Steel Module

Info

Steel Module 6

Datum: 110930

Latest information about programs from **WIN-Statik** and **FEM-Design** see

www.strusoft.com

1 Introduction

1.1 General

With the program **Steel Module** continuous beams and plane frame structures can be designed according to EuroCode EC3. The **Steel Module** is used together with the program **Frame Analysis** in which calculations of section forces is performed.

Design with consideration taken to flexural buckling in the frame plane can be performed for second order theory or for first order theory with help from buckling lengths defined by the user.

Continuous support, as well as support in certain points of the beam as well as transverse stiffeners for open cross-sections can be defined.

The program displays all code prescribed checks depending on type of section and current load.

1.2 Restrictions

In the present version **Steel Module** 6.2 it is not possible to design sections in section class 4.

Support conditions can be defined as hinged, fixed or free edge with consideration to buckling and lateral instability.

Important! A node defined with the **Member** tool will automatically be considered laterally restrained with respect to flexural buckling out of the frame plane at a possible design.

Control of instability out of the frame´s plan is then performed with regard to these defined support conditions.

With the help of the support conditions for each node specified under the option **Support Conditions** equal to one of four Euler buckling cases a reduction factor for flexural buckling, torsional buckling or flexural torsional buckling is calculated and the most dangerous value is chosen.

These support conditions are also used for calculating lateral torsional buckling.

The conditions for the program are such that each node is assumed supported out of the plane in such a way that corresponds to one of these buckling cases.

If a larger buckling length with regard to instability out of the plane than what corresponds to one of the above buckling cases can be assumed, the node should instead be defined with the tool **Unsupported joint**.

The node is then only considered to have sufficient torsional stiffness to be assumed as a hinged support with regard to lateral torsional buckling and flexural buckling is calculated for a buckling length assumed to extend between the nearest supported nodes. If with the above procedure it is not possible to adequately model the reality regarding instability out of the plane, the calculation can alternatively be supplemented with a 2-dimensional calculation also perpendicular to the frame plane.

No design of necessary dimensions for possible buckling stiffeners is performed.

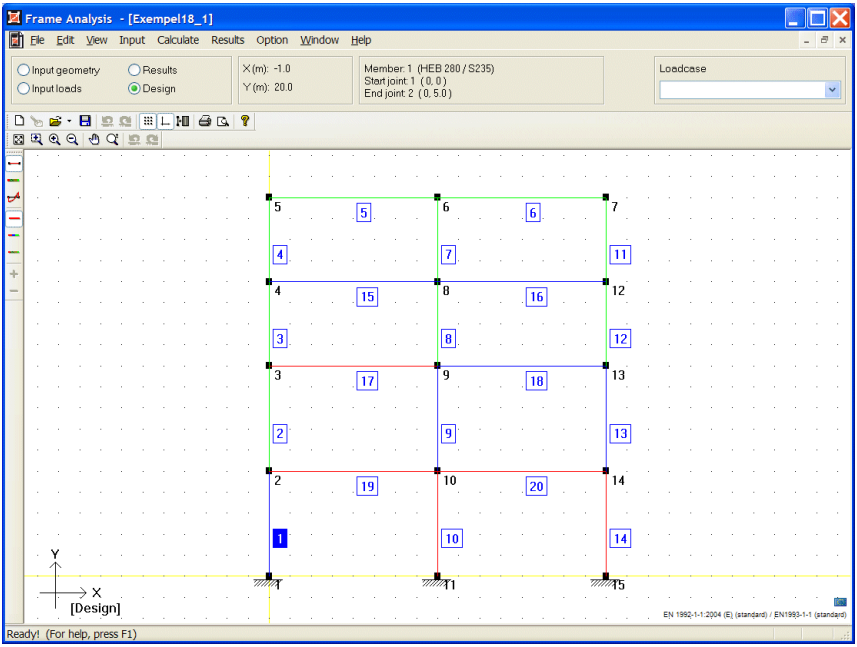
Loads that affect the construction are assumed to be working through the shear centre **SC** of the cross section, which means that there is no torsion of the cross section.

2 Program structure

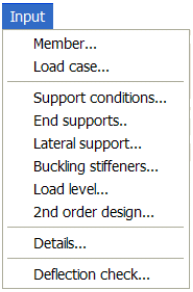
2.1 Design

When the calculation has been completed the **Design** program mode is enabled, and the appearance of the main menu changes as seen below. The **Design** option in the main menu makes it possible to define any additional information that may occur for a design check as well as study the results of the calculation.

Note! This mode is only available if **Steel Module** has been installed.

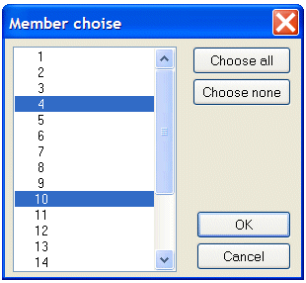


2.2 Input



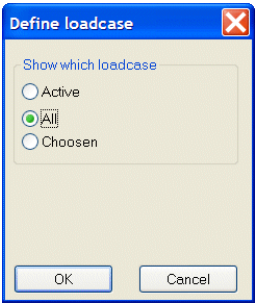
The **Input** menu option makes it possible to define possible additional information for a design. Under **Input** the **Member**, **Load case**, **Support conditions**, **End supports**, **Lateral support**, **Buckling stiffeners**, **Load level**, **2nd order design**, **Details** and **Deflection check** options are situated.

2.2.1 Member



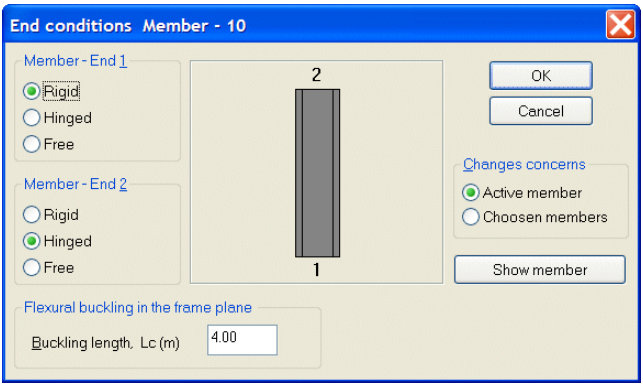
The **Member** option displays a dialog-box in which you can select requested members.

