1 Contents

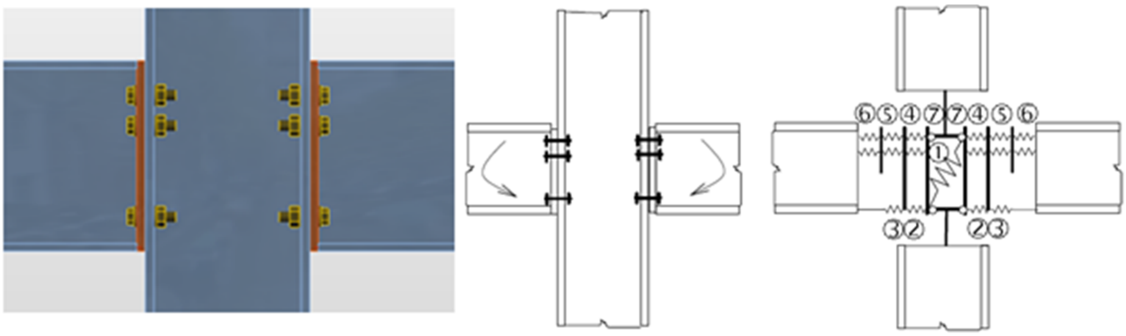
General

1 Introduction

Bar members are preferred by engineers when designing steel structures. However, there are many locations on the structure where the theory of members is not valid, e.g., welded joints, bolted connections, footing, holes in walls, the tapering height of cross-section and point loads. The structural analysis in such locations is difficult and it requires special attention. The behavior is non-linear and the nonlinearities must be respected, e.g., yielding of the material of plates, contact between end plates or base plate and concrete block, one-sided actions of bolts and anchors, welds. Design codes, e.g. EN1993-1-8, and also technical literature offer engineering solution methods. Their general feature is derivation for typical structural shapes and simple loadings. The method of components is used very often.

Component method

Component method (CM) solves the joint as a system of interconnected items – components. The corresponding model is built per each joint type to be able to determine forces and stresses in each component – see the following picture.



1 – column web in shear, 2 – column web in compression, 3 – beam flange and web in compression, 4 - column flange in bending, 5 – bolts in tension, 6 – end plate in bending and 7 – column web in tension.

The components of a joint with bolted end plates modeled by springs

Each component is checked separately using corresponding formulas. As the proper model must be created for each joint type, the method usage has limits when solving joints of general shapes and general loads.

IDEA StatiCa together with a project team of Department of Steel and Timber Structures of Faculty of Civil Engineering in Prague and Institute of Metal and Timber Structures of Faculty of Civil Engineering of Brno University of Technology developed a new method for advanced design of steel structural joints.

The new **Component Based Finite Element Model** (CBFEM) method is:

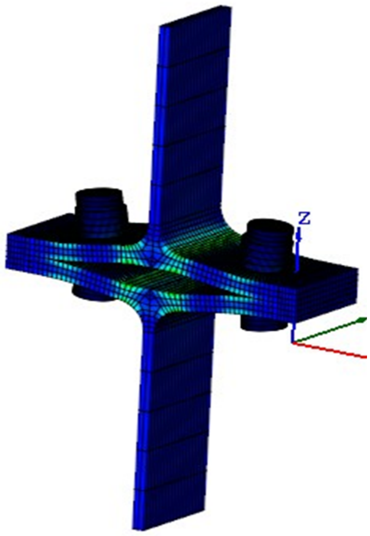
- **General** enough to be usable for most of joints, footings and details in engineering practice.
- **Simple and fast** enough in daily practice to provide results in a time comparable to current methods and tools.
- **Comprehensive** enough to provide structural engineer clear information about joint behavior, stress, strain and reserves of individual components and about overall safety and reliability.

The CBFEM method is based on the idea that the most of the verified and very useful parts of CM should be kept. The weak point of CM – its generality when analyzing stresses of individual components – was replaced by modeling and analysis using Finite Element Method (FEM).

2 CBFEM components

FEM is a general method commonly used for structural analysis. Usage of FEM for modeling of joints of any shapes seems to be ideal (Virdi, 1999). The elastic-plastic analysis is required, as the steel ordinarily yields in the structure. In fact, the results of the linear analysis are useless for joint design.

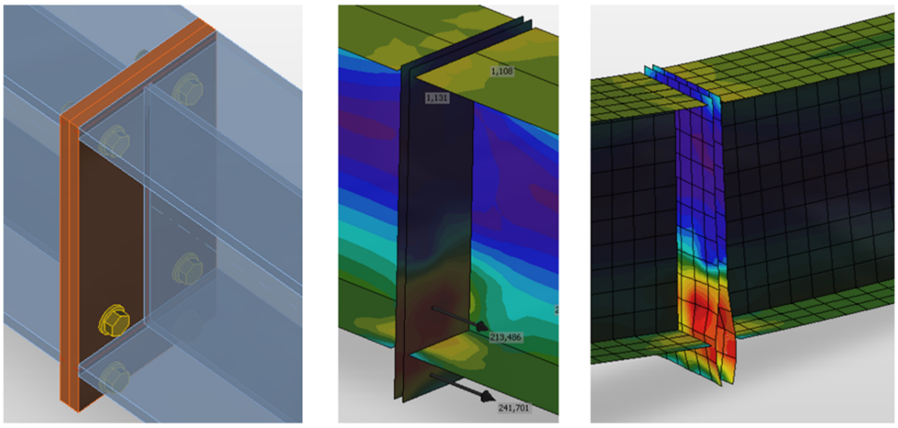
FEM models are used for research purposes of joint behavior, which usually apply spatial elements and measured values of material properties.



FEM model of a joint for research. It uses spatial 3D elements for both plates and bolts

Both webs and flanges of connected members are modeled using shell elements in CBFEM model for which the known and verified solution is available.

The fasteners – bolts and welds – are the most difficult in the point of view of the analysis model. Modeling of such elements in general FEM programs is difficult because the programs do not offer required properties. Thus, special FEM components had to be developed to model the welds and bolts behavior in a joint.

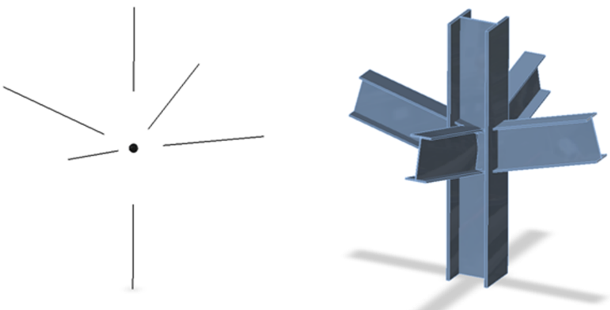


CBFEM model of bolted connection by end plates

Joints of members are modeled as massless points when analyzing steel frame or girder structure. Equilibrium equations are assembled in joints and internal forces on ends of beams are determined after solving the whole structure. In fact, the joint is loaded by those forces. The resultant of forces from all members in the joint is zero – the whole joint is in equilibrium.

The real shape of a joint is not known in the structural model. The engineer only defines whether the joint is assumed to be rigid or hinged.

It is necessary to create the trustworthy model of joint, which respect the real state, to design the joint properly. The ends of members with the length of a 2-3 multiple of maximal cross-section height are used in the CBFEM method. These segments are modeled using shell elements.



A theoretical (massless) joint and a real shape of the joint without modified member ends

For better precision of CBFEM model, the end forces on 1D members are applied as loads on the segment ends. Sextuplets of forces from the theoretical joint are transferred to the end of segment – the values of forces are kept, but the moments are modified by the actions of forces on corresponding arms.

The segment ends at the joint are not connected. The connection must be modeled. So-called manufacturing operations are used in the CBFEM method to model the connection. Manufacturing operations are especially: cuts, offsets, holes, stiffeners, ribs, end plates and splices, angles, gusset plates and other. Fastening elements (welds and bolts) are also added.

IDEA StatiCa Connection can perform two types of analysis:

1. Geometrically linear analysis with material and contact nonlinearities for stress and strain analysis,
2. Eigenvalue analysis to determine the possibility of buckling.

In the case of connections, the geometrically nonlinear analysis is not necessary unless plates are very slender. Plate slenderness can be determined by eigenvalue (buckling) analysis. For the limit slenderness where geometrically linear analysis is still sufficient, see Chapter 3.9. The geometrically nonlinear analysis is not implemented in the software.

2.1 Material model

The most common material diagrams which are used in finite element modeling of structural steel are the ideal plastic or elastic model with strain hardening and the true stress-strain diagram. The true stress-strain diagram is calculated from the material properties of mild steels at an ambient temperature obtained in tensile tests. The true stress and strain may be obtained as follows:

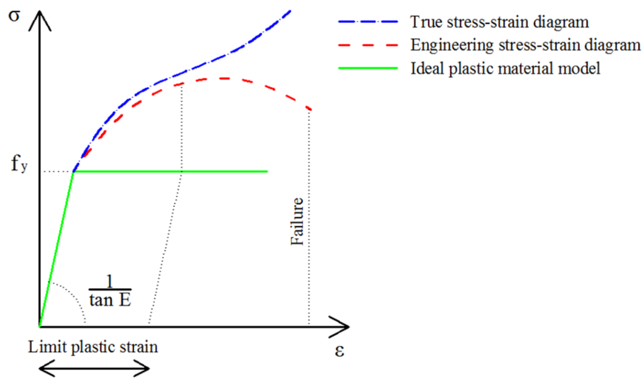
$$\sigma_{true} = \sigma(1 + \epsilon)$$

$$\epsilon_{true} = \ln(1 + \epsilon)$$

where σ_{true} is true stress, $\varepsilon_{\mathrm{true}}$ true strain, σ engineering stress and ε engineering strain.

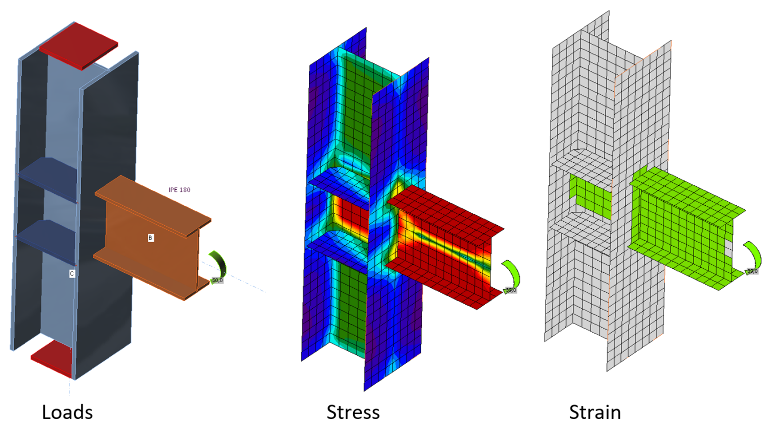
The plates in IDEA StatiCa Connection are modeled with elastic-plastic material with a nominal yielding plateau slope according to EN1993-1-5, Par. C.6, (2). The material behavior is based on von Mises yield criterion. It is assumed to be elastic before reaching the yield strength, f_{y}.

The ultimate limit state criterion for regions not susceptible to buckling is reaching the limiting value of the principal membrane strain. The value of 5 % is recommended (e.g. EN1993-1-5, App. C, Par. C.8, Note 1).

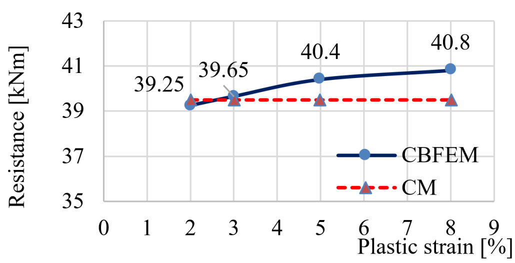


Material diagrams of steel in numerical models

The limit value of plastic strain is often discussed. In fact, the ultimate load has low sensitivity to the limit value of plastic strain when the ideal plastic model is used. It is demonstrated on the following example of a beam to column joint. An open section beam IPE 180 is connected to an open section column HEB 300 and loaded by bending moment. The influence of the limit value of plastic strain on the resistance of the beam is shown in the following figure. The limit plastic strain changes from 2 % to 8 %, but the change in moment resistance is less than 4 %.



An example of prediction of ultimate limit state of a beam to column joint



The influence of the limit value of plastic strain on the moment resistance

2.2 Plate model and mesh convergence

2.2.1 Plate model

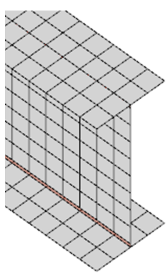
Shell elements are recommended for modeling of plates in FEA of structural connection. 4-node quadrangle shell elements with nodes at its corners are applied. Six degrees of freedom are considered in each node: 3 translations (u_x, u_y, u_z) and 3 rotations (φ_x, φ_y, φ_z). Deformations of the element are divided into the membrane and the flexural components.

The formulation of the membrane behavior is based on the work by Ibrahimbegovic (1990). Rotations perpendicular to the plane of the element are considered. Complete 3D formulation of the element is provided. The out-of-plane shear deformations are considered in the formulation of the flexural behavior of an element based on Mindlin hypothesis. The MITC4 elements are applied, see Dvorkin (1984). The shell is divided into five integration points along the height of the plate and plastic behavior is analyzed in each point. It is called Gauss–Lobatto integration. The nonlinear elastic-plastic stage of material is analyzed in each layer based on the known strains.

2.2.2 Mesh convergence

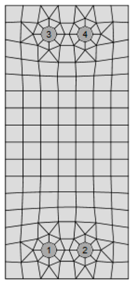
There are some criteria for the mesh generation in the connection model. The connection check should be independent of the element size. Mesh generation on a separate plate is problem-free. The attention should be paid to complex geometries such as stiffened panels, T-stubs and base plates. The sensitivity analysis considering mesh discretization should be performed for complicated geometries.

All plates of a beam cross-section have a common division into elements. The size of generated finite elements is limited. The minimal element size is set to 10 mm and the maximal element size to 50 mm (can be set in Code setup). Meshes on flanges and webs are independent of each other. The default number of finite elements is set to 8 elements per cross-section height as shown in the following figure. The user can modify the default values in Code setup.



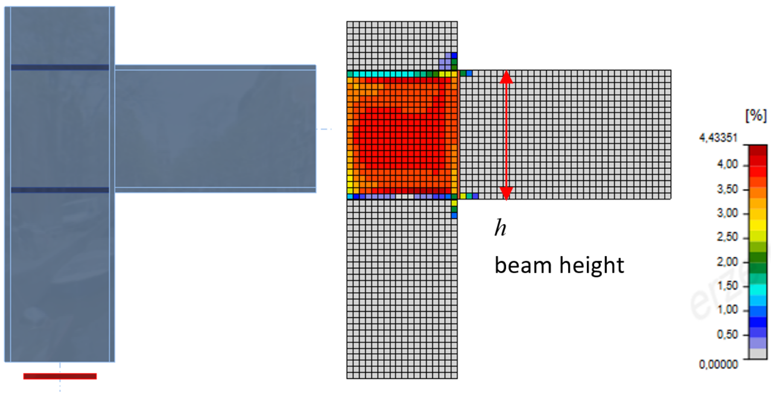
The mesh on a beam with constraints between the web and the flange plate

The mesh of the end plates is separate and independent of other connection parts. Default finite element size is set to 16 elements per cross-section height as shown in the figure.

