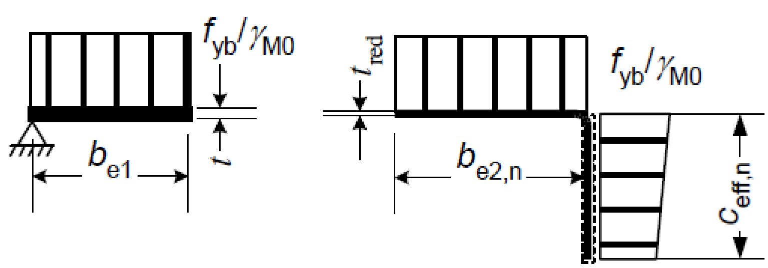


Figure B-2 Effective properties at the final n^{th} iteration (Dubina et al., 2012)

ii. <u>*Partially stiffened element with simple lip edge stiffener under stress gradient:*</u>
For the element partially stiffened with a simple lip edge stiffener under stress gradient (e.g. the flange and lip of C section under bending about minor axis such that the lips are in compression,) the iterative procedure described in Section *i* is carried out for both flanges and lips simultaneously to compute the effective width and thickness of stiffener, but with b_{e2}

replaced by b_{e1} of each of the stiffeners because b_{e1} is the effective part that is adjacent to the lip as shown in Table B-2 for the case with stress gradient.

As the effective width, $b_{eff,i}$, and effective thickness, $t_{eff,i}$ (t or t_{red}), of each of the elements in the cross-section are determined, the effective section properties are calculated as follows:

Effective area: $A_{eff} = \sum_i b_{eff,i} t_{eff,i}$

Effective neutral axis: $\bar{z}_{eff} = \frac{\sum_i b_{eff,i} t_{eff,i} z_{eff,i}}{\sum_i b_{eff,i} t_{eff,i}}$

Effective moment of inertia:

$$I_{y,eff} = I_y - \sum t b_{red,i} \bar{z}_i^2 - \sum (t - t_{eff,i}) b_{eff,i} \bar{z}_i^2 - \sum I_{y,i} - A_{eff} (\bar{z} - \bar{z}_{eff})^2$$

where:

- $z_{eff,i} =$ distance of the effective element i measured from the top flange centerline.
- $\bar{z}_i =$ distance of the effective element i measured from centroid of the unreduced gross cross-section.
- $\bar{z} =$ centroid of the unreduced gross cross-section measured from the top flange centerline.
- $b_{red,i} =$ the reduced amount of width of element i (e.g., $b_{red} = b_p - b_{e1} - b_{e2}$ for the flange as shown in Figure B-1)
- $I_y =$ moment of inertia of the unreduced gross cross-section about y-y major axis calculated using the formulations as described in Part I in Vol. 1 of the AISI 2016 (AISI, 2016) as the effects of rounded corners cannot be neglected. In case the rounded corners can be ignored, it is computed assuming the section consists of plane elements with sharp corners.
- $I_{y,i} =$ moment of inertia of reduced portion of element i about its own axis.

In case the influence of the rounded corners must be considered, the effective section properties are reduced as follows:

$A_{eff,red} = A_{eff}(1 - \delta)$ (EC3-3 5.1(4) Eq. 5.1a)

$I_{eff,red} = I_{eff}(1 - 2\delta)$ (EC3-3 5.1(4) Eq. 5.1b)

where

$$\delta = 0.43 \frac{\sum_{j=1}^{n} r_j \frac{\phi_j}{90°}}{\sum_{i=1}^{m} b_{p,i}}$$

- $r_j =$ the internal radius of the curved element j.
- $\phi_j =$ the angle between two plane elements of the curved element j.
- $n =$ the number of curved elements.
- $b_{p,i} =$ the notional flat width of plane element i for the cross-section with sharp corners
- $m =$ the number of plane elements.

Appendix C
References

AISC, 2016a. ANSI/AISC 360-16: An American National Standard – Specification for Structural Steel Buildings, American Institute of Steel Construction. 130 East Randolph Street, Suite 2000, Chicago, Illinois, 60601. July 7.