2.2.2 Load case



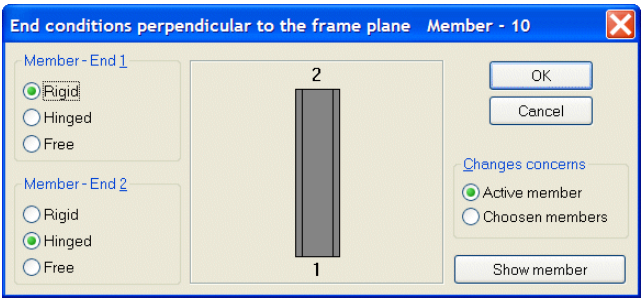
In **Load case** the load cases to be calculated for design check can be chosen.

2.2.3 Support conditions



In the **Support conditions** dialog buckling lengths are defined regarding buckling in the frame plane if the analysis has been performed according to 1st order theory. Here is also support conditions regarding buckling out of frame plane defined. Initially all members defined with the tool **Member** in **Frame Analysis** is supposed to have hinged support conditions at both ends. This means that the buckling length is equal to the member length.

Note! A joint that has been defined with the tool **Member** will automatically be regarded as supported with regard to buckling out of the frame plane. If no lateral support with regard to flexural buckling is expected the tool **Unsupported joint** should be used instead.

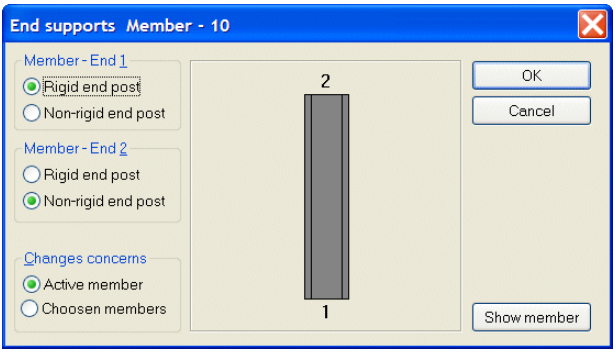


If design is performed according to 2nd order theory only support conditions regarding buckling out of frame plane should be defined.

When defining, only the active member or selected members are affected depending on what is defined above. In the above picture only 1 member is selected which is seen in the title row of the dialog. In order to make it possible to define support conditions at least one member must be selected. To make it possible to define a free end the other end must be fixed.

Buckling length will also be used to calculate initial bow imperfection out of the plane.

2.2.4 End supports



In the **End supports** dialog **Rigid**- or **Non-rigid** end post are defined. The option affects the web shear capacity.

End supports can only defined for open sections.

2.2.5 Lateral supports

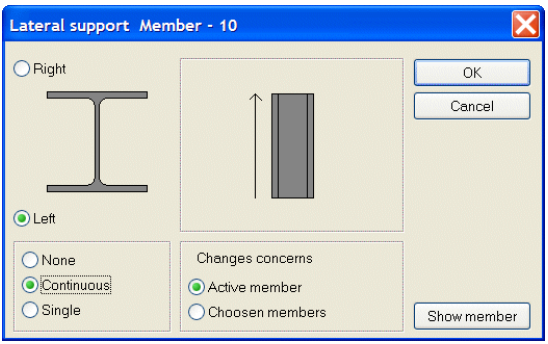
In the **Lateral supports** dialog the user can define lateral support to reduce instability out of the frame plane. **Lateral supports** are defined as continuous or single supports. When a lateral support has been defined it will prevent the

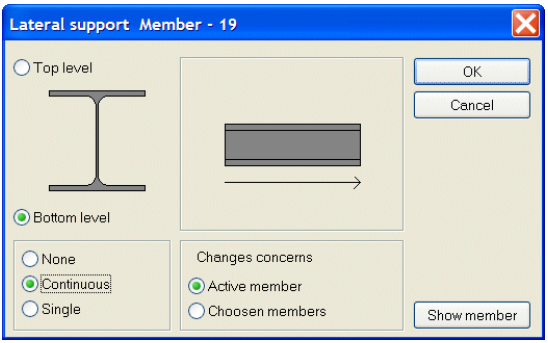
member to bend sideways in the support point. This requires the side supporting structure to have enough strength to be able to prevent the lateral movement.

Max number of lateral point supports allowed are **10**. The single supports may not be defined closer to each other or to a member end than **L/10**.

Lateral support cannot be defined for circular sections.

2.2.5.1 Continuous supports



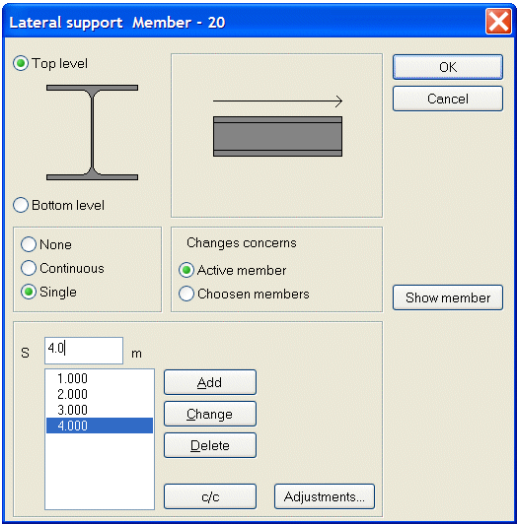


Continuous supports can be defined for one or both edges of the section.

In the above dialogs the left side of a column member (member 10) and the lower edge of a beam member (member 19) are defined continuously supported in the weak direction.

2.2.5.2 Single supports

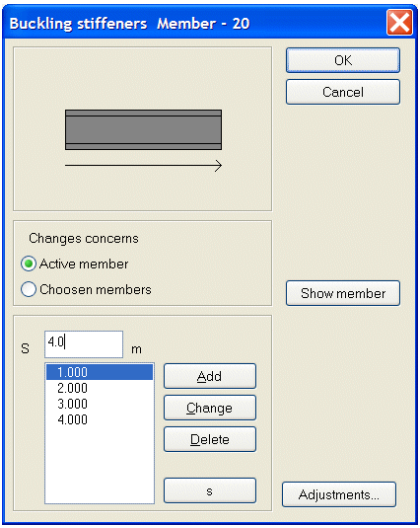
Single supports can be defined for one or both edges of the section. Maximum number of supports is **10**. The supports must not be situated closer to each other or to a member end than **L/10**.



In the above dialog the upper edge of a beam element is supported at four points situated 1,0 m, 2,0 m 3,0 and 4.0 m from the left member end.