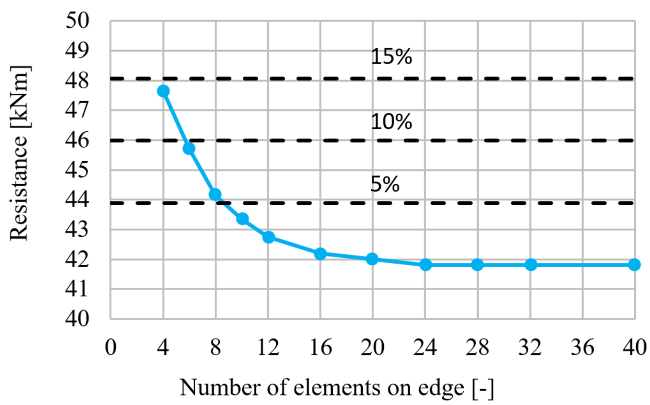


The mesh on an end plate with 7 elements along its width

The following example of a beam to column joint shows the influence of mesh size on the moment resistance. An open section beam IPE 220 is connected to an open section column HEA 200 and loaded by a bending moment as shown in the following figure. The critical component is the column panel in shear. The number of the finite elements along the cross-section height varies from 4 to 40 and the results are compared. Dashed lines are representing the 5%, 10% and 15% difference. It is recommended to subdivide the cross-section height into 8 elements.

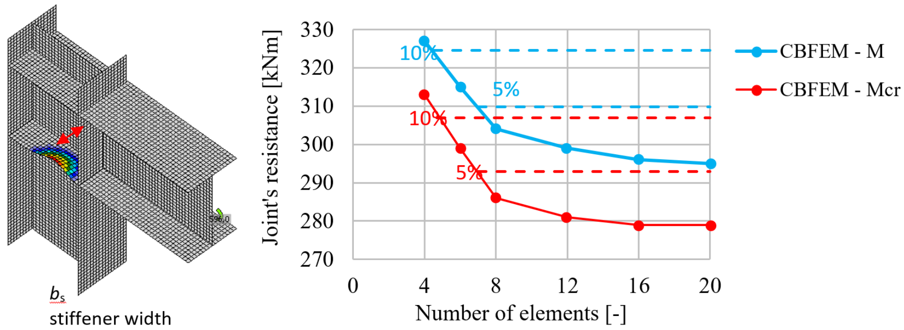


A beam to column joint model and plastic strains at ultimate limit state



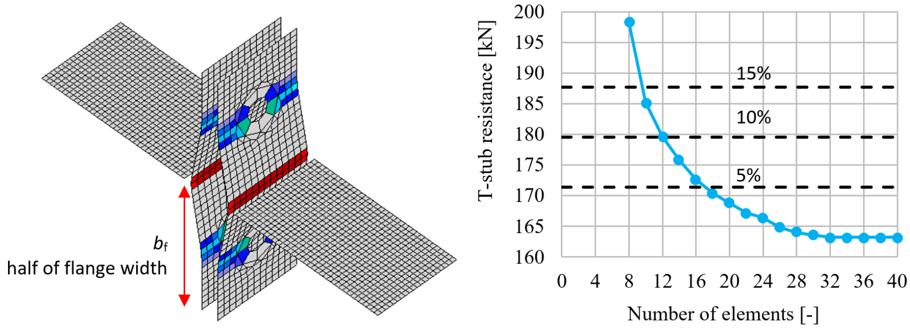
The influence of number of elements on the moment resistance

The mesh sensitivity study of a slender compressed stiffener of column web panel is presented. The number of elements along the width of the stiffener varies from 4 to 20. The first buckling mode and the influence of a number of elements on the buckling resistance and critical load are shown in the following figure. The difference of 5% and 10% are displayed. It is recommended to use 8 elements along the stiffener width.



The first buckling mode and the influence of number of elements along the stiffener on the moment resistance

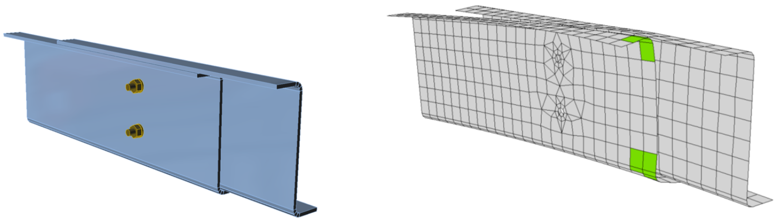
The mesh sensitivity study of a T-stub in tension is presented. The half of the flange width is subdivided into 8 to 40 elements and the minimal element size is set to 1 mm. The influence of the number of elements on the T-stub resistance is shown in the following figure. The dashed lines are representing the 5%, 10% and 15% difference. It is recommended to use 16 elements on the half of the flange width.



The influence of the number of elements on the T-stub resistance

2.3 Contacts

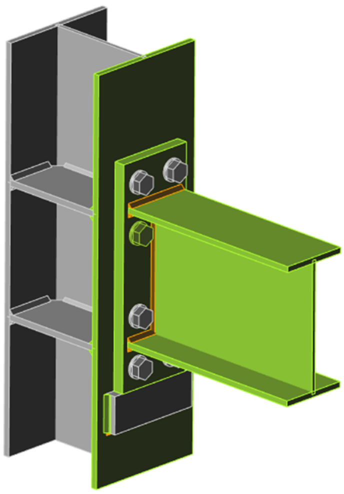
The standard penalty method is recommended for modeling of a contact between plates. If penetration of a node into an opposite contact surface is detected, penalty stiffness is added between the node and the opposite plate. The penalty stiffness is controlled by heuristic algorithm during the nonlinear iteration to get a better convergence. The solver automatically detects the point of penetration and solves the distribution of contact force between the penetrated node and nodes on the opposite plate. It allows creating the contact between different meshes as shown. The advantage of the penalty method is the automatic assembly of the model. The contact between the plates has a major impact on the redistribution of forces in connection.



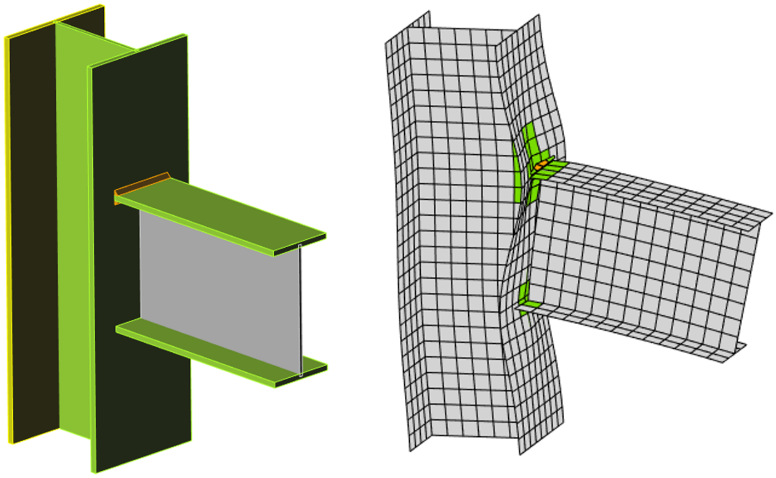
An example of separation of plates in contact between the web and flanges of two overlapped Z sections purlins

It is possible to add contact between

- two surfaces,
- two edges,
- edge and surface.



An example of edge to edge contact between the seat and the end plate



An example of edge to surface contact between the lower flange of the beam and the column flange

2.4 Welds

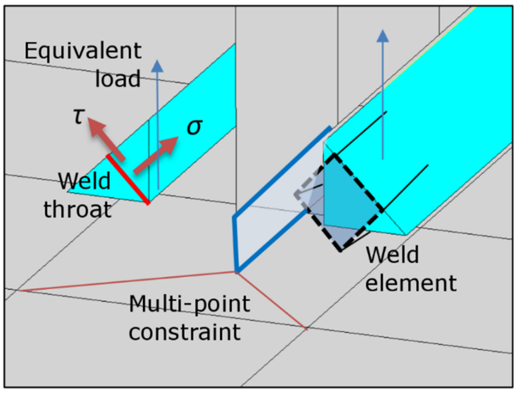
There exist several options how to treat welds in numerical models. The large deformations make the mechanical analysis more complex and it is possible to use different mesh descriptions, different kinetic and kinematic variables and constitutive models. The different types of geometric 2D and 3D models and thereby finite elements with their applicability for different accuracy levels are generally used. Most often used material model is the common rate-independent plasticity model based on von Mises yield criterion. Two approaches which are used for welds are described. Residual stress and deformation caused by welding is not assumed in the design model.

The load is transmitted through force-deformation constraints based on the Lagrangian formulation to opposite plate. The connection is called multi-point constraint (MPC) and relates the finite element nodes of one plate edge to another. The finite element nodes are not connected directly. The advantage of this approach is the ability to connect meshes with different densities. The constraint allows to model midline surface of the connected plates with the offset, which respects the real weld configuration and throat thickness. The load distribution in the weld is derived from the MPC, so the stresses are calculated in the throat section. This is important for the stress distribution in the plate under the weld and for modeling of T-stubs.

2.4.1 Plastic stress redistribution in welds

The model with only multi-point constraints does not respect the stiffness of the weld and the stress distribution is conservative. Stress peaks which appear at the end of plate edges, in corners and rounding, govern the resistance along the whole length of the weld. To eliminate the effect, a special elastoplastic element is added between the plates. The element respects the weld throat thickness, position and orientation. The equivalent weld solid is inserted with the corresponding weld dimensions. The nonlinear material analysis is applied and elastoplastic behavior in equivalent weld solid is determined. The plasticity state is controlled by stresses in the weld throat section. The stress peaks are redistributed along the longer part of the weld length.

Elastoplastic model of welds gives real values of stress and there is no need to average or interpolate the stress. Calculated values at the most stressed weld element are used directly for checks of weld component. This way, there is no need to reduce the resistance of multi-oriented welds, welds to unstiffened flanges or long welds.



Constraint between weld element and mesh nodes

General welds, while using plastic redistribution, can be set as continuous, partial and intermittent. Continuous welds are over the whole length of the edge, partial allows user to set offsets from both sides of the edge, and intermittent welds can be additionally set with a set length and a gap.

2.5 Bolts

In the Component Based Finite Element Method (CBFEM), bolt with its behavior in tension, shear and bearing is the component described by the dependent nonlinear springs. The bolt in tension is described by spring with its axial initial stiffness, design resistance, initialization of yielding and deformation capacity. The axial initial stiffness is derived analytically in the guideline VDI2230. The model corresponds to experimental data, see Gödrich et al. (2014). For initialization of yielding and deformation capacity, it is assumed that plastic deformation occurs in the threaded part of the bolt shank only. The force at beginning of yielding, $F_{y,ini}$, is

$F_{y,ini} = f_{y,b} A_t$

where $f_{y,b}$ is the yield strength of bolts and A_t the tensile area of the bolt. Relation gives higher values for materials with low ratio of the ultimate strength to yield strength than the design resistance, $F_{t,Rd}$. To assure a positive value of plastic stiffness, it should be taken

$F_{y,ini} \leq F_{t,Rd}$

Deformation capacity of the bolt, δ_c, consists of elastic deformation of bolt shank δ_{el} and plastic one of the threaded part only δ_{pl}.

$\delta_c = \delta_{el} + \delta_{pl}$

$\delta_{el} = F_{t,Rd} / k_{ini}$

where k_{ini} is the initial deformation stiffness of the bolt in tension according to the guideline VDI2230, and

$\delta_{pl} = \varepsilon_{pl} \; l_t$

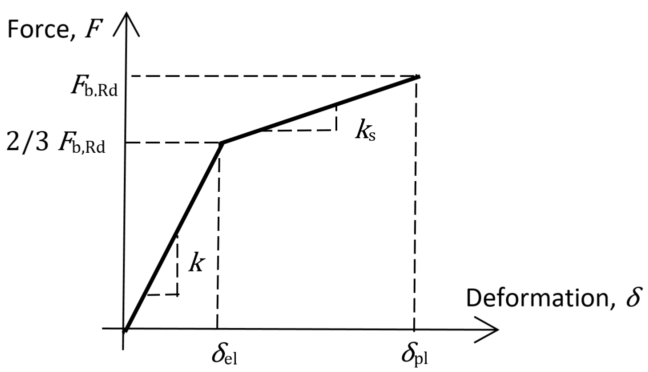
where ε_{pl} is the limit plastic strain given by the value of 5% and l_t is the length of the threaded part. The tensile force is transmitted to the plates by interpolation links between the bolt shank and nodes in the plate. The transfer area corresponds to the mean value of the bolt shank and the circle inscribed in the hexagon of the bolt head.

Deformation capacity is considered according to Wald et al. (2002) as

$\delta_{pl} = 3\ \delta_{el}$

Initialization of yielding is expected at (see the following figure)

$F_{ini} = 2 / 3\ F_{b,Rd}$

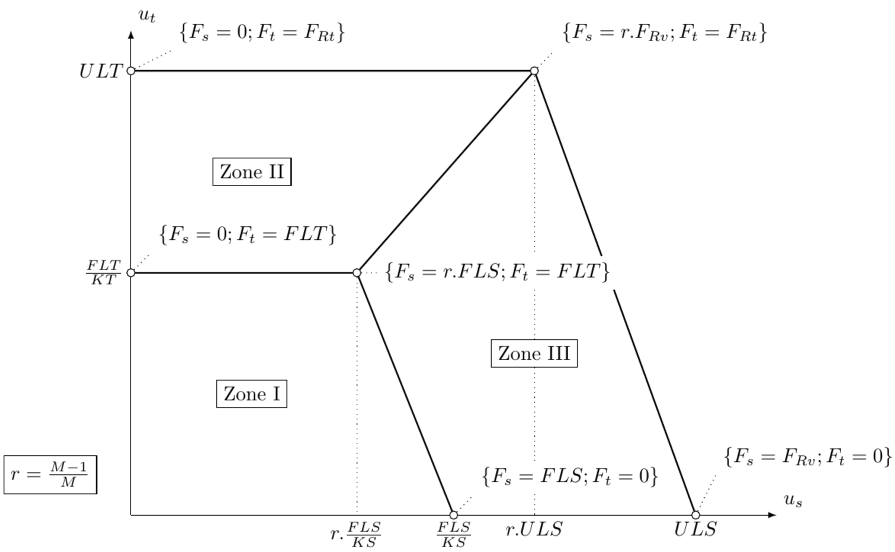


Force-deformation diagram for bearing of the plate

Only the compression force is transferred from the bolt shank to the plate in the bolt hole. It is modeled by interpolation links between the shank nodes and holes edge nodes. The deformation stiffness of the shell element modeling the plates distributes the forces between the bolts and simulates the adequate bearing of the plate.

Bolt holes are considered as standard (default) or slotted (can be set in plate editor). Bolts in standard holes can transfer shear force in all directions, bolts in slotted holes have one direction excluded and can move in this selected direction freely.

Interaction of the axial and the shear force can be introduced directly in the analysis model. Distribution of forces reflects the reality better (see enclosed diagram). Bolts with a high tensile force take less shear force and vice versa.