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Appendix D
Cold-Formed Steel Frame Design Preferences

The Cold-formed steel Frame Design Preferences are basic assignments that apply to all of the cold-formed steel frame members. Tables B-1, B-2, and B-3 list Cold-formed steel Frame Design Preferences for “EN 1993-1-3.” Default values are provided for all preference items. Thus, it is not necessary to specify or change any of the preferences. However, at least review the default values to ensure they are acceptable. Some of the preference items also are available as member specific overwrite items. The Overwrites are described in Appendix C. Overwritten values take precedence over the preferences.

Table F-1: Cold-formed steel Design Preferences

Item	Possible Values	Default Value	Description
Design Code	Design codes available in the current version	AISC360-10/ IBC 2006	The selected design code. Subsequent design is based on this selected code.
Country	“CEN Default”, “Bulgaria”, “Denmark”, “Finland”, “Germany”, “Ireland”, “Norway”, “Poland”, “Portugal”, “Singapore”, “Slovenia”, “Sweden”, “United Kingdom”.	CEN Default	Country specific implementation considering country National Annex. CEN Default is the general version without an annex.

Table F-1: Cold-formed steel Design Preferences

Item	Possible Values	Default Value	Description
Multi-Response Case Design	Envelopes, Step-by-Step, Last Step, Envelopes, All, Step-by-Step - All	Envelopes	Select to indicate how results for multivalued cases (Time history, Nonlinear static or Multi-step static) are considered in the design. - Envelope - considers enveloping values for Time History and Multi-step static and last step values for Nonlinear static. Step-by-Step - considers step by step values for Time History and Multi-step static and last step values for Nonlinear static. Last Step - considers last values for Time History, Multi-step static and Nonlinear static. Envelope - All - considers enveloping values for Time History, Multi-step static and Nonlinear static. Step-by-Step - All - considers step by step values for Time History, Multi-step static and Nonlinear static. Step-by-Step and Step-by-Step - All default to the corresponding Envelope if more then one multivalued case is present in the combo.
Demand/Capacity Ratio Limit	≤1.0	0.95	The demand/capacity ratio limit to be used for acceptability. D/C ratios that are less than or equal to this value are considered acceptable.
Combinations Equation	"Eq. 6.10" and maximum of "Eq. 6.10a" and "Eq. 6.10b"	"Eq. 6.10"	Design load combinations to consider. These are either generated from Eq. 6.10 or from both Eqs. 6.10a and 6.10b. For the second choice both set of equations are considered.
Reliability Class	Class 1, Class 2, Class 3	Class 2	Reliability class defines a scale factor for load combinations. This is currently only used for Denmark, Finland, and Sweden.
Interaction Factors Method	Method 1, Method 2	Method 2	Method for determining the interaction factors for Eurocode 3-2005. This is either Method 1 from Annex A or Method 2 from Annex B.
Consider P-Delta Done?	Yes, No	No	Toggle to consider whether P-Delta analysis is done. This affects K factor calculation.
γ_{M0}	≥ 1.0	1.0	The partial factor for resistance of cross-sections.
γ_{M1}	≥ 1.0	1.0	Partial factor for the resistance of members to instability.
γ_{M2}	≥ 1.0	1.25	Partial factor for the resistance of members in tension to fracture.
Pattern Live Load Factor	≤1.0	0.75	The live load factor for automatic generation of load combinations involving pattern live loads and dead loads.

Appendix E
Cold-formed Steel Frame Design Overwrites

The structural model may contain frame elements made of several structural materials: steel, concrete, cold-formed steel, cold-formed steel and other materials. The program supports separate design procedures for each material type. By default, the program determines the design procedure from the material of the frame member.

The software allows the user to turn off or turn on design of specific members by selecting *No Design* or *Default from material*. Refer to the program Help for information about overwriting the design procedure.