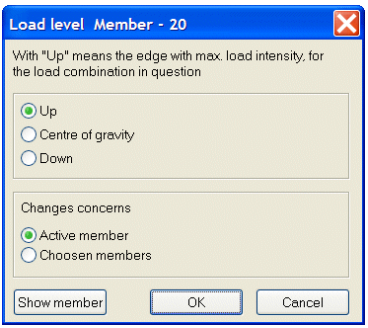
2.2.6 Buckling stiffeners

In the **Buckling stiffeners** dialog possible web stiffeners are defined. This means that the shear buckling capacity could be increased for slender webs. Stiffeners can only be defined for open sections. Maximum number of stiffeners are **10**. The stiffeners must not be situated closer to each other or the member end than **L/10**.



In the dialog above stiffeners are defined at 1,0 m, 2,0 m, 3,0 m and 4,0 m from the left member end.

2.2.7 Load level

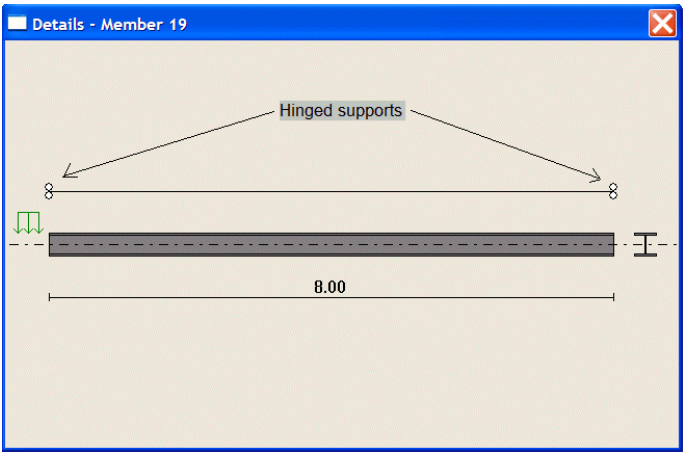


For members sensitive to lateral torsional buckling the capacity depends on the level where the loads are situated.

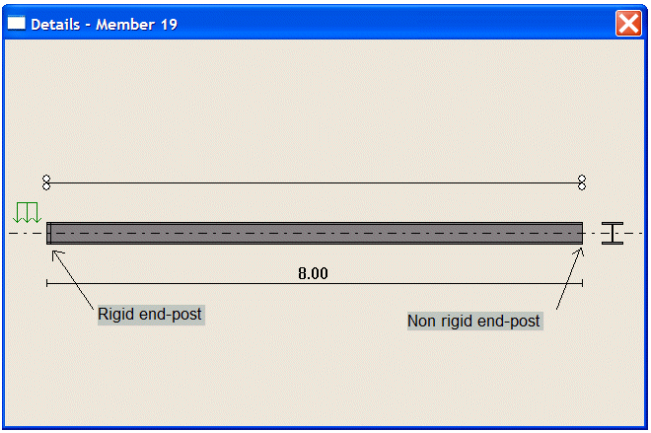
In the dialog to the left is shown how the load level is defined. The program knows in what direction the loads act and can place the load accordingly.

2.2.8 Details

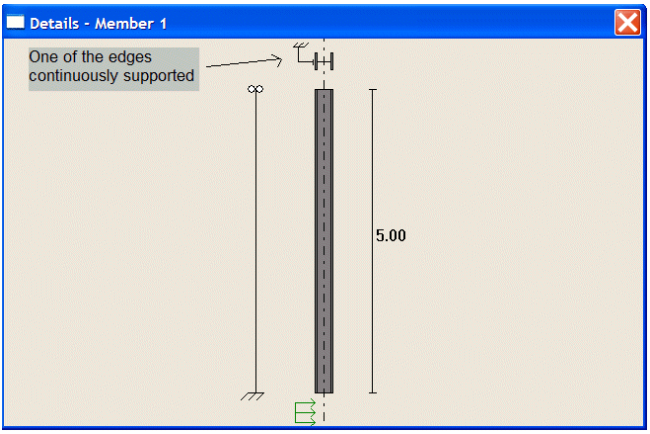
When the option **Details** is chosen a window showing the current member is presented as a beam or a column.



Details can only be defined if the member's stiff direction is in the frame plane. From the picture above the current support conditions can be seen.



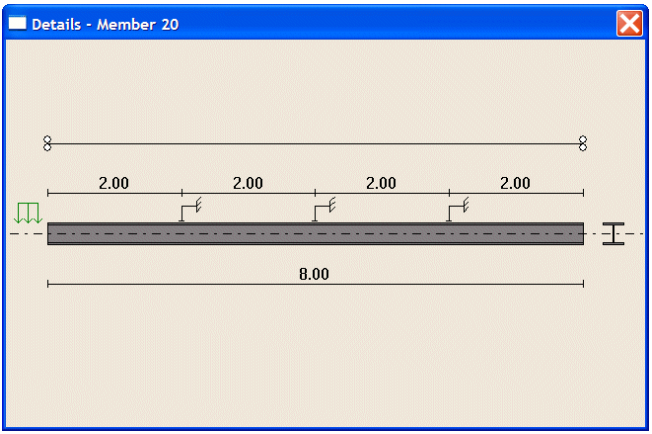
The picture above shows a beam member where rigid end-post has been chosen at the left and non rigid end-post at the right end.



The picture above shows a column member where the left edge is defined continuously supported. For a continuously supported edge the supports are assumed to be situated so closely along the whole member that no instability can occur between the supports.

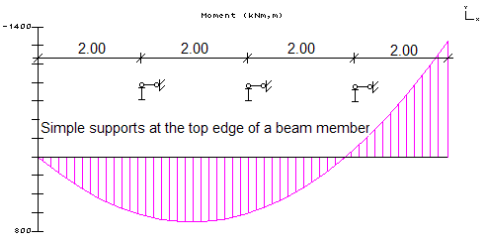
If both edges are continuously supported, no flexural buckling in the weak direction, flexural-torsional buckling or lateral torsional buckling can occur. If only one edge is supported flexural-torsional buckling can occur.

For lateral torsional buckling the capacity depends on how large part of the supported edge that is in compression. If an edge is in compression along the entire member and continuously supported no lateral torsional buckling is possible but if the supported edge is in tension the capacity is calculated with regard to this. The connection between the supports and the member is assumed to be hinged.