Example of interaction of axial and shear force (EC)

2.6 Preloaded bolts

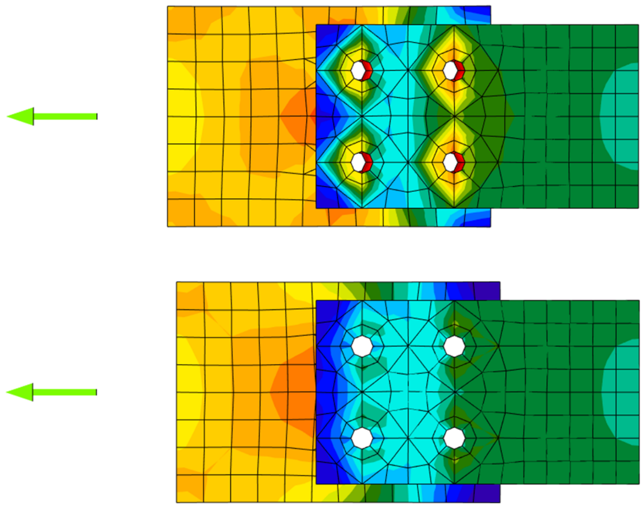
Preloaded bolts are used in cases when minimization of deformation is needed. The tension model of a bolt is the same as for standard bolts. The shear force is not transferred via bearing but via friction between gripped plates.

The design slip resistance of a preloaded bolt is affected by an applied tensile force.

IDEA StatiCa Connection checks the pre-slipping limit state of preloaded bolts. If there is a slipping effect, bolts do not satisfy the check. Then the post-slipping limit state should be checked as a standard bearing check of bolts where bolt holes are loaded in bearing and bolts in shear.

The user can decide which limit state will be checked. Either it is resistance to major slip or post-slipping state in shear of bolts. Both checks on one bolt are not combined in one solution. It is assumed that bolt has a standard behaviour after a major slip and can be checked by the standard bearing procedure.

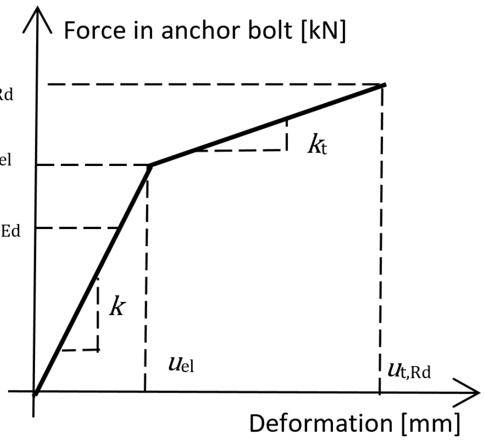
The moment load of connection has a small influence on the shear capacity. Nevertheless, a friction check on each bolt simply is solved separately. This check is implemented in FEM component of the bolt. There is no information in a general way whether the external tension load of each bolt is from the bending moment or from the tension load of connection.



Stress distribution in standard and slip-resistant shear bolt connection

2.7 Anchor bolts

The anchor bolt is modeled with the similar procedures as the structural bolts. The bolt is fixed on one side to the concrete block. Its length, L_b, used for bolt stiffness calculation is taken as a sum of half of the nut thickness, washer thickness, t_w, base plate thickness, t_{bp}, grout or gap thickness, t_g, and free the length embedded in concrete which is expected as 8*d* where *d* is a bolt diameter. The factor 8 is editable in Code setup. This value is in accordance with the Component Method (EN1993-1-8); the free length embedded in concrete can be modified in Code setup. The stiffness in tension is calculated as $k = E\,A_s / L_b$. The load–deformation diagram of the anchor bolt is shown in the following figure. The values according to ISO 898:2009 are summarized in the table and in formulas below.



Load–deformation diagram of the anchor bolt

$$F_{t,el} = \frac{F_{t,Rd}}{c_1 c_2 - c_1 + 1}$$

$$k_t = c_1 k; \qquad c_1 = \frac{R_m - R_e}{\frac{1}{4} A - \frac{R_e}{E} E}$$

$$u_{el} = \frac{F_{t,el}}{k}; \qquad u_{t,Rd} = c_2 u_{el}; \qquad c_2 = \frac{AE}{4R_e}$$

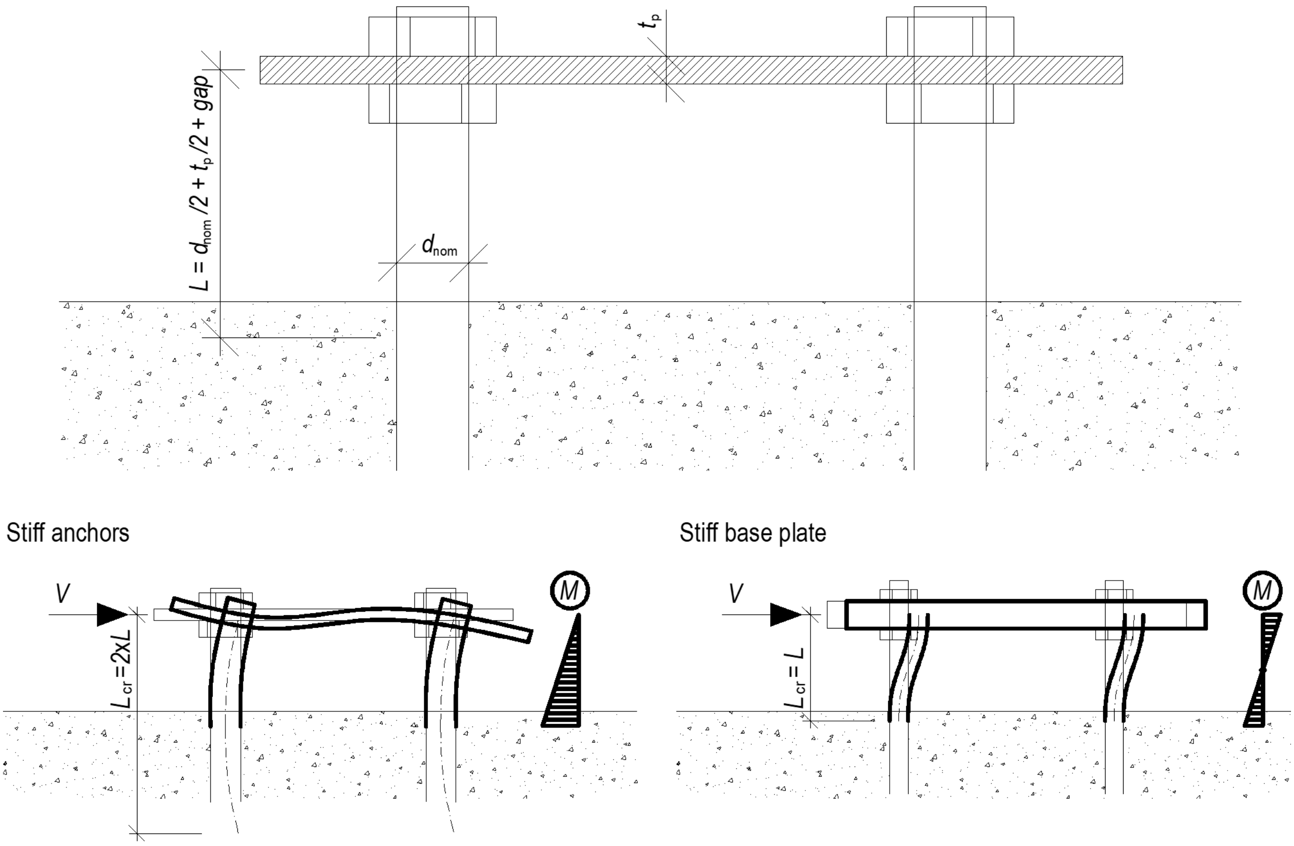
where:

- A – elongation
- E – Young's modulus of elasticity
- $F_{t,Rd}$ – steel tensile resistance of anchor
- R_m – ultimate (tensile) strength
- R_e – yield strength

The stiffness of the anchor bolt in shear is taken as the stiffness of the structural bolt in shear.

2.7.1 Anchor bolts with stand-off

Anchors with stand-off can be checked as a construction stage before the column base is grouted or as a permanent state. Anchor with stand-off is designed as a bar element loaded by shear force, bending moment and compressive or tensile force. These internal forces are determined by finite element model. The anchor is fixed on both sides, one side is 0.5×d below the concrete level, the other side is in the middle of the thickness of the plate. The buckling length is conservatively assumed as twice the length of the bar element. Plastic section modulus is used. The forces in anchor with stand-off are determined using finite element analysis. Bending moment is dependent on the stiffness ratio of anchors and base plate.



Anchors with stand-off – determination of lever arm and buckling lengths; stiff anchors are safe assumption

2.8 Concrete block

2.8.1 Design model

In CBFEM, it is convenient to simplify the concrete block as 2D contact elements. The connection between the concrete and the base plate resists in compression only. Compression is transferred via Winkler-Pasternak subsoil model which represents deformations of the concrete block. The tension force between the base plate and concrete block is carried by the anchor bolts. The shear force is transferred by friction between a base plate and a concrete block, by shear key and by bending of anchor bolts and friction. The resistance of bolts in shear is assessed analytically. Friction and shear key are modeled as a full single point constraint in the plane of the base plate – concrete contact.

2.8.2 Deformation stiffness

The stiffness of the concrete block may be predicted for the design of column bases as an elastic hemisphere. A Winkler-Pasternak subsoil model is commonly used for a simplified calculation of foundations. The stiffness of subsoil is determined using modulus of elasticity of concrete and the effective height of a subsoil as:

$$k = \frac{E_c}{(\alpha_1 + \upsilon)\sqrt{\frac{A_{eff}}{A_{ref}}}} \left(\frac{1}{\frac{h}{a_2 d} + a_3} + a_4 \right)$$

where:

- k – stiffness of concrete subsoil in compression
- E_c – modulus of elasticity of concrete
- υ – Poisson's coefficient of the concrete block
- A_{eff} – effective area in compression
- A_{ref} = 10 m^2 – reference area
- d – base plate width
- a_1 = 1.65; a_2 = 0.5; a_3 = 0.3; a_4 = 1.0 – coefficients

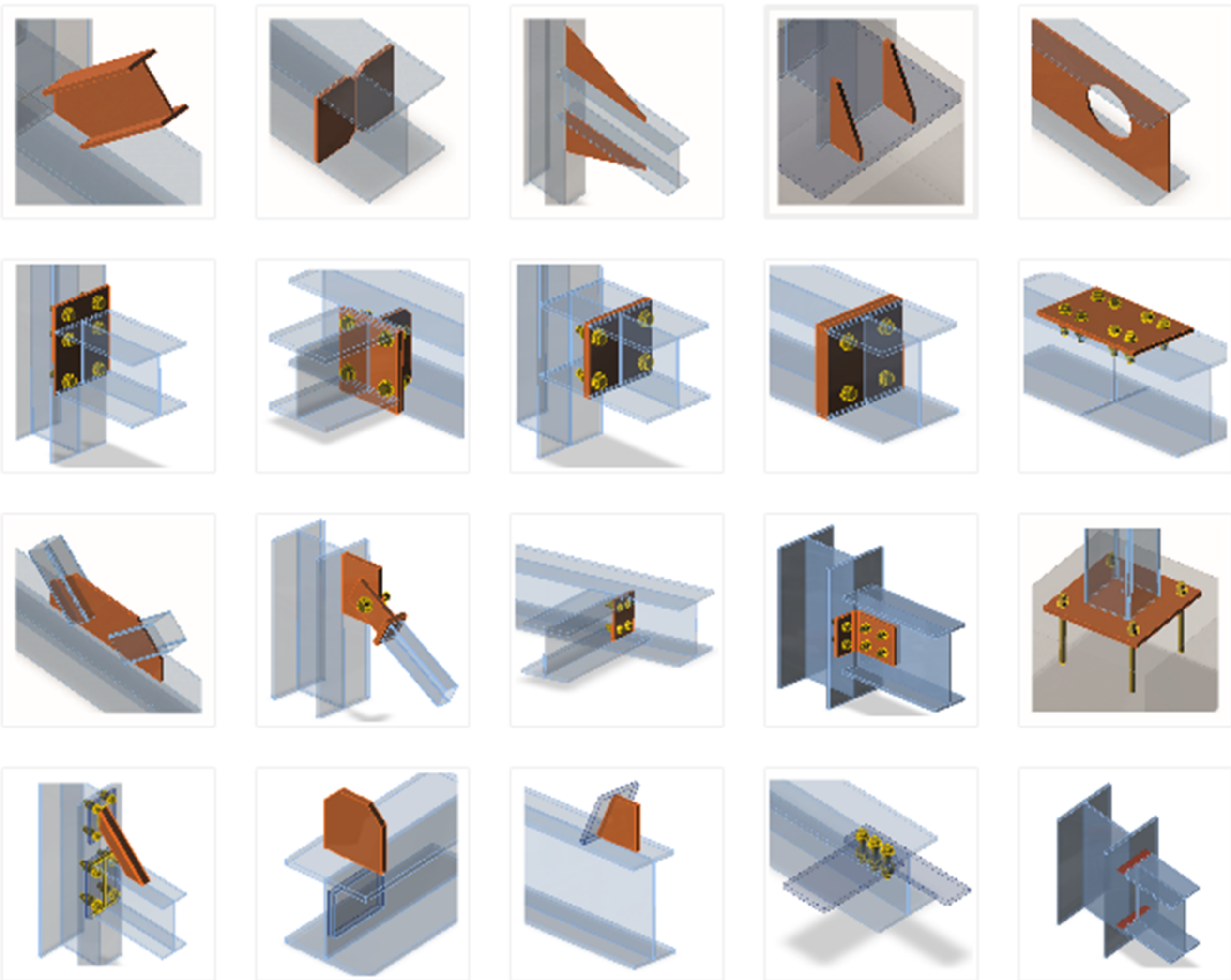
SI units must be used in the formula, the resulting unit is N/m^3.

3 Analysis

3.1 Analysis model

The newly developed method (CBFEM – Component Based Finite Element Model) enables fast analysis of joints of several shapes and configurations. The model consists of members, to which the load is applied, and manufacturing operations (including stiffening members), which serve to connect members to each other. Members must not be confused with manufacturing operations because their cut edges are connected via rigid links to the connection node so they are not deformed properly if used instead of manufacturing operations (stiffening members).

The analyzed FEM model is generated automatically. The designer does not create the FEM model, he creates the joint using manufacturing operations – see the figure.



Manufacturing operations/items which can be used to construct the joint

Each manufacturing operation adds new items to the connection – cuts, plates, bolts, welds.

3.2 Bearing member and supports

One member of the joint is always set as “bearing”. All other members are “connected”. The bearing member can be chosen by the designer. The bearing member can be “continuous” or “ended” in the joint. “Ended” members are supported on one end, “continuous” members are supported on both ends.