Overwrites

The cold-formed steel frame design Overwrites are basic assignments that apply only to those elements to which they are assigned. Table C-1 lists Cold-formed steel Frame Design Overwrites for "EN 1993-1-3." Default values are provided for all overwrite items. Thus, it is not necessary to specify or change any of the overwrites. However, at least review the default values to ensure they are acceptable. When changes are made to overwrite items, the program applies the changes only to the elements to which they are specifically assigned. overwritten values take precedence over the preferences (Appendix B).

Table G-1 Cold-formed steel Frame Design Overwrites for "EN 1993-1-3"

Item	Possible Values	Default Value	Description
Current Design Section			The design section for the selected frame objects. When this overwrite is applied, any previous auto select section assigned to the frame object is removed. Program determined value means it is taken from the analysis section.
D/C Ratio Limit			The demand/capacity ratio limit to be used for acceptability. D/C ratios that are less than or equal to this value are considered acceptable. Specifying zero means the value is program determined.
Live Load Reduction Factor			The reducible live load is multiplied by this factor to obtain the reduced live load for the frame object. Specifying zero means the value is program determined.

Table G-1 Cold-formed steel Frame Design Overwrites for "EN 1993-1-3"

Item	Possible Values	Default Value	Description
Yield Stress, fyb			Material base yield strength used in the design/check. Specifying zero means the value is program determined. The program determined value is taken from the material property assigned to the frame object.
Net Area to Total Area Ratio			The ratio of the net area at the end joint to gross cross-sectional area of the section. This ratio affects the design of axial tension members. Specifying zero means the value is the program default, which is 1.
Unbraced Length Ratio (Major)			Unbraced length factor for buckling about the frame object major axis; specified as a fraction of the frame object length. This factor times the frame object length gives the unbraced length for the object. Specifying zero means the value is program determined.
Unbraced Length Ratio (Minor)			Unbraced length factor for buckling about the frame object minor axis; specified as a fraction of the frame object length. This factor times the frame object length gives the unbraced length for the object. Specifying zero means the value is program determined.
Unbraced Length Ratio (LTB)			Unbraced length factor for lateral-torsional buckling for the frame object; specified as a fraction of the frame object length. This factor times the frame object length gives the unbraced length for the object. Specifying zero means the value is program determined.
Effective Length Factor Sway (K2 Major)			Effective length factor for buckling about the frame object major axis assuming that the frame is braced at the joints against sidesway; specified as a fraction of the frame object length. This factor times the frame object length gives the effective length for the object. Specifying zero means the value is program determined. The factor is used for axial compression capacity.
Effective Length Factor Sway (K2 Minor)			Effective length factor for buckling about the frame object minor axis assuming that the frame is braced at the joints against sidesway; specified as a fraction of the frame object length. This factor times the frame object length gives the effective length for the object. Specifying zero means the value is program determined. The factor is used for axial compression capacity.
Effective Length Factor Sway (LTB)			Effective length factor for lateral-torsional buckling; specified as a fraction of the frame object length. This factor times the frame object length gives the effective length for the object. Specifying zero means the value is program determined. For beam design, this factor is taken as 1 by default. The values should be set by the user.
Effective Length Factor Braced (K1 Major)			Effective length factor for buckling about the frame object major axis; specified as a fraction of the frame object length. This factor times the frame object length gives the effective length for the object. Specifying zero means the value is program determined. For beam design, this factor is always taken as 1, regardless of any other value specified in the Overwrites. This factor is used for the calculation of *k* factors.
Effective Length Factor Braced (K1 Minor)			Effective length factor for buckling about the frame object minor axis; specified as a fraction of the frame object length. This factor times the frame object length gives the effective length for the object. Specifying zero means the value is program determined. For beam design, this factor is always taken as 1, regardless of any other value specified in the Overwrites. This factor is used for calculation of the *k* factors.