The picture above shows single supports at the top edge for a beam member. For single supports instability between these supports can occur. When calculating the capacity due to pure flexural buckling in the weak direction the buckling length is calculated with regard to all single supports regardless which edge these are situated on. For flexural-torsional buckling the buckling length is calculated with regard to single supports situated at the same edge and the maximum length from the two edges will be decisive.

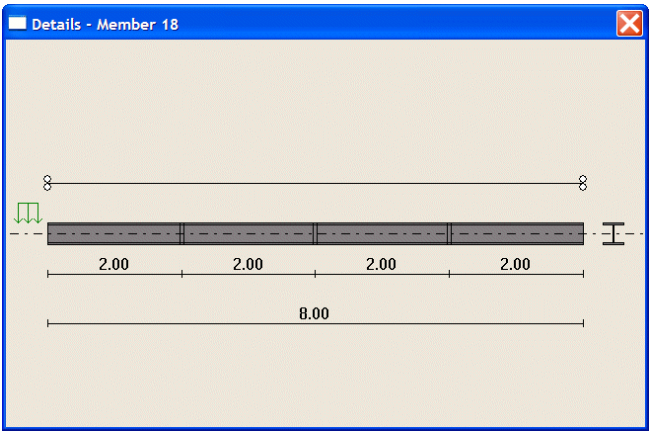
The capacity due to lateral torsional buckling is only dependent of single supports defined at an edge in compression for the current load case. No consideration to possible supports defined at an edge in tension.



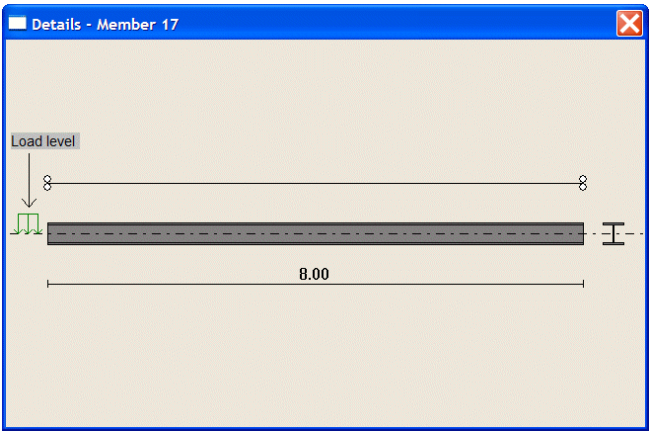
For a beam member with moment distribution and lateral support as above no consideration is made to the rightmost support concerning lateral torsional buckling as the top edge is in tension at this point. The calculation will thus be made

for three areas with length 2,0 m, 2,0 m and 4,0 m. If single supports are defined at an edge which is totally or partly in compression no consideration is made to a possible continuous support at the other edge.

The calculation of the elastic critical lateral torsional buckling moment assumes a linearly varying moment distribution between the supports. A check is therefore made if the supports are situated close enough for such an assumption.

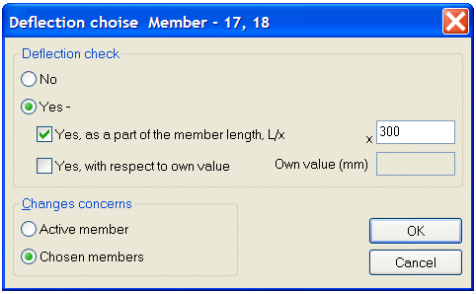


The picture shows how buckling stiffeners are displayed for a beam member.



The picture shows how the load level is displayed for a beam member.

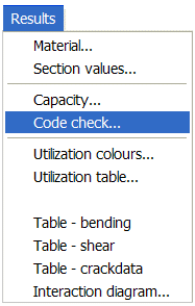
2.2.9 Deflection check



In the **Deflection check** dialog a deflection check for active or selected members are defined. The deflection check can then be performed with respect to a factor of current span length or a stated value.

In the latter case the same criterion will be used for all selected members.

2.3 Results



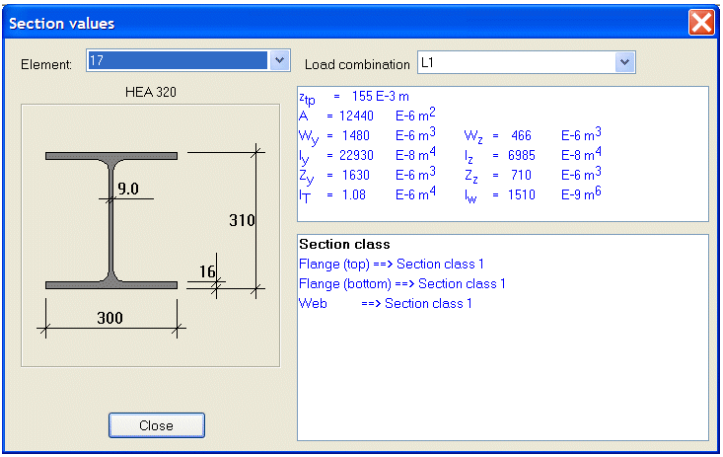
The **Results** menu option makes it possible to study the results generated by the calculation. Under **Results** the following options can be found: **Material**, **Section values**, **Capacity**, **Code check**, **Utilization colours and Utilization table**. The remaining options are only relevant for concrete members.

2.3.1 Material

Material design strengths are shown in the **Material** dialog box.

2.3.2 Section values

The **Section values** option displays a graphic picture of the current profile with associated geometrical properties and the calculated section classes.

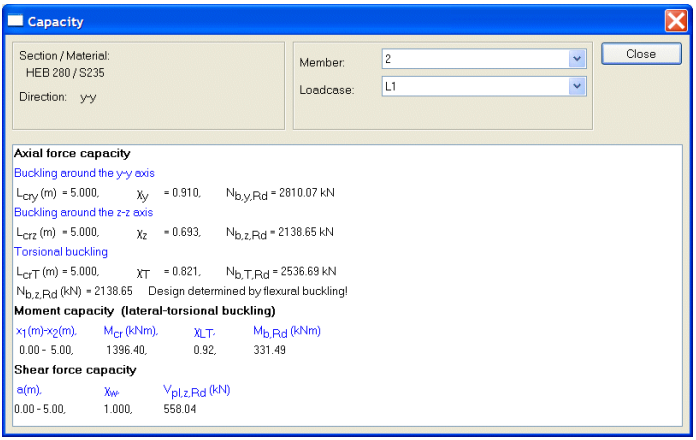


In the above picture a double symmetrical I-section is shown where both flanges and web end up in section class 1.