Connected members can be of several types, according to the load, which the member can take:

- Type N-Vy-Vz-Mx-My-Mz – member is able to transfer all 6 components of internal forces
- Type N-Vy-Mz – member is able to transfer only loading in XY plane – internal forces N, V_y, M_z
- Type N-Vz-My – member is able to transfer only loading in XZ plane – internal forces N, V_z, M_y
- Type N-Vy-Vz – member is able to transfer only normal force N and shear forces V_y and V_z

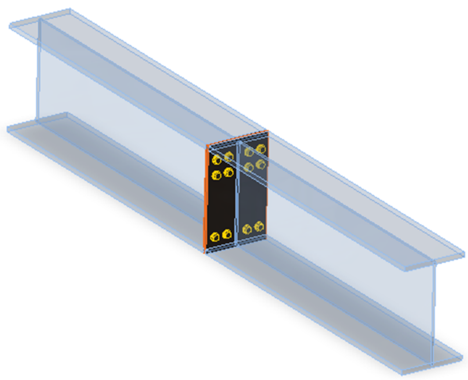
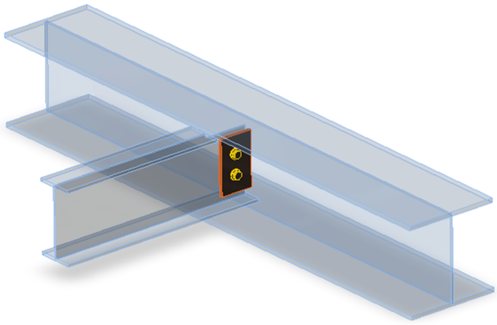
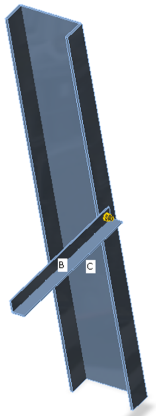


Plate to plate connection transfers all components of internal forces



Fin plate connection can transfer only loads in XZ plane – internal forces *N*, V_z, M_y



Gusset connection – connection of truss member can transfer only axial force N and shear forces Vy and Vz

Each joint is in the state of equilibrium during the analysis of the frame structure. If the end forces of the individual members are applied to detailed CBFEM model, the state of equilibrium is met too. Thus, it would not be necessary to define supports in analysis model. However, for practical reasons, the support resisting all translations is defined in the first end of the bearing member. It does influence neither the state of stress nor the internal forces in the joint, only the presentation of deformations.

Appropriate support types respecting the type of the individual members are defined at the ends of the connected members to prevent the occurrence of unstable mechanisms.

The default length of each member is twice its height. The length of a member should be at least 1× the height of the member after the last manufacturing operation (weld, opening, stiffener etc.) due to the correct deformations after the rigid links connecting the cut end of a member to the connection node.

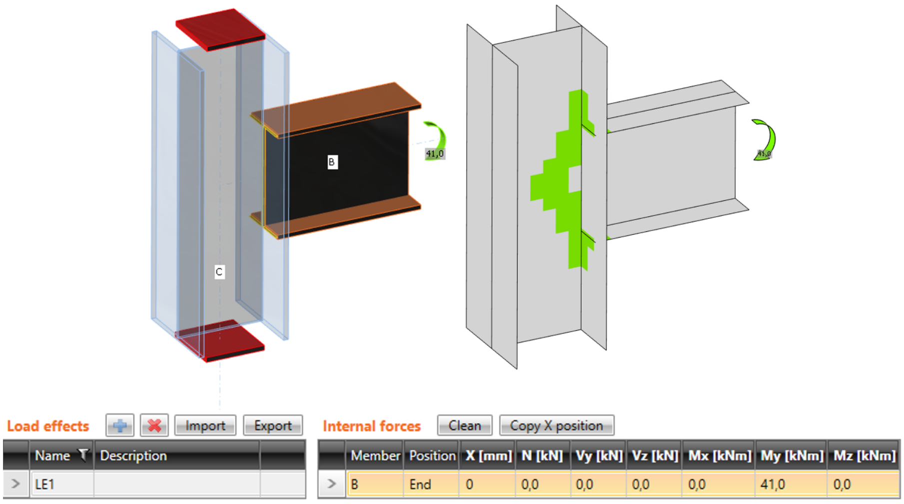
3.3 Equilibrium in node

Each node of the 3D FEM model must be in equilibrium. The equilibrium requirement is correct, nevertheless, it is not necessary for the design of simple joints. One member of the joint is always „bearing“ and the others are connected. If only the connection of connected members is checked, it is not necessary to keep the equilibrium. Thus, there are two modes of loads input available:

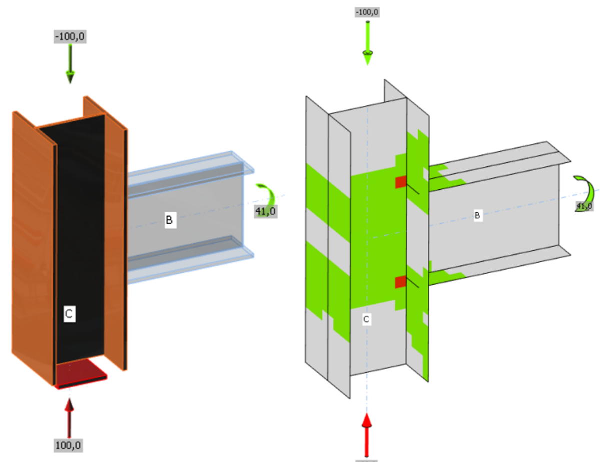
- **Simplified** – for this mode, the bearing member is supported (continuous member on both sides) and the load is not defined on the member
- **Advanced** (exact with equilibrium check) – the bearing member is supported on one end, the loads are applied to all members and the equilibrium has to be found

The mode can be switched in the ribbon group **Loads in equilibrium**.

The difference between the modes is shown in the following example of T-connection. The beam is loaded by the end bending moment of 41 kNm. There is also a compressive normal force of 100 kN in the column. In the case of simplified mode, the normal force is not taken into account because the column is supported on both ends. The program shows only the effect of bending moment of the beam. Effects of normal force are analyzed only in the full mode and they are shown in results.



Simplified input: normal force in column is NOT taken into account



Load effects | Import | Export

Name	Description
LE1	

Internal forces | Clean | Copy X position

Member	Position	X [mm]	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
C	Begin	0	100,0	0,0	0,0	0,0	0,0	0,0
C	End	0	-100,0	0,0	0,0	0,0	0,0	0,0
B	End	0	0,0	0,0	0,0	0,0	41,0	0,0

Unbalanced forces

X [kN]	Y [kN]	Z [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
0,0	0,0	0,0	0,0	41,0	0,0

Advanced input: normal force in column is taken into account

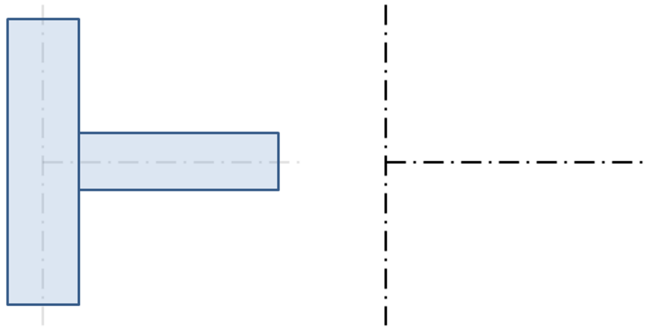
The simplified method is easier for the user but it can be used only when the user is interested in studying connection items and not the behavior of the whole joint.

For cases where the bearing member is heavily loaded and close to its limit capacity, the advanced mode with respecting all the internal forces in the joint is necessary.

3.4 Loads

The end forces of a member of the frame analysis model are transferred to the ends of member segments. Eccentricities of the members caused by the joint design are respected during transfer.

The analysis model created by CBFEM method corresponds to the real joint very precisely, whereas the analysis of internal forces is performed on much idealized 3D FEM bar model, where individual beams are modeled using center lines and the joints are modeled using immaterial nodes.

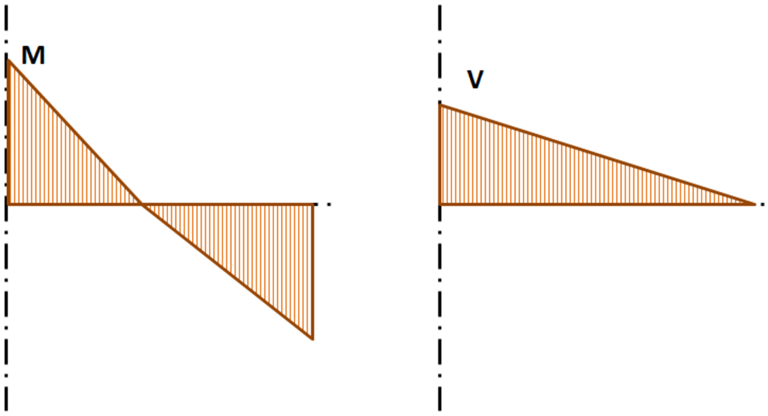


Real shape of joint

Theoretical shape in 3D FEM model

Joint of a vertical column and a horizontal beam

The internal forces are analyzed using 1D members in the 3D model. There is an example of the internal forces in the following figure.

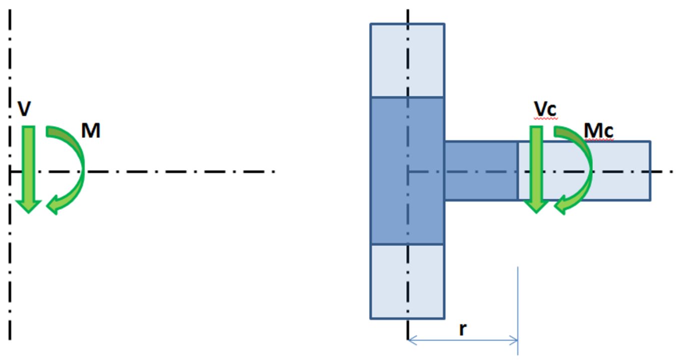


Bending moment

Shear force

Internal forces in horizontal beam. M and V are the end forces at joint.

The effects caused by a member on the joint are important to design the joint (connection). The effects are illustrated in the following figure:



1D member model

CBFEM model – dark blue colour

Effects of the member on the joint. CBFEM model is drawn in dark blue color.

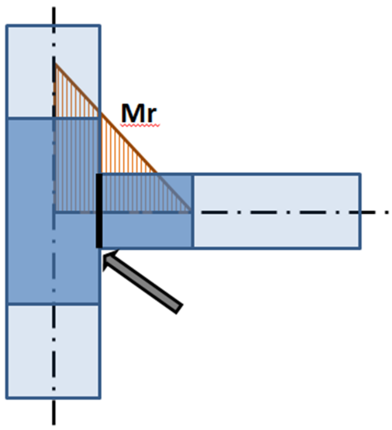
Moment M and shear force V act in the theoretical joint. The point of the theoretical joint does not exist in the CBFEM model, thus the load cannot be applied here. The model must be loaded by actions M and V which have to be transferred to the end of segment in the distance r

$M_c = M - V \cdot r$

$V_c = V$

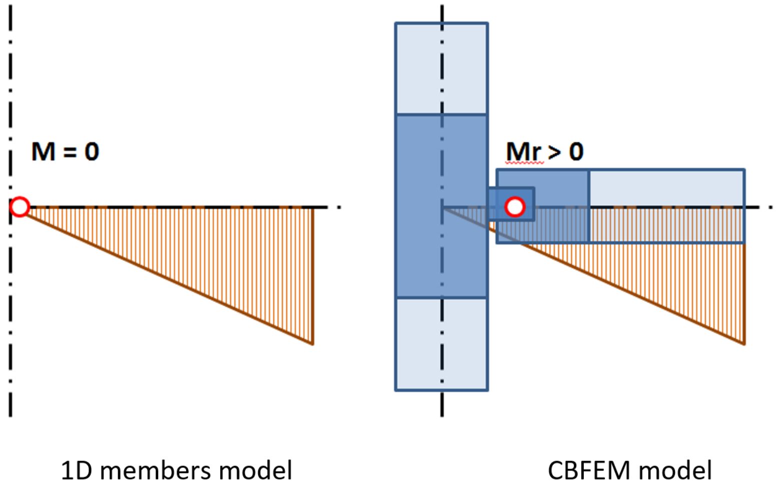
In the CBFEM model, the end section of the segment is loaded by moment M_c and force V_c.

When designing the joint, its real position relative to the theoretical point of joint must be determined and respected. The internal forces in the position of the real joint are mostly different to the internal forces in the theoretical point of joint. Thanks to the precise CBFEM model, the design is performed on reduced forces – see moment M_r in the following figure:



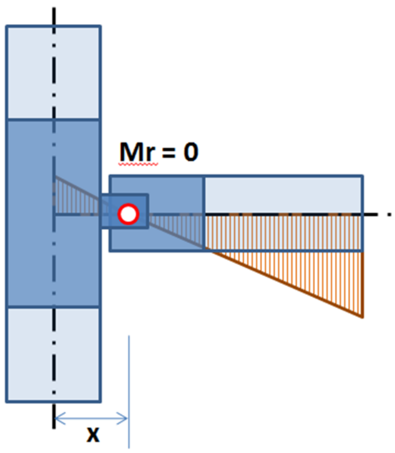
Bending moment on CBFEM model: The arrow points to the real position of joint

When loading the joint, it must be respected that the solution of the real joint must correspond to the theoretical model used for calculation of internal forces. This is fulfilled for rigid joints but the situation may be completely different for hinges.



Position of hinge in theoretical 3D FEM model and in the real structure

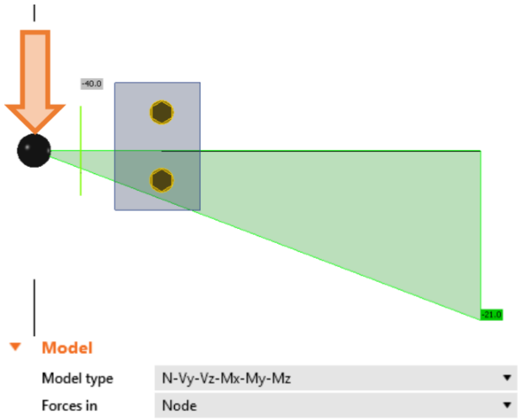
It is illustrated in the previous figure that the position of the hinge in the theoretical 1D members model differs from the real position in the structure. The theoretical model does not correspond to the reality. When applying the calculated internal forces, a significant bending moment is applied to the shifted joint and the designed joint is overlarge or cannot be designed either. The solution is simple – both models must correspond. Either the hinge in 1D member model must be defined in the proper position or the shear force must be shifted to get a zero moment in the position of the hinge.

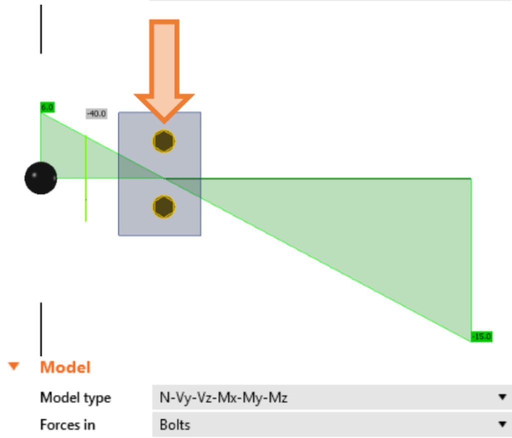


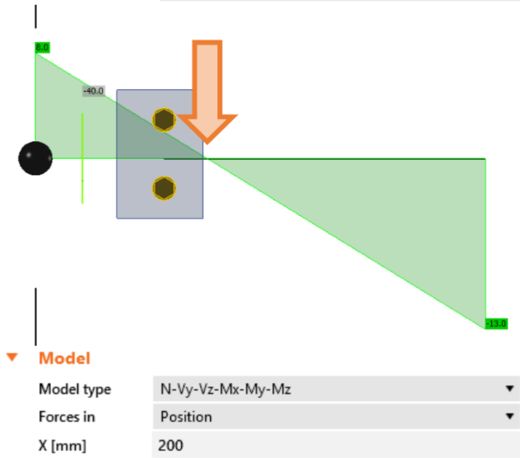
Shifted distribution of bending moment on beam: zero moment is at the position of the hinge

The shift of the shear force can be defined in the table for the internal forces definition.