Table G-1 Cold-formed steel Frame Design Overwrites for "EN 1993-1-3"

Item	Possible Values	Default Value	Description
Bending Coefficient (C1)			Unitless factor; C_1 is used in determining the interaction ratio and calculation of elastic critical moment, M_{cr}. Inputting zero means the value is program determined.
Bending Coefficient (C2)			Unitless factor; C_2 is used in calculation of elastic critical moment, M_{cr} . Inputting zero means the value is program determined.
Bending Coefficient (C3)			Unitless factor; C_2 is used in calculation of elastic critical moment, M_{cr} . Inputting zero means the value is program determined.
Moment coefficient (k_{yy})			Moment coefficient for major axis bending determined by Method 1 or Method 2 from Annex A or B of the code.
Moment coefficient (k_{zz})			Moment coefficient for minor axis bending determined by Method 1 or Method 2 from Annex A or B of the code.
Moment coefficient (k_{zy})			Moment coefficient determined by Method 1 or Method 2 from Annex A or B of the code.
Moment coefficient (k_{yz})			Moment coefficient determined by Method 1 or Method 2 from Annex A or B of the code.
Column Buckling Curve (y-y)			Column buckling curve to be used for flexural buckling about major axis. This can be "a0," "a," "b," "c," or "d." It determines the imperfection factors for buckling curve. If not overwritten, it is taken from Table 6.2 of the EN 1993-1-1:2005 code.
Column Buckling Curve (z-z)			Column buckling curve to be used for flexural buckling about minor axis. This can be "a0," "a," "b," "c," or "d." It determines the imperfection factors for buckling curve. If not overwritten, it is taken from Table 6.2 of the EN 1993-1-1:2005 code.
Buckling Curve for LTB			Buckling curve to be used for lateral-torsional buckling. This can be "a0," "a," "b," "c," or "d." The program gives one extra option "a0" following flexural buckling mode. It determines the imperfection factors for buckling curve. If not overwritten, it is taken from the Table 6.4 of the EN 1993-1-1:2005 code.
Elastic torsional buckling force, $N_{cr,T}$			Elastic torsional buckling force. This affects axial buckling.
Elastic flexural-torsional buckling force, $N_{cr,TF}$			Elastic flexural-torsional buckling force. This affects axial buckling.
Axial compressive capacity, N_{Rk}			Allowable axial compressive capacity. Specifying 0 means the value is program determined.
Axial tensile capacity, $N_{t,Rd}$			Allowable axial tensile capacity. Specifying 0 means the value is program determined.
Major bending capacity, M_{yRk}			Allowable bending moment capacity without reduction factor for buckling and partial factor for resistance in major axis bending. Specifying 0 means the value is program determined. For symmetrical sections major bending is bending about the local 3-axis. For unsymmetrical sections (e.g., angles) major bending is the bending about the section principal axis with the larger moment of inertia.
Minor bending capacity, M_{zRk}			Allowable bending moment capacity without reduction factor for buckling and partial factor for resistance in minor axis bending. Specifying 0 means the value is program determined. For symmetrical sections minor bending is bending about the local 2-axis. For unsymmetrical sections (e.g., angles) minor bending is the bending about the section principal axis with the smaller moment of inertia.

Table G-1 Cold-formed steel Frame Design Overwrites for "EN 1993-1-3"

Item	Possible Values	Default Value	Description
Major shear capacity, $V_{y,Rd}$			Allowable shear capacity force for major direction shear. Specifying 0 means the value is program determined. For symmetrical sections major shear is shear in the local 2-axis direction. For unsymmetrical sections (e.g., angles) major shear is the shear associated with major bending. Note that major bending is the bending about the section principal axis with the larger moment of inertia.
Minor shear capacity, $V_{z,Rd}$			Allowable shear capacity force for minor direction shear. Specifying 0 means the value is program determined. For symmetrical sections minor shear is shear in the local 3-axis direction. For unsymmetrical sections (e.g., angles) minor shear is the shear associated with minor bending. Note that minor bending is the bending about the section principal axis with the smaller moment of inertia.
k_w			End warping factor. It is used in calculation of elastic critical moment, M_{cr}. It should have a value between 0.5 and 1.0. Specifying zero means the value is program default which is 1.0.
Z_a is Program Determined?			Toggle to select an option of determination of the coordinate of the point of load application, Z_a. If "Yes" is selected, Z_a will be program determined, which is defaulted to be on top of the section. If "No" is selected, Z_a will be defined by the users.
Z_a			Coordinate of the point of load application with respect to the minor principle axis in the coordinate system of the centroid of the section. It is used in calculation of elastic critical moment, M_{cr}. If the input value results in a location beyond the top or bottom of the section, it will be automatically adjusted to the top or bottom of the section, respectively, in the design.