2.3.3 Capacity

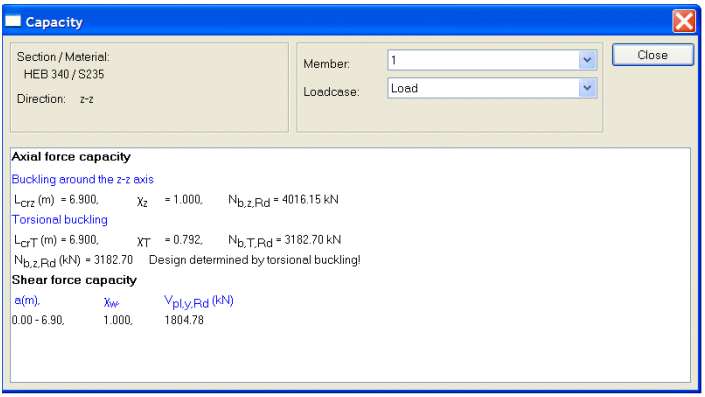
The **Capacity** option displays a dialog box where the capacity of the active member with regard to instability corresponding to the current section forces is displayed. What is shown depends on section type, loading and possible lateral bracing or defined buckling stiffeners. Below the appearance of the capacity dialog box for some cases are shown:

Double symmetrical section designed according to 1th order theory. Axial force, moment and shear force in the stiff direction.



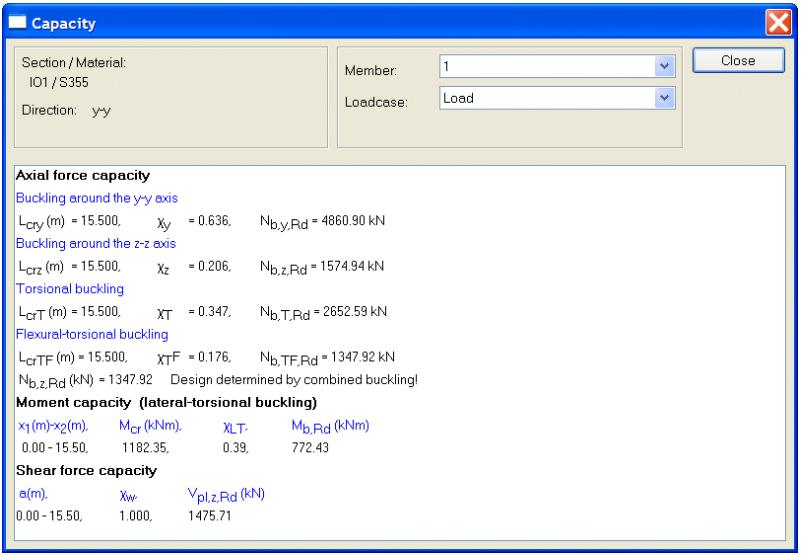
In this case the axial force capacity with regard to flexural buckling around the local y-axis which in this case means buckling in the plane of the frame is displayed together with flexural buckling out of the frame plane and torsional buckling. $\mathbf{L_{cry}}$, $\mathbf{L_{crz}}$, $\mathbf{L_{crT}}$ are the corresponding buckling lengths and $\boldsymbol{\chi_y}$, $\boldsymbol{\chi_z}$, $\boldsymbol{\chi_T}$ are the corresponding reduction factors.

Double symmetrical section designed according to 2nd order theory. Axial force, moment and shear force in the stiff direction.



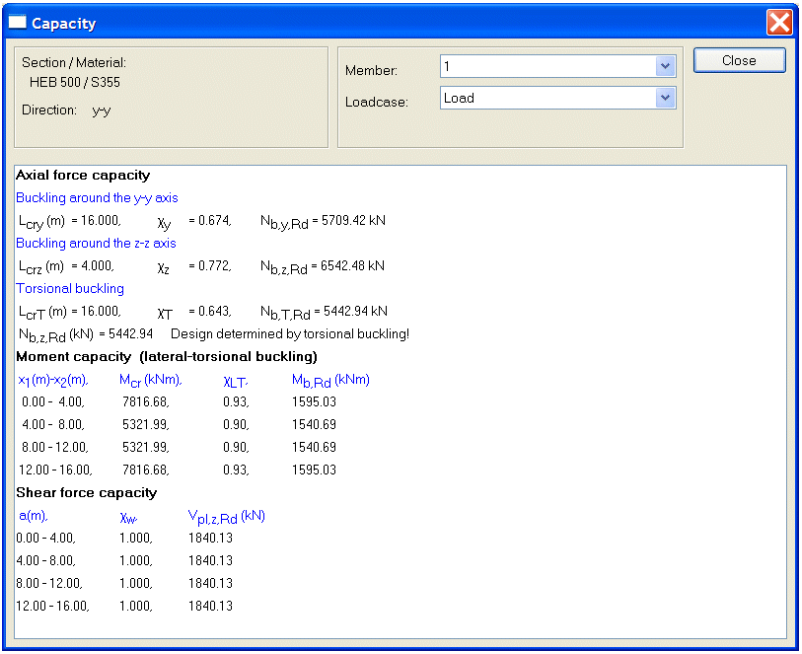
In this case no capacity with regard to flexural buckling in the frame plane is displayed as this is checked with the 2nd order moment.

Single symmetrical section designed according to 1th order theory. Axial force, moment and shear force in the stiff direction.



In this case also flexural-torsional buckling must be considered when calculating the axial force capacity.

Double symmetrical section designed according to 1th order theory. Axial force, moment and shear force in stiff direction. The member is laterally supported at the compressed edge. The web is reinforced with buckling stiffeners.



In the picture above is shown that for the axial force capacity in the weak direction torsional buckling is decisive in this case. Moment- and shear capacity are shown for all the areas created because of defined single supports and buckling stiffeners.