The location of load effect has a big influence on the correct design of the connection. To avoid all misunderstandings, we allow the user to select from three options – **Node** / **Bolts** / **Position**.







Note that when selecting the Node option, the forces are applied at the end of a selected member which is usually at the theoretical node unless the offset of the selected member is set in geometry.

3.4.1 Import loads from FEA programs

IDEA StatiCa enables to import internal forces from third-party FEA programs. FEA programs use an envelope of internal forces from combinations. IDEA StatiCa Connection is a program which resolves steel joint nonlinearly (elastic/plastic material model). Therefore, the envelope combinations cannot be used. IDEA StatiCa searches for extremes of internal forces (N, V_y, V_z, M_x, M_y, M_z) in all combinations at the ends of all members connected to the joint. For each such extreme value, also all other internal forces from that combination in all remaining members are used. Idea StatiCa determines the worst combination for each component (plate, weld, bolt etc.) in the connection.

The user can modify this list of load cases. He can work with combinations in the wizard (or BIM) or he can delete some cases directly in IDEA StatiCa Connection.

Warning!

It is necessary to take into account unbalanced internal forces during the import. This can happen in following cases:

- Nodal force was applied into the position of the investigated node. The software cannot detect which member should transfer this nodal force and, therefore, it is not taken into account in the analysis model. *Solution: Do not use nodal forces in global analysis. If necessary, the force must be manually added to a selected member as a normal or shear force.*
- Loaded, non-steel (usually timber or concrete) member is connected to the investigated node. Such member is not considered in the analysis and its internal forces are ignored in the analysis. *Solution: Replace the concrete member with a concrete block and anchorage.*
- The node is a part of a slab or a wall (usually from concrete). The slab or the wall are not part of the model and its internal forces are ignored. *Solution: Replace the concrete slab or wall with a concrete block and anchorage.*
- Some members are connected to the investigated node via rigid links. Such members are not included in the model and their internal forces are ignored. *Solution: Add these members into the list of connected members manually.*
- Seismic load cases are analysed in the software. Most FEA software offer the modal analysis to solve seismicity. The results of internal forces of seismic load cases provide usually only internal force envelopes in sections. Due to the evaluation method (square root of the sum of squares – SRSS), the internal forces are all positive and it is not possible to find the forces matching to the selected extreme. It is not possible to achieve a balance of internal forces. *Solution: Change the positive sign of some internal forces manually.*

3.5 Strength analysis

The analysis of joint is materially non-linear. The load increments are applied gradually and the state of stress is searched. There are two optional analysis modes in IDEA Connection:

- **The response of structure (joint) to the overall load.** All defined load (100 %) is applied in this mode and the corresponding state of stress and deformation is calculated.

Status of FE analysis

	Status	Loads	Applied [%]
>	✓	LE1	100.0

- **Analysis termination at reaching the ultimate limit state.** The checkbox in Code setup “Stop at limit strain” should be ticked. The state is found when the plastic strain reaches the defined limit. In the case when the defined load is higher than the calculated capacity, the analysis is marked as non-satisfying and the percentage of used load is printed. Note that the analytical resistance of components, for example of bolts, can be exceeded.

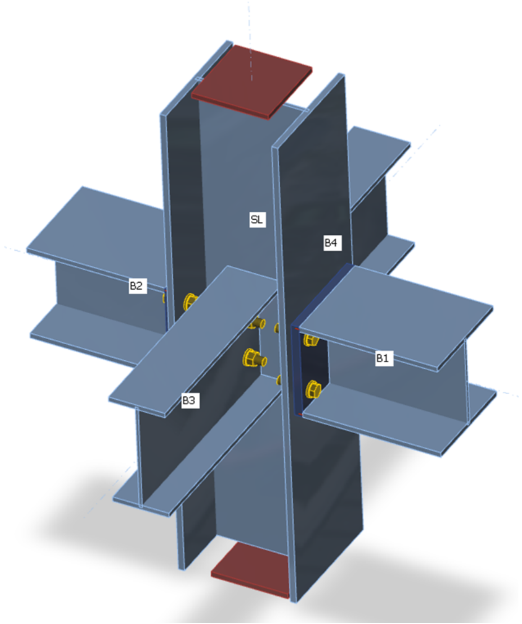
Status of FE analysis

	Status	Loads	Applied [%]
>	✗	LE1	67.2

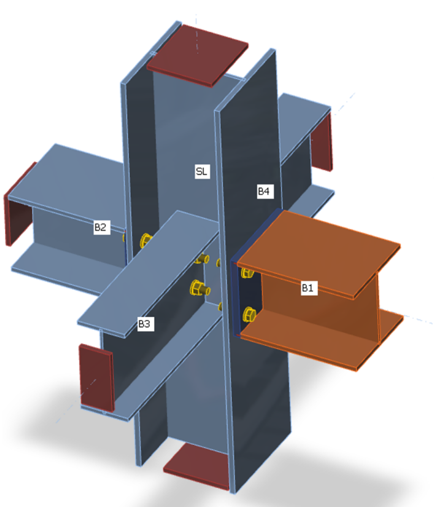
The second mode is more suitable for a practical design. The first one is preferable for detailed analysis of complex joints.

3.6 Stiffness analysis

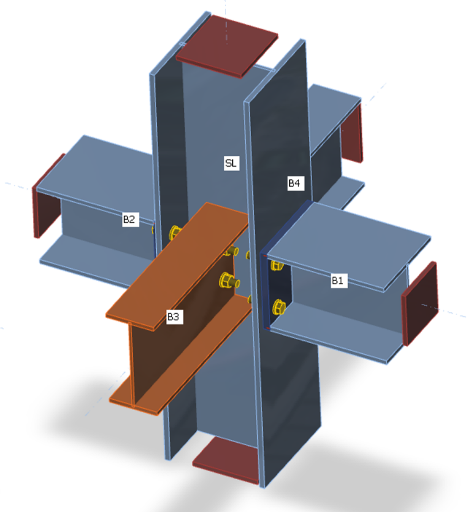
The CBFEM method enables to analyze the stiffness of connection of individual joint members. For the proper stiffness analysis, the separate analysis model must be created for each analyzed member. Then, the stiffness analysis is not influenced by the stiffness of other members of joint but only by the node itself and the construction of connection of the analyzed member. Whereas the bearing member is supported for the strength analysis (member SL in the figure below), all members except the analyzed one are supported by the stiffness analysis (see two figures below for stiffness analysis of members B1 and B3).



Supports on members for strength analysis



Supports on members for stiffness analysis of member B1



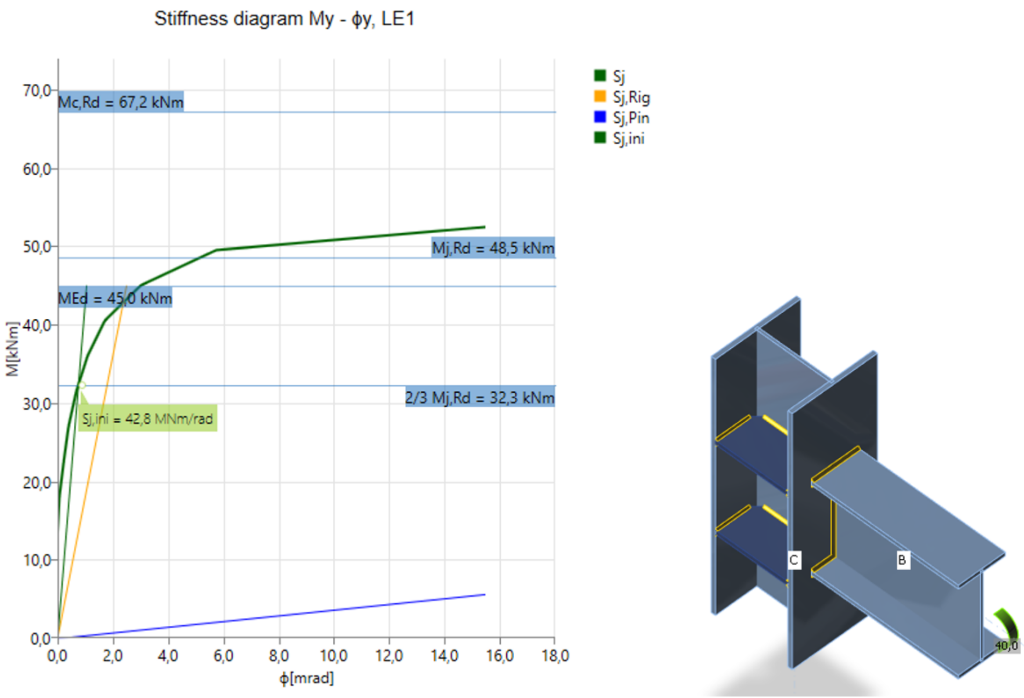
Supports on members for stiffness analysis of member B3

Loads can be applied only on the analyzed member. If bending moment, M_y, is defined, the rotational stiffness about the y-axis is analyzed. If bending moment M_z is defined, the rotational stiffness about the z-axis is analyzed. If axial force N is defined, the axial stiffness of connection is analyzed.

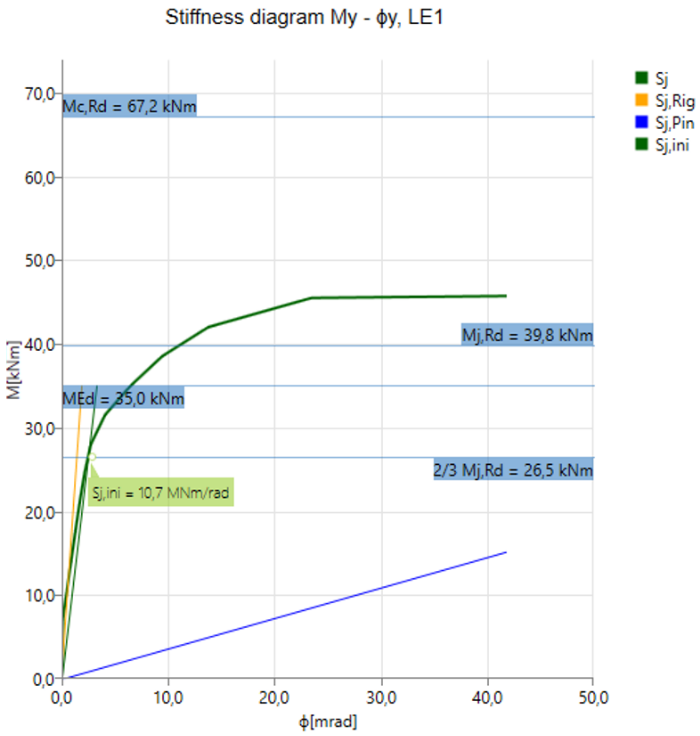
The program generates complete diagram automatically, it is directly displayed in GUI and can be added into the output report. Rotational or axial stiffness can be studied for specific design load. IDEA StatiCa Connection can also deal with the interaction of the other internal forces.

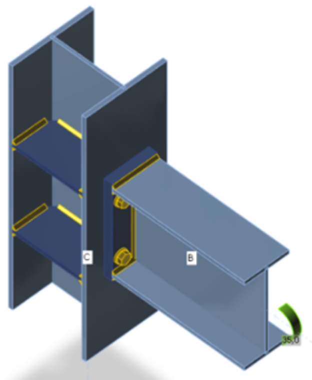
Diagram shows:

- Level of design load
- Limit value of capacity of connection for 5% equivalent strain
- Limit value of capacity of connected member (useful also for seismic design)
- 2/3 of limit capacity for calculation of initial stiffness
- Value of initial stiffness $S_{j,ini}$
- Value of secant stiffness S_{js}
- Limits for the classification of connection – rigid and pinned
- Rotational deformation Φ
- Rotational capacity Φ_c

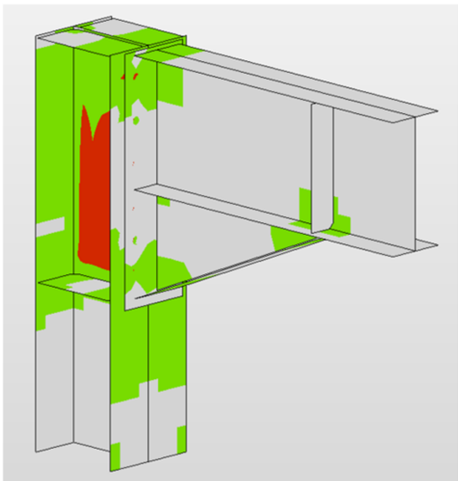


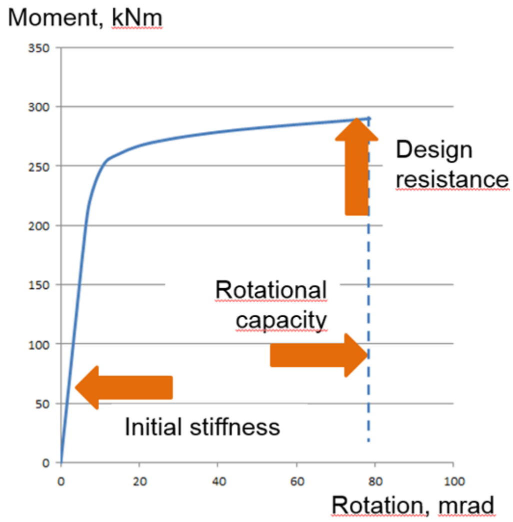
Rigid welded connection





Semi-rigid bolted connection





After reaching the 5 % strain in the column web panel in shear, the plastic zones propagate rapidly

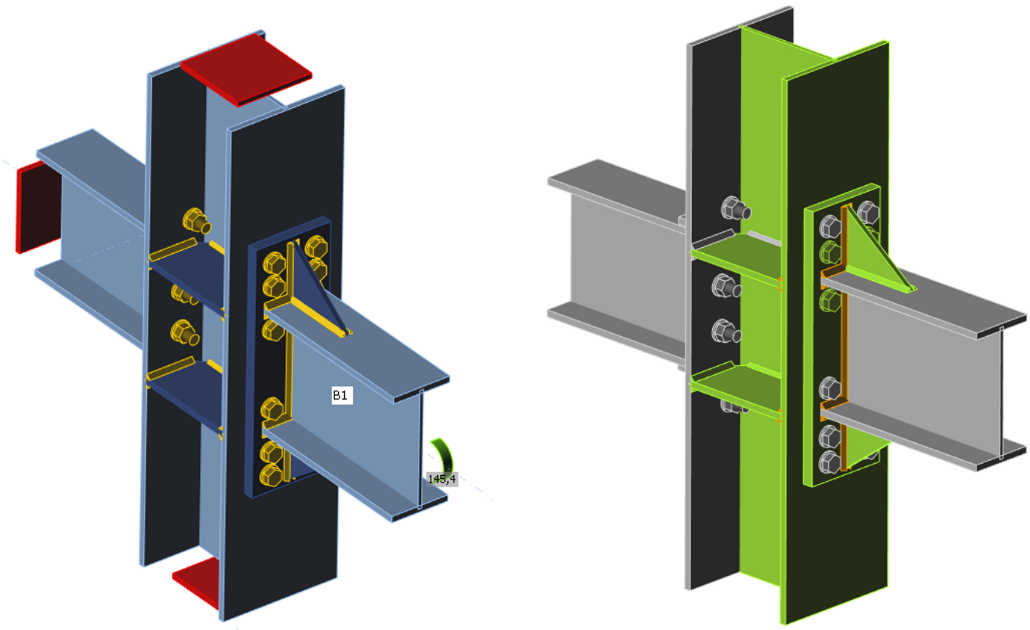
3.7 Member capacity design

IDEA Connection checks the connection on applied design load. In many regions with the danger of seismicity, it is required to check the connection on the maximal moment which can be transferred by the connected member. We calculate this moment in the software and apply it to the specific member. All other members in the joint are supported.