Appendix F
Nationally Determined Parameters (NDPs)

This appendix provides a listing of the Nationally Determined Parameters (NDPs) used by default for the various country implementations. Several of these parameters can be modified either through the design preferences or the design overwrites.

F.1 CEN Default

Table F.1: CEN Default NDP Values

Code Clause	NDP	Default Value
EC3 6.1(1)	γ_{M0}	1.00
EC3 6.1(1)	γ_{M1}	1.00
EC3 6.1(1)	γ_{M2}	1.25
EC0 6.4.3.2	Combinations equation	Eq. 6.10
EC3 6.3.3(5)	Interaction factors method	Method 2
EC0 Table A1.2(B)	$\gamma_{Gj,sup}$	1.35
EC0 Table A1.2(B)	$\gamma_{Gj,inf}$	1.00
EC0 Table A1.2(B)	$\gamma_{Q,1}$	1.5
EC0 Table A1.1	$\psi_{0,i}$	0.7 (live load) 0.6 (wind load)
EC0 Table A1.2(B)	ξ	0.85
EC0 Table A1.1	$\psi_{2,i}$	0.3 (assumed office/residential)
EC3 6.3.2.2(2)	α_{LT}	0.21 for buckling curve a 0.34 for buckling curve b 0.49 for buckling curve c 0.76 for buckling curve d
EC3 6.3.2.3(1)	β	0.75
EC3-1-5 5.1(2)	η	1.20 for $f_y \leq 460$ N/mm^2 1.00 for $f_y > 460$ N/mm^2

F.2 Bulgaria

Table F.2 lists the NDP values for the Bulgarian National Annex, where they differ from the CEN Default values listed in Table F.1.

Table F.2: Bulgaria NDP Values

Code Clause	NDP	Default Value
EC3 6.1(1)	γ_{M0}	1.05
EC3 6.1(1)	γ_{M1}	1.05

F.3 Slovenia

The NDP values for the Slovenian National Annex, are the same as the CEN Default values listed in Table F.1.

F.4 United Kingdom

Table F.3 lists the NDP values for the United Kingdom National Annex, where they differ from the CEN Default values listed in Table F.1.

Table F.3: United Kingdom NDP Values

Code Clause	NDP	Default Value
EC3 6.1(1)	γ_{M2}	1.10
EC0 Table A1.1	$\psi_{0,i}$	0.7 (live load) 0.5 (wind load)
EC0 Table A1.2(B)	ξ	0.925
EC3-1-5 5.1(2)	η	1.00

F.5 Norway

Table F.4 lists the NDP values for the Norwegian National Annex, where they differ from the CEN Default values listed in Table F.1.

Table F.4: Norway NDP Values

Code Clause	NDP	Default Value
EC3 6.1(1)	γ_{M0}	1.05
EC3 6.1(1)	γ_{M1}	1.05
EC0 Table A1.2(B)	ξ	0.89

F.6 Sweden

Table F.5 lists the NDP values for the Sweden National Annex, where they differ from the CEN Default values listed in Table F.1.