2.3.4 Code Check

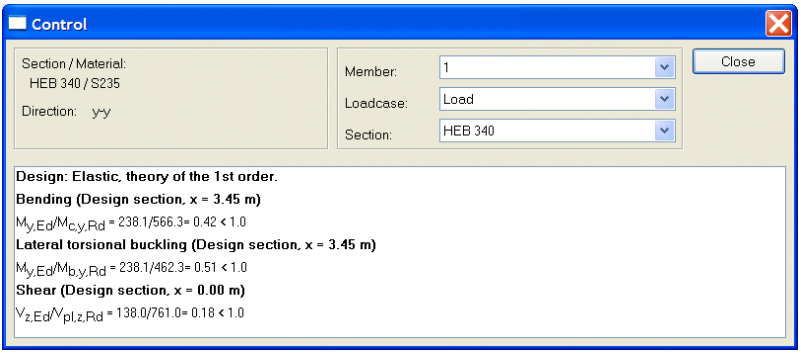
The **Code Check** option displays for the active member at Ultimate Limit State all capacity checks according to **EC3** as well as a possible deflection check for the Serviceability Limit State. It is also shown where the most critical section is located, and if upper or lower edge is critical. For vertical members the right side corresponds to the upper edge. What is displayed depends on section type,

section class, loading and possible lateral bracing. Checks that might indicate that the load-bearing capacity is insufficient are shown with red text. The sum of the interaction formula displayed is the maximum value along the member length for the current load case.

If more than one load case is defined the maximum utilization with regard to all load cases are displayed for each code check and also wich load case that is descisive.

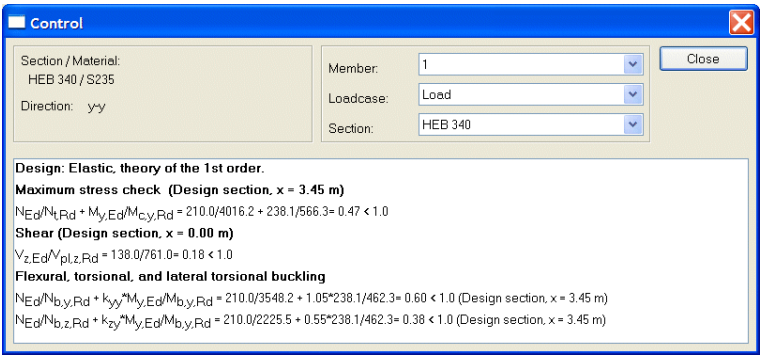
Below the appearance of the code check dialog box for some cases are shown:

Moment and shear force in stiff direction. Double symmetrical section without lateral bracing.



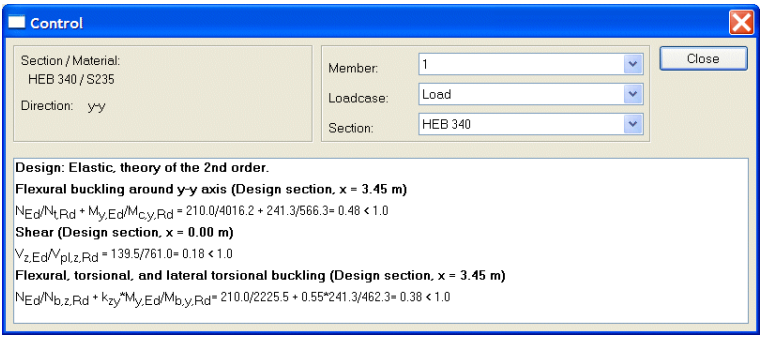
For the above member bending, lateral torsional buckling and shear are checked.

Axial force, moment and shear force in stiff direction. Double symmetrical section without lateral bracing. Design according to 1th order theory.



For the above member maximum stress, shear and relevant buckling phenomena are checked.

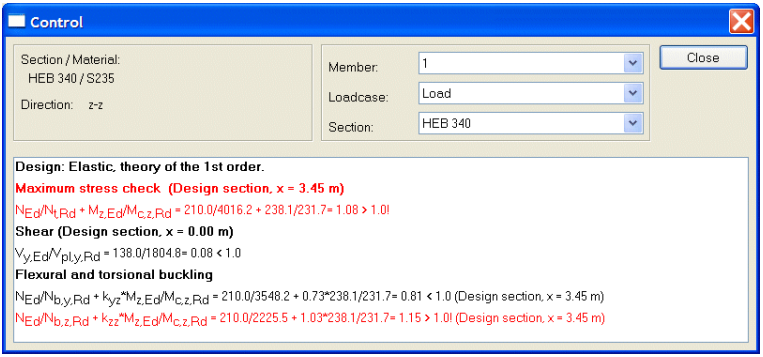
Axial force, moment and shear force in stiff direction. Double symmetrical section without lateral bracing. Design according to 2nd order theory.



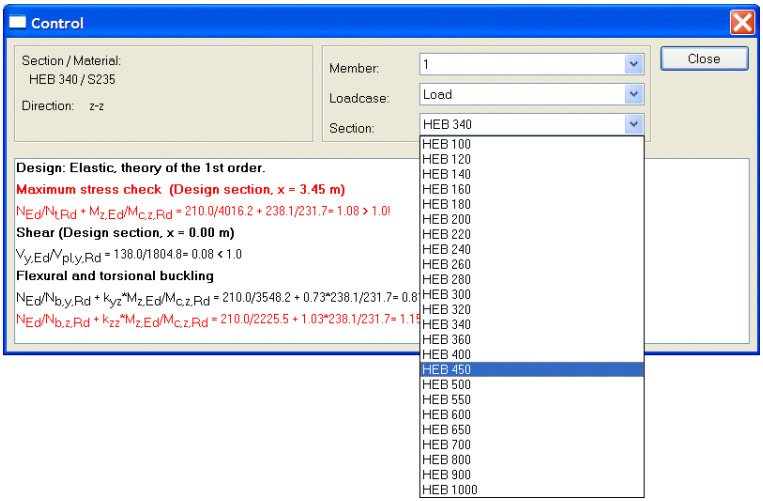
For the above member the flexural buckling check in the stiff direction is made with regard to 2nd order moments.

2.3.5 Quick way to change section

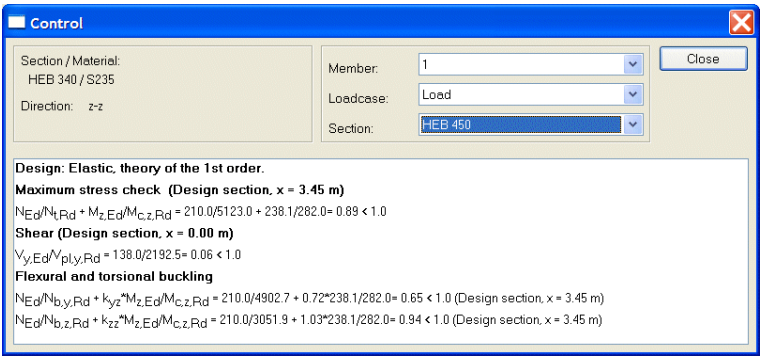
In the **Code check** dialog box it is also possible to change section temporarily. This is very useful if the capacity is inadequate in order to quickly find a section with sufficient capacity.