LE-MCn [Load]

	Member	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
>	B / End	0.0	0.0	0.0	0.0	-498.8	0.0

The value of M_y is calculated automatically using full cross-section of the analyzed member and strength properties according to the selected code. User can modify this number, e.g. in case of reduction of the beam cross-section, and can add for example shear force calculated from applied loads and the distance between plastic hinges.



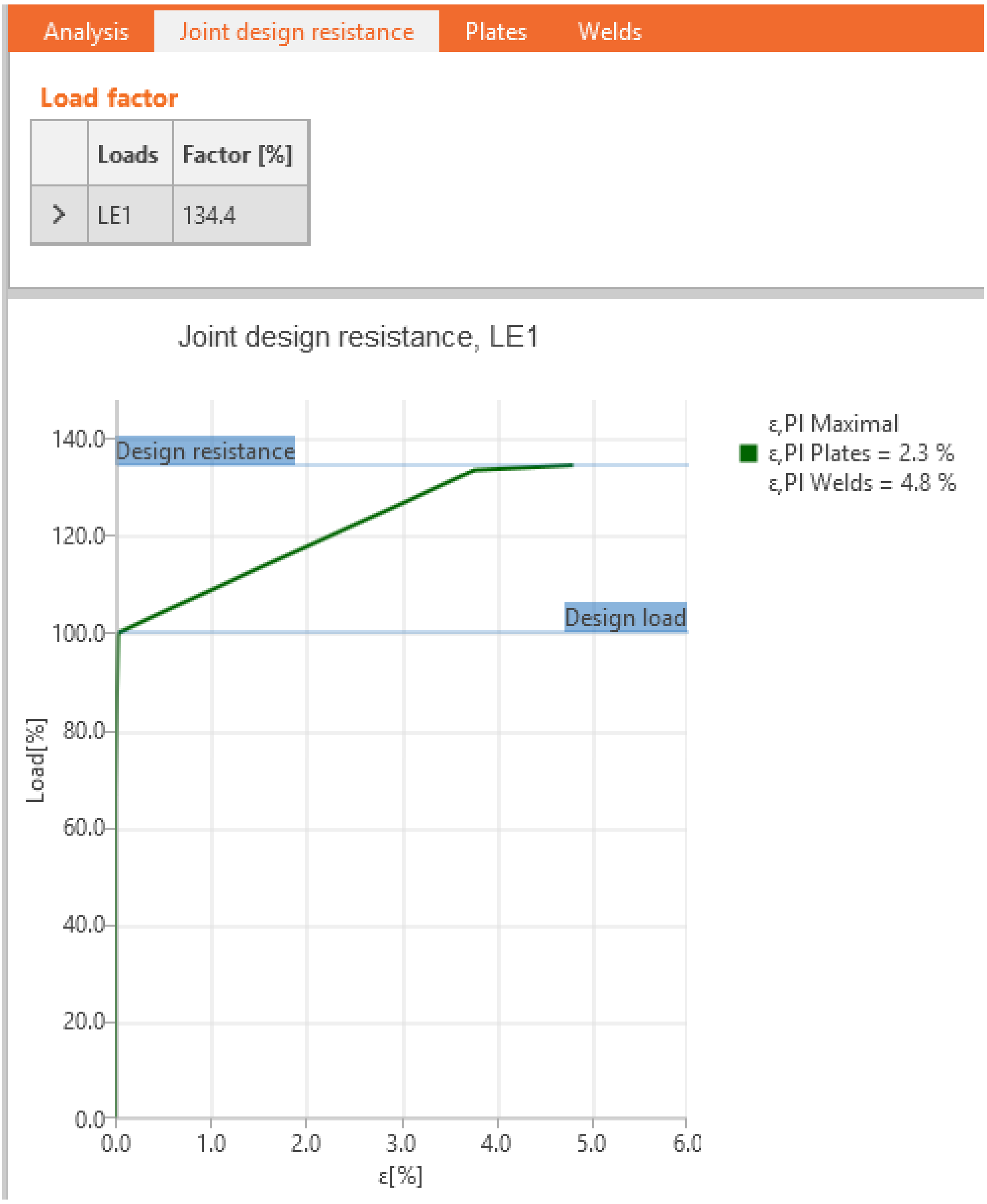
Joints designed to transfer moment equal to the member resistance (full-strength joints) usually need to be much more stiffened than the partial-strength joints.

Connected member is not checked. It has to be properly designed in the global analysis of the structure.

3.8 Joint design resistance

The designer usually solves the task to design the connection/joint to transfer the known design load. But it is also useful to know how far the design from the limit state is, i.e., how big the reserve in the design is and how safe it is. This can be done simply by the type of analysis – Design joint resistance.

The user inputs design load like in a standard design. The software automatically proportionally increases all load components until one of the checks does not satisfy. The checks of steel plates, shear and tension resistance of bolts, and approximate weld checks are included. The user gets the ratio of maximal load to the design load. Also, a simple diagram is provided. It is necessary to perform Stress/Strain analysis for accurate joint assessment.

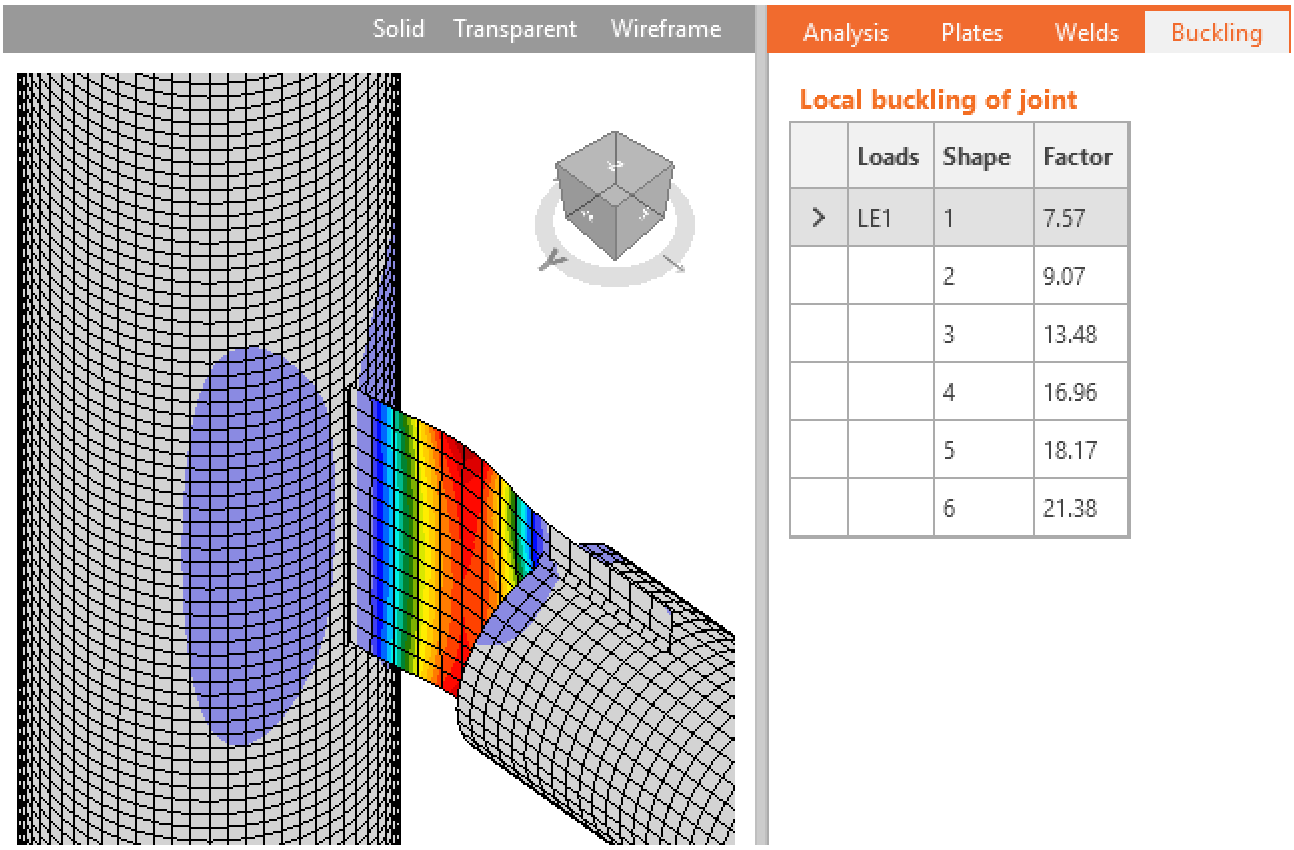


The results of user defined load case are shown unless the Joint design resistance Factor is smaller than 100 % which means that the calculation did not converge and the last converged step of the load case is shown.

3.9 Stability analysis

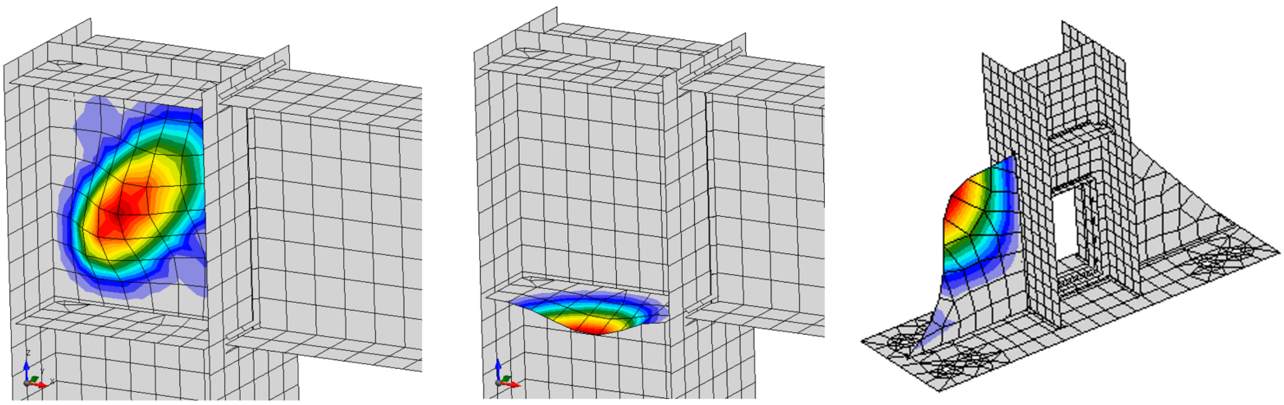
IDEA StatiCa Connection is able to perform linear buckling analysis and provide the user with the buckling factor.

It is important to distinguish between global buckling (buckling of whole members) and local buckling (buckling of individual plates). In the case of global buckling (the plate is an elongation of a member, see figure below), it is necessary to check that the buckling factor is larger than 15. For buckling factor smaller than 15, buckling has to be taken into account in the design of connected member.



Critical buckling factor for a gusset plate as an elongation of a truss

In the case of most plates in connections, the maximum value of the critical buckling factor that requires thorough buckling analysis is usually smaller; it has been verified that for stiffener and column panel in shear, it is not necessary to take into account buckling reduction factor if the critical buckling factor is higher than 3.



Examples of buckling shapes where the buckling can be neglected if critical buckling factor is higher than 3

It is possible to follow the results of IDEA StatiCa Connection with calculations or with geometrically nonlinear analysis with initial imperfections in advanced FEM software if the buckling factor is smaller than the critical value. Nevertheless, it is often more economical to use stiffeners or thicker plates in design.

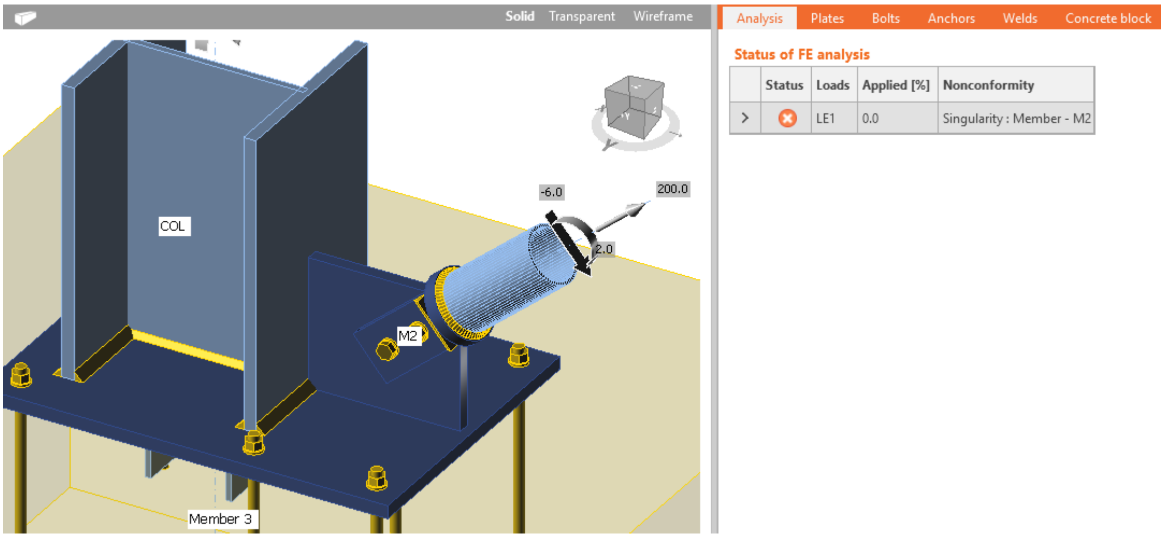
3.10 Deformation capacity

The deformation capacity/ductility δ_{Cd} belongs with the resistance and the stiffness to the three basic parameters describing the behavior of connections. In moment-resistant connections, the ductility is achieved by a sufficient rotation capacity φ_{Cd}. The deformation/rotation capacity is calculated for each connection in the joint separately.

The estimation of the rotation capacity is important in connections exposed to seismic, see Gioncu and Mazzolani (2002) and Grecea (2004) and extreme loading, see Sherbourne and Bahaari (1994 and 1996). The deformation capacity of components has been studied from the end of the last century (Foley and Vinnakota, 1995). Faella et al. (2000) carried out tests on T-stubs and derived the analytical expressions for the deformation capacity. Kuhlmann and Kuhnemund (2000) performed tests on the column web subjected to transverse compression at different levels of compression axial force in the column. Da Silva et al. (2002) predicted deformation capacity at different levels of axial force in the connected beam. Based on the test results combined with FE analysis, deformation capacities are established for the basic components by analytical models by Beg et al. (2004). In the work, components are represented by non-linear springs and appropriately combined in order to determine the rotation capacity of the joint for the end-plate connections, with an extended or flush end-plate and welded connections. For these connections, the most important components that may significantly contribute to the rotation capacity were recognized as the web in compression, column web in tension, column web in shear, column flange in bending, and end-plate in bending. Components related to the column web are relevant only when there are no stiffeners in the column that resist compression, tension or shear forces. The presence of a stiffener eliminates the corresponding component, and its contribution to the rotation capacity of the joint can be therefore neglected. End-plates and column flanges are important only for end-plate connections where the components act as a T-stub, where also the deformation capacity of the bolts in tension is included. The questions and limits of deformation capacity of connections of high strength steel were studied by Girao at al. (2004).