Table F.5: Sweden NDP Values

Code Clause	NDP	Default Value
EC0 6.4.3.2	Combinations equation	Eq. 6.10a/b
EC3 6.3.3(5)	Interaction factors method	Method 1
EC0 Table A1.2(B)	γ_d	Class 1 = 0.83, Class 2 = 0.91, Class 3 = 1.0
EC0 Table A1.2(B)	$\gamma_{Gj,sup}$	1.35*γ_d
EC0 Table A1.2(B)	$\gamma_{Q,1}$	1.5*γ_d
EC0 Table A1.1	$\psi_{0,i}$	0.7 (live load) 0.3 (wind load)
EC0 Table A1.2(B)	ξ	0.89

F.7 Finland

Table F.6 lists the NDP values for the Finland National Annex, where they differ from the CEN Default values listed in Table F.1.

Table F.6: Finland NDP Values

Code Clause	NDP	Default Value
EC0 6.4.3.2	Combinations equation	Eq. 6.10a/b
EC0 Table A1.2(B)	K_{FI}	Class 1 = 0.9, Class 2 = 1.0, Class 3 = 1.1
EC0 Table A1.2(B)	$\gamma_{Gj,sup}$	1.35*K_{FI}
EC0 Table A1.2(B)	$\gamma_{Gj,inf}$	0.9
EC0 Table A1.2(B)	$\gamma_{Q,1}$	1.5*K_{FI}

F.8 Denmark

Table F.7 lists the NDP values for the Denmark National Annex, where they differ from the CEN Default values listed in Table F.1.

Table F.7: Denmark NDP Values

Code Clause	NDP	Default Value
EC3 6.1(1)	γ_{M0}	1.10
EC3 6.1(1)	γ_{M1}	1.20
EC3 6.1(1)	γ_{M2}	1.35
EC0 6.4.3.2	Combinations equation	Eq. 6.10a/b
EC0 Table A1.2(B)	K_{FI}	Class 1 = 0.9, Class 2 = 1.0, Class 3 = 1.1
EC0 Table A1.2(B)	$\gamma_{Gj,sup}$	1.2 / 1.0 (Eq. 6.10a / 6.10b)*K_{FI}
EC0 Table A1.2(B)	$\gamma_{Gj,inf}$	1.0 / 0.9 (Eq. 6.10a / 6.10b)
EC0 Table A1.2(B)	$\gamma_{Q,1}$	1.5*K_{FI}
EC0 Table A1.1	$\psi_{0,i}$	0.6 (live load) 0.6 (wind load)
EC0 Table A1.2(B)	ξ	1.0
EC0 Table A1.1	$\psi_{2,i}$	0.2 (assumed office/residential)

F.9 Portugal

Table F.8 lists the NDP values for the Portugal National Annex, where they differ from the CEN Default values listed in Table F.1.

Table F.8: Portugal NDP Values

Code Clause	NDP	Default Value
EC3 6.3.2.3(1)	$\bar{\lambda}_{LT,0}$	0.2
EC3 6.3.2.3(1)	β	1.0

F.10 Germany

Table F.9 lists the NDP values for the German National Annex, where they differ from the CEN Default values listed in Table F.1.

Table F.9: Germany NDP Values

Code Clause	NDP	Default Value
EC3 6.1(1)	γ_{M1}	1.1

F.11 Singapore

Table F.10 lists the NDP values for the Singapore National Annex, where they differ from the CEN Default values listed in Table F.1.

Table F.10: Singapore NDP Values

Code Clause	NDP	Default Value
EC3 6.1(1)	γ_{M2}	1.1
EC3 Annex B	$\overline{\lambda}_Z$ for non-doubly symmetric sections	$\max\left(\overline{\lambda}, \overline{\lambda}_T\right)$
EC3 Table B.1 and B.2	Where sections are not I, H, or hollow sections, k_{yy}, k_{yz}, k_{zy}, and k_{zz} for Class 1 and 2 sections are calculated as if the section is Class 3.	

F.12 Poland

Table F.11 lists the NDP values for the Poland National Annex, where they differ from the CEN Default values listed in Table F.1.

Table F.11: Poland NDP Values

Code Clause	NDP	Default Value
EC3 6.1(1)	γ_{M2}	1.1