Above the current section HEB340 has inadequate capacity.



It is possible to change section within the same section type.



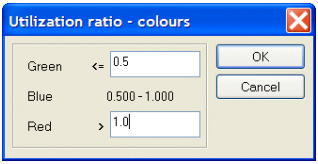
By changing to a HEB450 the capacity is sufficient for the current load case.

Warning!
Changing the section in the code check window means that a new design is made but the analysis is the same although the stiffness distribution in the structure has changed. The ability of section change has been created for the user to quickly find an appropriate section but then a new analysis has to be performed with the new sections. The program will therefore change back to the original section when the code check dialog window is closed.

2.3.6 Utilization colours

When the design has been completed the included members will get different colours depending on how the result turns out. The members that pass all checks will get a green colour and the others will get a red colour as default.

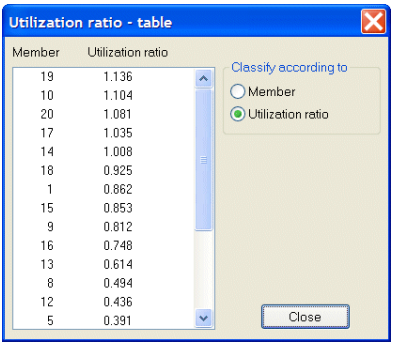
Members can also be coloured blue according to the **Utilization ratio-colours** option. The purpose with this can e.g. be to swiftly survey, not only which members will pass but also to see which members are being utilized over a user-given limit.



In the picture above an example is shown where all members that are being utilized less than 50% are being green coloured, those that are being utilized 50-100% are being blue coloured, and those that don't have enough capacity are being red coloured.

2.3.7 Utilization table

In order to quickly get an opinion of the utilization in different parts of the structure the **Utilization ratio-table** option can be used. A table is displayed, showing the utilization value for all members. The table can be sorted according to utilization or member number.

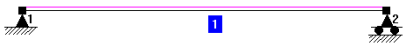


2.4 Option

2.4.1 Lateral supports

It is possible to support the members laterally continuously or with simple supports. If the option **Lateral support** in the menu **Option** is activated this lateral bracing is displayed as shown below:

If a number of members are supported simultaneously the top edge for a horizontal member corresponds to the right edge for a vertical member.

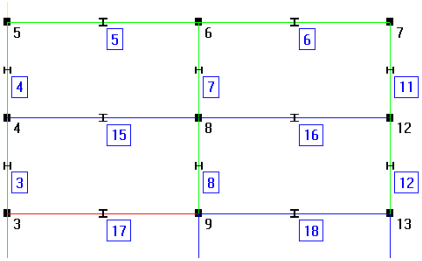


Member continuously supported at the top edge

Member with simple supports at the top edge

2.4.2 Section image

If the option **Section image** in the menu **Option** is activated the section for each member will be displayed as for the figure.



3 Methods of calculation

3.1 Design methods with regard to buckling

The design is being performed according to [1] along with all code prescribed checks depending on section type, section class and loading. Flexural buckling in the frame plane can be performed according to 1th or 2nd order theory. For buckling out of the plane frame 1th order theory is used. The user chooses desired method in **Frame Analysis**.

3.1.1 The First Order Theory

The following are valid when designing according to the first order theory:

The capacity with regard to flexural buckling is calculated according to [1] 6.3 ”Buckling resistance of members”. In the frame plane the buckling length is defined by the user and out of the plane the buckling length is calculated from support conditions defined by the user. The effect of initial bow imperfection is already considered and has not to be defined separately, see EN 1993-1-1 5.3.4.

3.1.2 The Second Order Theory

The second order theory considers the influence of deflection on force and moment distribution within the frame. There is no need to define buckling lengths in the frame plane when using this method since the buckling effect will be considered with the second order moments in the members. The capacity with regard to flexural buckling in the frame plane is shown in the code check window with regard to 2nd order moments. When all members pass the code checks there will be enough buckling safety within the frame plane for each member as well as for the frame as a whole.

Calculations according to the second order theory must have been selected in **Frame Analysis** in order to perform the design in this way in **Steel Module**. When using this method the following demands must be fulfilled:

Consideration shall be taken to initial bow imperfection including residual stresses. Use the tool ”Initial bow imperfection” in **Frame Analysis** to state which members shall have initial curvature due to imperfection. It is particularly important for hinged members loaded with axial force only otherwise there will be no effect from 2nd order moments. Therefore no design is allowed for compression members if the 1th order moment does not correspond to the initial bow imperfection moment.

When working with sway-frames (or isolated columns) consideration must also be taken to initial deviation from the vertical loads of the frame. **Frame Analysis** will not automatically consider this, so the user must choose another method in order to include effect of initial deviation. A suitable method is to add fictive horizontal loads corresponding to the effect of the deviation.

With regard to instability out of the frame plane, also a flexural torsional check is made unless the member is prevented from lateral movement by lateral bracing.

Instability out of the frame plane is also affected from the support conditions defined by the user under the option **Support conditions**. As default it is presumed that all members are hinged at both ends.

3.2 Section values

The section class is calculated according to EN 1993-1-1 5.5.

The program calculates the section class at 20 points along the member and the highest class is selected.

3.3 Axial force capacity

The capacity for members in tension and compression in section class 1, 2 and 3 is calculated according to EN 1993-1-1 6.2, 6.3.1.

For members in section class 4 no calculation is available in the present version of the **Steel Module** 6.2.

The capacity for compressed members is calculated with regard to flexural buckling and torsional buckling for double symmetrical sections and also with regard to flexural-torsional buckling for other sections.

The member can be laterally supported continuously or at certain points in order to increase the capacity with regard to instability out of the frame plane. Most sections can be supported at both top and bottom edge. **L**- and **T**-sections can only be supported at their shear centre and for circular sections no lateral supports can be defined.

3.4 Moment capacity

The capacity is calculated according to EN 1993-1-1 6.2.5, 6.3.2.

3.4.1 Lateral torsional buckling

Lateral torsional buckling is calulated for rectangular massive sections, **I**-sections, rectangular hollow sections and **U**-sections if the height > width.

The reduction factor with regard to lateral torsional buckling is calculated according to EN 1993-1-1 6.3.2.2.

The elastic critical moment for lateral torsional buckling $\mathbf{M_{cr}}$ is calculated as described below.

Elastic critical moment for lateral-torsional buckling $\mathbf{M_{cr}}$

When calculating lateral-torsional buckling it is required to find the critical moment $\mathbf{M_{cr}}$ for arbitrary sections, where the moment distribution and support conditions greatly influences the result. A theoretical solution could be written as:

$$M_{cr} = m / L \left(E I_z \left(G I_T + E I_w (\kappa \pi / L)^2\right)\right)^{0.5}$$

Where **m** is a coefficient depending on the section type, loading, support conditions and load level and $\boldsymbol{\kappa}$ is depending on the support conditions.

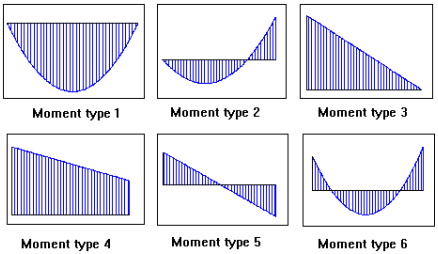
$\mathbf{I_T}$ is the torsional constant and $\mathbf{I_w}$ the warping constant.

It is however in most cases not possible to find a theoretical solution for the critical moment $\mathbf{M_{cr}}$ for arbitrary sections, loading situations and support conditions.

Present solution

In the steel design module routines to calculate the critical moment is used where a method to transfer the current case to a case with a known theoretical solution is the so called "Deflection analogy" where the deflection for the member as simply supported with the current load situation is compared to deflection for a standard load in this case a uniform load. To consider different moment distributions, support conditions and load levels [6] and [7] are used. As the centre of gravity of the load action along the member is important especially when the support conditions differ between the edges, this point is calculated and the critical moment is influenced accordingly.

The following moment types will be used in the calculation:



No consideration is taken to an increased critical moment due to the continuity between members but each member is treated as simply supported, however with regard to the actual moment distribution. Defining fixed support conditions can also increase the critical moment.

Moment types 1,2 and 6

The current load case is transferred to the case uniform distributed load by the deflection analogy. The load level in this case influences the critical moment. The capacity could be increased by fixed support conditions in the weak direction at one or both ends. If only one end is fixed the calculation will also consider the position of the centre of gravity for the current load case with regard to the fixed end.

For moment type 2, if only one end is fixed the critical moment will differ depending on if the support moment is situated at the fixed end or not.

For moment type 2 and 6 the critical moment is influenced by the relation between the magnitudes of the positive and the negative moments.

Moment types 3,4 and 5

For these types the calculation is made for a linear moment distribution, where the relation between the magnitudes of the end moments highly influences the result. The critical moment could be increased by fixed support conditions in the weak direction at one or both ends. If only one end is fixed the capacity will differ depending on if the largest moment is situated at the fixed end or not.

For a cantilever the current load case is transferred for moment type 3 to the case uniform distributed load by the deflection analogy. The load level will in this case influence the critical moment.

For moment type 4 the calculation for a cantilever will be performed as for a member hinged at both ends with double length.

The present routines can handle symmetrical and single symmetrical sections i.e. most of the sections used.

3.5 Shear capacity

The capacity is calculated according to EN 1993-1-1 6.2.6, EN 1993-1-5 5.1-5.3 and for circular tubes according to EN 1993-1-6 Annex D.

3.5.1 Buckling stiffeners

For open sections with slender webs the web capacity can be increased with buckling stiffeners. The capacity is then calculated according to EN 1993-1-5 5.1-5.3.

3.6 Interaction Equations

3.6.1 Axial force

3.6.1.1 2:nd order calculation

This case will not occur as moments $\mathbf{M_y}$ or $\mathbf{M_z}$ at least corresponding to initial bow imperfections always will be calculated if a design should be allowed!

3.6.1.2 1:th order calculation

A stress check is made according to EN 1993-1-1 6.2.3 and a Flexural buckling check is made according to EN 1993-1-1 6.3.1.

3.6.2 Bending moment and shear force

The bending check is performed according to EN 1993-1-1 6.2.5, the shear check according to 6.2.6 and the possible bending and shear check according to EN 1993-1-1 6.2.8 and EN 1993-1-5 7.1. The possible lateral torsional buckling check is performed according to EN 1993-1-1 6.3.2. A possible shear buckling check is made according to EN 1993-1-5-5.

3.6.3 Axial force, bending moment and shear force

3.6.3.1 1:th order calculation

The maximum stress check is made according to EN 1993-1-1 6.2.1, 6.2.8, 6.2.9 and EN 1993-1-5 7.1, the instability checks according to EN 1993-1-1 6.3.2, the shear check according to 6.2.6 and a possible shear buckling check according to EN 1993-1-5-5.

The interaction factors $\mathbf{k_{ij}}$ used in the instability checks are calculated according to EN 1993-1-1 annex A.

3.6.3.2 2:nd order calculation

A maximum stress check is made according to EN 1993-1-1 6.2.1, 6.2.8, 6.2.9 and EN 1993-1-5 7.1. The flexural buckling check in the frame plane is made according to EN 1993-1-1 6.2.1 with 2nd order moments. The instability check out of the frame plane is made according to EN 1993-1-1 6.3.2, the shear check according to 6.2.6 and a possible shear buckling check according to EN 1993-1-5-5.

4 References

1. EN 1993-1-1 General rules and rules for buildings.
2. EN 1993-1-3 Supplementary rules for cold-formed members and sheeting.
3. EN 1993-1-4 Supplementary rules for stainless steels.
4. EN 1993-1-5 Plated structural element.
5. EN 1993-1-6 Strength and stability of shell structures.
6. Statik und Stabilität der Baukonstruktion, Petersen, 1982.
7. Knäckning, Vippning, Buckling, StBK-K2, 1973.
8. The Behaviour and Design of Steel Structures, Trahair and Bradford 1991.
9. Handboken Bygg Allmänna grunder, Avsnitt A30, Stockholm, 1985.