3.11 Analysis convergence

Finite element analysis requires slightly increasing stress-strain diagram of material models. In some cases of complicated models, e.g. with multiple contacts, the increase in divergent iterations might help with convergence. This value can be set in Code setup. Most common cause of analysis failure are singularities when the parts of a model are not connected properly and are free to move or rotate. A user is notified and should check the model for missing welds or bolts. The deformed shape is shown with the items which caused the first singularity moved 1 m so that singularity may be easily detected.



Missing welds at gusset plates leading to singularity

4 Check of components according to Eurocode

CBFEM method combines advantages of general Finite Element Method (FEM) and standard Component Method (CM). The stresses and internal forces calculated on the accurate CBFEM model are used in checks of all components.

Individual components are checked according to Eurocode EN 1993-1-8.

4.1 Plates

The resulting equivalent stress (Huber-Mises-Hencky – HMH, von Mises) and plastic strain are calculated on plates. Elasto-plastic material model is used for steel plates. A check of an equivalent plastic strain is performed. The limiting value of 5 % is suggested in Eurocode (EN 1993-1-5, app. C, par. C8, note 1), this value can be modified by the user in Code setup.

Plate element is divided into 5 layers and elastic/plastic behavior is investigated in each layer separately. Output summary lists the most critical check from all 5 layers.

Check of members and steel plates for extreme load effect

	Status	Item	Th [mm]	Loads	σ,Ed [MPa]	ε,Pl [%]
>		COL-bfl 1	19,0	LE1	164,2	0,0
		COL-tfl 1	19,0	LE1	159,8	0,0
		COL-w 1	11,0	LE1	55,8	0,0
		BP1	20,0	LE1	189,9	0,0

Design data

	Material	Fy [MPa]	ε,lim [%]
>	S 355	355,0	5,0

CBFEM method can provide stress rather higher than the yield strength. The reason is the slight inclination of the plastic branch of the stress-strain diagram which is used in the analysis to improve the stability of interaction calculation. This is not a problem for practical design. At higher loads, the equivalent plastic strain is rising and the joint fails while exceeding the plastic strain limit.

4.2 Welds

4.2.1 Fillet welds

Design resistance

The plastic strain in weld is limited to 5 % as in the plate (EN1993-1-5 App. C, Par. C.8, Note 1). The stress in the throat section of a fillet weld is determined according to EN 1993-1-8 Cl. 4.5.3. Stresses are calculated from the stresses in weld element. Bending moment around the weld longitudinal axis is not taken into account.

$$\sigma_{w,Ed} = \sqrt{\sigma_{\perp}^2 + 3\left(\tau_{\perp}^2 + \tau_{\parallel}^2\right)}$$

$$\sigma_{w,Rd} = \frac{f_u}{\beta_w \gamma_{M2}}$$

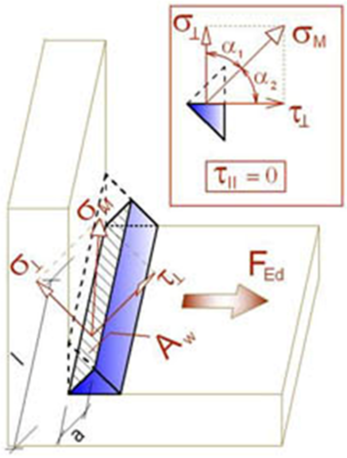
Weld utilization

$$U_t = \min\left\{\frac{\sigma_{w,Ed}}{\sigma_{w,Rd}}, \frac{\sigma_{\perp}}{0.9 f_u/\gamma_{M2}}\right\}$$

where:

- β_w – correlation factor (EN 1993-1-8 – Table 4.1)
- f_u – ultimate strength, chosen as the lower of the two connected base materials
- γ_{M2} – safety factor (EN 1993-1-8 – Table 2.1; editable in Code setup)

The plastic strain in weld is limited to 5 % as in the plate (EN1993-1-5 App. C, Par. C.8, Note 1). The stress in the throat section of a fillet weld is determined according to EN 1993-1-8 Cl. 4.5.3. Stresses are calculated from the stresses in weld element. Bending moment around the weld longitudinal axis is not taken into account.



All values required for check are printed in tables.

4.2.2 Butt welds

Welds can be specified as butt welds. Complete joint penetration is considered for butt welds, thus such welds are not checked.

4.3 Bolts

The initial stiffness and design resistance of bolts in shear are in CBFEM modeled according to Cl. 3.6 and 6.3.2 in EN 1993-1-8. The spring representing bearing and tension has a bi-linear force-deformation behavior with an initial stiffness and design resistance according to Cl. 3.6 and 6.3.2 in EN 1993-1-8.

Design tension resistance of bolt (EN 1993-1-8 – Table 3.4):

$$F_{t,Rd} = 0.9 f_{ub} A_s / \gamma_{M2}$$

Design punching shear resistance of bolt head or nut (EN 1993-1-8 – Table 3.4):

$$B_{p,Rd} = 0.6 \pi d_m t_p f_u / \gamma_{M2}$$

Design shear resistance per one shear plane (EN 1993-1-8 – Table 3.4):

$$F_{v,Rd} = \alpha_v f_{ub} A_s / \gamma_{M2}$$

Design shear resistance can be multiplied by reduction factor β_p if packing is present (EN 1993-1-8 – Cl. 3.6.1. (12)) and this option is selected in Code setup.

Design bearing resistance of plate (EN 1993-1-8 – Table 3.4):

$$F_{b,Rd} = k_1 a_b f_u d t / \gamma_{M2}$$ for standard holes

$$F_{b,Rd} = 0.6 k_1 a_b f_u d t / \gamma_{M2}$$ for slotted holes

Utilisation in tension [%]:

$$Ut_t = \frac{F_{t,Ed}}{\min(F_{t,Rd}, B_{p,Rd})}$$

Utilisation in shear [%]:

$$Ut_t = \frac{V}{\min(F_{v,Rd}, F_{b,Rd})}$$

Interaction of shear and tension [%]:

$$Ut_{ts} = \frac{V}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4 F_{t,Rd}}$$

where:

- A_s – tensile stress area of the bolt
- f_{ub} – ultimate tensile strength of the bolt
- d_m – mean of the across points and across flats dimensions of the bolt head or the nut, whichever is smaller
- d – bolt diameter
- t_p – plate thickness under the bolt head/nut
- f_u – ultimate steel strength
- α_v = 0.6 for classes (4.6, 5.6, 8.8) or 0.5 for classes (4.8, 5.8, 6.8, 10.9)
- $k_1 = \min\left(2.8\frac{e_2}{d_0} - 1.7,\ 1.4\frac{p_2}{d_0} - 1.7,\ 2.5\right)$ – factor from Table 3.4

- α_b = 1.0 if the bearing check with ab is deactivated in Code setup; if the check is activated, the value of α_b is determined according to EN 1993-1-8 – Table 3.4:
  $\alpha_d = \min\left(\frac{e_1}{3d_0}, \frac{p_1}{3d_0} - \frac{1}{4}\right)$
- e_1, e_2 – edge distances in the direction of the load and perpendicular to the load
- p_1, p_2 – bolt pitches in the direction of the load and perpendicular to the load
- $F_{t,Ed}$ – design tensile force in bolt
- V – design shear force in bolt
- γ_{M2} – safety factor (EN 1993-1-8 – Table 2.1; editable in Code setup)

Check of bolts for extreme load effect

		Status	Item	Loads	Ft [kN]	V [kN]	Fb,Rd [kN]	Utt [%]	Uts [%]	Utts [%]
>	+	✓	B1	LE1	5,9	19,9	92,5	6,5	33,1	37,8
	+	✓	B2	LE1	2,6	20,1	92,5	2,9	33,3	35,3

Design data

	Item	Ft,Rd [kN]	Bp,Rd [kN]	Fv,Rd [kN]
>	M16 8.8 - 1	90,4	109,0	60,3

4.4 Preloaded bolts

Design slip resistance per bolt grade 8.8 or 10.9 (EN 1993-1-8, Cl. 3.9 – Equation 3.8):

$$F_{s,Rd} = \frac{k_s n \mu (F_{p,C} - 0.8 F_{t,Ed})}{\gamma_{M3}}$$

The preload (EN 1993-1-8 – Equation 3.7)

$F_{p,C} = 0.7\, f_{ub}\, A_s$

The preloading force factor 0.7 can be modified in Code setup.

Utilization [%]:

$$Ut_s = \frac{V}{F_{s,Rd}}$$

where:

- A_s – tensile stress area of the bolt
- f_{ub} – ultimate tensile strength
- k_s – a coefficient (EN 1993-1-8 – Table 3.6; k_s = 1 for normal round holes, k_s = 0.63 for slotted holes)
- μ – slip factor editable in Code setup (EN 1993-1-8 – Table 3.7)
- n – number of the friction surfaces. Check is calculated for each friction surface separately
- γ_{M3} – safety factor (EN 1993-1-8 – Table 2.1; editable in Code setup – recommended values are 1.25 for ultimate limit state and 1.1 for serviceability limit state design)
- V – design shear force in bolt
- $F_{t,Ed}$ – design tensile force in bolt

4.5 Anchors

The anchor bolt resistance caused by concrete failure is evaluated according to ETAG 001 Annex C and Cl. 6.2.6.12 in EN 1993-1-8. Steel tensile failure mode is determined according to EN 1993-1-8 – Cl. 3.6.1.

Tensile resistance (EN 1993-1-8 – Cl. 3.6.1):

$$F_{t,Rd} = \frac{c k_2 f_{ub} A_s}{\gamma_{M2}} \geq F_t$$

where:

- c – decrease in tensile resistance of bolts with cut thread according to EN 1993-1-8 – Cl. 3.6.1. (3) editable in Code setup
- k_2 = 0.9 – factor from Table 3.4 in EN 1993-1-8
- f_{ub} – anchor bolt ultimate strength
- A_s – anchor bolt tensile stress area
- γ_{M2} – safety factor (EN 1993-1-8 – Table 2.1; editable in Code setup)

Concrete cone failure resistance of anchor or group of anchors (ETAG-001 – 5.2.2.4):

$$N_{Rd,c} = \frac{N^0_{Rk,c} \frac{A^0_{c,N}}{A_{c,N}} \psi_{s,N} \psi_{re,N}}{\gamma_{Mc}} \geq F_t$$

The initial value of characteristic resistance (cracked concrete is assumed):

$$N_{Rk,c}^{0} = 7.2 f_{ck,cube}^{0.5} h_{ef}^{1.5}$$

where:

- $A_{c,N}^{0}$ – area of concrete of an individual anchor with large spacing and edge distance at the concrete surface
- h_{ef} – length of the anchor in concrete (for three or more close edges, ETAG 001 – Annex C – Chapter 5.2.2.4 f) applies)
- $f_{ck,cube}$ – characteristic cubic concrete compressive strength
- $A_{c,N}$ – actual area of concrete cone of the anchorage at the concrete surface respecting influence of edges and adjoining anchors
- $\psi_{s,N}$ = 1.0
- $\psi_{re,N}$ = 1.0
- $\gamma_{Mc} = \gamma_c \cdot \gamma_{inst}$ – safety factor (ETAG 001 – Annex C – Chapter 3.2.2.1)
- γ_c – partial safety factor for concrete (editable in Code setup)
- γ_{inst} – partial safety factor taking account of the installation safety of an anchor system (editable in Code setup)

Anchor shear resistance in case of transfer of shear forces with direct stand-off. Friction is not taken into account. Valid in case, that the anchor failure precedes the concrete failure (ETAG-001 – 5.2.3.2 a)):

$$V_{Rd,s} = \frac{\alpha_M M_{Rk,s}/l}{\gamma_{Ms}} \geq V$$

where:

- α_M = 2 – full restraint is assumed (ETAG 001 – Annex C – Chapter 4.2.2.4)
- $M_{Rk,s} = M_{Rk,s}^{0}\left(1 - \frac{F_t}{F_{t,Rd}}\right)$ – characteristic bending resistance of the anchor decreased by the tensile force in the anchor
- $M_{Rk,s}^{0} = 1.2\, W_{el}\, f_{ub}$ – characteristic bending resistance of the anchor
- $W_{el} = \frac{\pi d_{nom}^{3}}{32}$ – section modulus of the anchor
- d_{nom} – anchor bolt nominal diameter

- f_{ub} – anchor bolt ultimate strength
- F_t – tensile force in the anchor
- $F_{t,Rd}$ – tensile resistance of the anchor
- $l = 0.5\ d_{nom} + t_{mortar} + 0.5\ t_{bp}$ – lever arm
- t_{mortar} – thickness of mortar (grout)
- t_{bp} – thickness of the base plate
- γ_{Ms} – partial safety factor for steel failure (ETAG 001 – Annex C – Chapter 3.2.2.2)

Concrete pry-out failure (ETAG-001 – 5.2.3.3):

$$V_{Rd,cp} = \frac{V_{Rk,cp}}{\gamma_{Mc}} \geq V$$

$$V_{Rk,cp} = kN_{Rk,c}$$

where:

- V – design shear force in anchor
- $k = 1$ for $h_{ef} < 60$ mm; $k = 2$ for $h_{ef} \geq 60$ mm
- $\gamma_{Mc} = \gamma_c$ – safety factor (ETAG 001 – Annex C – Chapter 3.2.2.1, $\gamma_{inst} = 1.0$ for shear loading)
- γ_c – partial safety factor for concrete (editable in Code setup)

Concrete edge failure (ETAG-001 – 5.2.3.4):

$$V_{Rd,c} = \frac{V_{Rk,c}}{\gamma_{Mc}} \geq V$$

$$V_{Rk,c} = V^0_{Rk,c} \frac{A_{c,V}}{A^0_{c,V}} \psi_{s,V} \psi_{h,V} \psi_{\alpha,V} \psi_{ec,V} \psi_{re,V}$$

$$V^0_{Rk,c} = 1.7 d^{\alpha}_{nom} h^{\beta}_{ef} f^{0.5}_{ck,cube} c^{1.5}_1$$

$$\alpha = 0.1 \left(\frac{l_f}{c_1} \right)^{0.2}$$

$$\beta = 0.1 \left(\frac{d_{nom}}{c_1} \right)^{0.2}$$

where:

- h_{ef} – length of the anchor in concrete
- l_f = min (h_{ef}, 8 d_{nom}) – effective length of the anchor in shear
- c_1 – edge distance
- d_{nom} – nominal anchor diameter
- $\psi_{s,V} = 0.7 + 0.3 \frac{c_2}{1.5c_1} \leq 1.0$ – factor which takes account of the disturbance of the distribution of stresses in the concrete due to further edges of the concrete member on the shear resistance
- $\psi_{h,V} = \left(\frac{1.5c_1}{h}\right)^{0.5} \leq 1.0$ – factor which takes account of the fact that the shear resistance does not decrease proportionally to the member thickness as assumed by the ratio $A_{c,V}$ / $A_{c,V}^0$.
- $\psi_{\alpha,V}$ = 1.0 – takes account of the angle α_V between the load applied, V, and the direction perpendicular to the free edge of the concrete member; conservatively is assumed as 1.0
- $\psi_{ec,V}$ = 1.0 – factor which takes account of a group effect when different shear loads are acting on the individual anchors of a group; each anchor is checked separately in IDEA StatiCa
- $\psi_{re,V}$ = 1.0 – factor takes account of the effect of the type of reinforcement used in cracked concrete
- c_2 – smaller distance to the concrete edge perpendicular to the distance c_1
- h – concrete block height
- $A_{c,V}^0$ – area of concrete cone of an individual anchor at the lateral concrete surface not affected by edges (4.5 c_1^2)
- $A_{c,V}$ – actual area of the concrete cone of anchorage at the lateral concrete surface
- $\gamma_{Mc} = \gamma_c$ – safety factor (ETAG 001 – Annex C – Chapter 3.2.2.1, γ_{inst} = 1.0 for shear loading)
- γ_c – partial safety factor for concrete (editable in Code setup)

Check of anchors for extreme load effect

		Status	Item	Grade	Loads	Ft [kN]	V [kN]	Nrdc [kN]	Vrds [kN]	Utt [%]	Uts [%]	Utts [%]	Vrd,cp [kN]	Vrd,c [kN]	Vrd,cp,s	Vrd,c,s
>	+	✓	A1	M16 8.8 - 1	LE1	35,5	8,9	58,3	50,2	60,9	17,7	57,0	109,0	42,8	✓	✓
	+	✓	A2	M16 8.8 - 1	LE1	46,2	9,7	58,3	50,2	79,2	19,2	81,2	109,0	42,8	✓	✓
	+	✓	A3	M16 8.8 - 1	LE1	0,0	7,2	-	50,2	0,0	14,4	6,9	109,0	42,8	✓	✓
	+	✓	A4	M16 8.8 - 1	LE1	0,0	6,8	-	50,2	0,0	13,6	6,4	109,0	42,8	✓	✓

Design data

	Item	Ft,Rd [kN]	Bp,Rd [kN]
>	M16 8.8 - 1	76,9	369,5

4.5.1 Anchors with stand-off

Anchor with stand-off is designed as a bar element loaded by shear force, bending moment and compressive or tensile force. These internal forces are determined by finite element model. The anchor is fixed on both sides, one side is 0.5×d below the concrete level, the other side is in the middle of the thickness of the plate. The buckling length is conservatively assumed as twice the length of the bar element. Plastic section modulus is used. The bar element is designed according to EN 1993-1-1. The shear force may decrease the yield strength of the steel according to Cl. 6.2.8 but the minimum length of the anchor to fit the nut under the base plate ensures that the anchor fails in bending before the shear force reaches half the shear resistance. The reduction is therefore not necessary. Interaction of bending moment and compressive or tensile strength is assessed according to Cl. 6.2.1.

Shear resistance (EN 1993-1-1 Cl. 6.2.6):

$$V_{pl,Rd} = \frac{A_V f_y / \sqrt{3}}{\gamma_{M2}}$$

- $A_V = 0.844\, A_s$ – shear area
- A_s – bolt area reduced by threads
- f_y – bolt yield strength
- γ_{M2} – partial safety factor

Tensile resistance (EN 1993-1-1 Cl. 6.2.3):

$$F_{t,Rd} = \frac{A_s f_y}{\gamma_{M2}}$$

Compressive resistance (EN 1993-1-1 Cl. 6.3):

$$F_{c,Rd} = \frac{\chi A_s f_y}{\gamma_{M2}}$$

- $\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \leq 1$ – buckling reduction factor
- $\Phi = 0.5\left[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2\right]$ – value to determine buckling reduction factor χ
- $\bar{\lambda} = \sqrt{\frac{A_s f_y}{N_{cr}}}$ – relative slenderness
- $N_{cr} = \frac{\pi^2 EI}{L_{cr}^2}$ – Euler's critical force
- $I = \frac{\pi d_s^4}{64}$ – moment of inertia of the bolt
- $L_{cr} = 2\,l$ – buckling length
- l – length of the bolt element equal to half the base plate thickness + gap + half the bolt diameter

Bending resistance (EN 1993-1-1 Cl. 6.2.5):

$$M_{pl} = \frac{W_{pl} f_y}{\gamma_{M2}}$$

- $W_{pl} = \frac{d_s^3}{6}$ – section modulus of the bolt

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{Ed}}{M_{Rd}} \leq 1$$

N_{Ed} – tensile (positive) or compressive (negative sign) design force

N_{Rd} – tensile (positive, $F_{t,Rd}$) or compressive (negative sign, $F_{c,Rd}$) design resistance

M_{Ed} – design bending moment

$M_{Rd} = M_{pl,Rd}$ – design bending resistance

4.6 Concrete block

The resistance of concrete in 3D compression is determined based on EN 1993-1-8 by calculating the design bearing strength of concrete in the joint, f_{jd}, under the effective area, A_{eff}, of the base plate. The design bearing strength of the joint, f_{jd}, is evaluated according to

Cl. 6.2.5 in EN 1993-1-8 and Cl. 6.7 in EN 1992-1-1. The grout quality and thickness is introduced by the joint coefficient, β_{jd}. For grout quality equal or better than the quality of the concrete block, $\beta_{jd} = 1.0$ is expected. The effective area, $A_{eff,cm}$ under the base plate is estimated to be of the shape of the column cross-section increased by additional bearing width, c.

$$c = t\sqrt{\frac{f_y}{3 f_j \gamma_{M0}}}$$

where t is the thickness of the base plate, f_y is the base plate yield strength and γ_{M0} is the partial safety factor for steel.

The effective area is calculated by iteration until the difference between the additional bearing widths of current and previous iteration $|c_i - c_{i-1}|$ is less than 1 mm.

The area where the concrete is in compression is taken from results of FEA. This area in compression, $A_{eff,FEM}$, allows determining the position of the neutral axis. The user can modify this area by editing “Effective area – influence of mesh size” in Code setup. The default value is 0.1 for which the verification studies were made. It is not recommended to decrease this value. Increasing this value makes the assessment of concrete bearing resistance safer. The value in Code setup determines the boundary of the area, $A_{eff,FEM}$, e.g. the value of 0.1 takes into account only areas where stress in concrete is higher than 0.1 times the maximum stress in concrete, $\sigma_{c,max}$. The intersection of the area in compression, $A_{eff,FEM}$, and the effective area, $A_{eff,cm}$, allows to assess the resistance for generally loaded column base of any column shape with any stiffeners and is labeled A_{eff}. The average stress σ on the effective area, A_{eff}, is determined as the compression force divided by the effective area. Check of the component is in stresses $\sigma \le f_{jd}$.

Concrete resistance at concentrated compression:

$$f_{jd} = \beta_j k_j \frac{f_{ck}}{\gamma_c}$$

Average stress under the base plate:

$$\sigma = \frac{N}{A_{eff}}$$

Utilization in compression [%]:

$$Ut = \frac{\sigma}{f_{jd}}$$

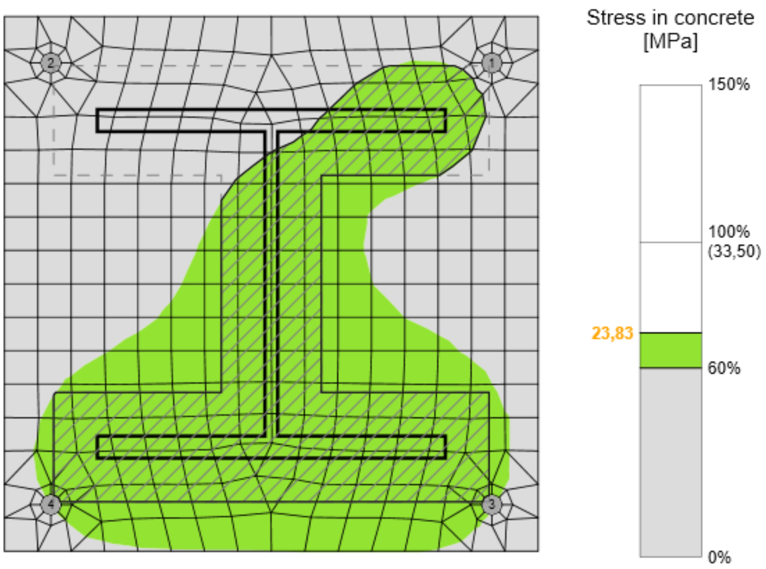
where:

- f_{ck} – characteristic compressive concrete strength
- β_j = 0.67 – factor of grout quality editable in Code setup
- k_j – concentration factor
- γ_c – safety factor for concrete
- A_{eff} – effective area on which the column normal force N is distributed

Check of contact stress in concrete for extreme load effect

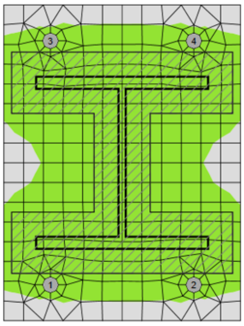
		Status	Item	Loads	c [mm]	Aeff [m2]	σ [MPa]	Kj	Fjd [MPa]	Ut [%]
>	+	✓	CB 1	LE1	38	0,04	10,2	3,00	33,5	30,6

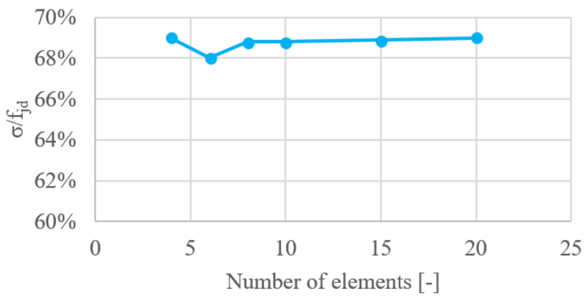
Effective area, $A_{eff,cm}$, as calculated according to EC for pure compression is marked with a dashed line. The graphical representation shows the way of checking. Calculated effective area, $A_{eff,fem}$, is marked as green. Final effective area, A_{eff}, for contact stress check is highlighted as hatched.



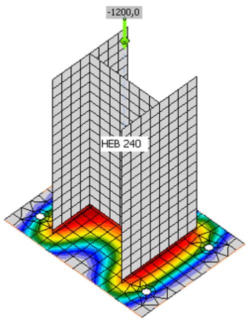
4.6.1 Mesh sensitivity

This procedure of assessing the resistance of the concrete in compression is independent on the mesh of the base plate as can be seen in the figures bellow. It is shown in the example of concrete in compression assessment according to EC. Two cases were investigated: loading by pure compression of 1200 kN and loading by a combination of compressive force 1200 kN and bending moment 90 kN.

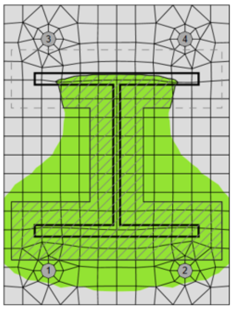


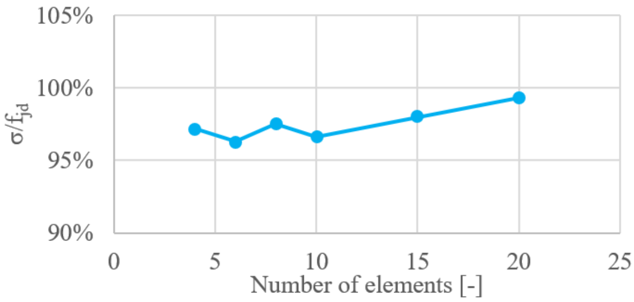


No. of elements	A_{eff} [m^2]	σ [MPa]	f_{jd} [MPa]
4	0.06	18.5	26.8
6	0.06	18.2	26.8
8	0.06	18.5	26.8
10	0.06	18.4	26.8
15	0.06	18.5	26.8
20	0.06	18.5	26.8

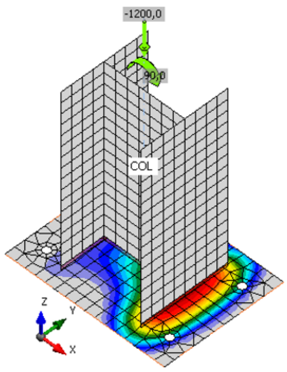


Influence of number of elements on prediction of resistance of concrete in compression in case of pure compression





No. of elements	A_{eff} [m^2]	σ [MPa]	f_{jd} [MPa]
4	0.05	26.0	26.8
6	0.04	25.8	26.8
8	0.04	26.1	26.8
10	0.05	25.9	26.8
15	0.04	26.3	26.8
20	0.04	26.6	26.8



The influence of number of elements on prediction of resistance of concrete in compression in case of compression and bending

4.7 Shear in concrete block

Shear in concrete block can be transferred via one of the three means:

1. Friction

   $V_{\mathrm{rd},y} = N\ C_\mathrm{f}$

   $V_{\mathrm{rd},z} = N\ C_\mathrm{f}$

2. Shear lug

   $$Ut = \max\left(\frac{V_y}{V_{Rd,y}},\ \frac{V_z}{V_{Rd,z}},\ \frac{V}{V_{c,Rd}}\right)$$

   $$V_{Rd,y} = \frac{A_{Vy} f_y}{\sqrt{3}\gamma_{M0}}$$

   $$V_{Rd,z} = \frac{A_{Vz} f_y}{\sqrt{3}\gamma_{M0}}$$

   $$V_{c,Rd} = A\sigma_{Rd,max}$$

   Shear iron and welds are also checked by FEM.

3. Anchors

   Check is provided according to ETAG 001 – Annex C

where:

- $A_{\mathrm{V},y}$, $A_{\mathrm{V},z}$ – shear areas of shear iron cross-section in the direction of axes *y* and *z*
- f_y – yield strength
- γ_{M0} – safety factor
- V_y – shear force component in the base plate plane in y-direction
- V_z – shear force component in the base plate plane in z-direction
- *V* – shear force (vector sum of both shear forces components)
- *N* – force perpendicular to the base plate
- C_f – friction coefficient
- *A* = *l b* – projected area of the shear lug excluding the portion above concrete
- *l* – length of the shear lug excluding the portion above concrete
- *b* – projected width of the shear lug in the direction of the shear load
- $\sigma_{Rd,max} = \frac{k_1 v' f_{ck}}{\gamma_c}$ – maximum stress which can be applied at the edges of the node
- k_1 – factor (EN 1992-1-1 – Equation (6.60))
- $v' = 1 - f_{\mathrm{ck}} / 250$– factor (EN 1992-1-1 – Equation (6.57N))

- f_{ck} – characteristic resistance of concrete in compression
- γ_c – safety factor for concrete

Shear in contact plane for extreme load effect

		Status	Item	Loads	Vy [kN]	Vz [kN]	Vrdy [kN]	Vrdz [kN]	Vcrd [kN]	Ut [%]
>	+	✓	BP1	LE1	-50,0	-99,8	2336,5	972,7	603,9	18,5

Design data

	Friction	Css
>	0,25	CON1(HEB300)

4.8 Member capacity design

Member capacity design is performed according to EN 1998

$R_d \geq 1.1\ \gamma_{ov}\ R_{fy}$

where:

- R_d – resistance of non-dissipative connection
- R_{fy} – yield strength
- γ_{ov} = 1.25

4.9 Stability analysis

There are five categories of finite element structural analysis with following assumptions:

1. Linear material, geometrically linear
2. Nonlinear material, geometrically linear
3. Linear material, linear loss of stability – buckling
4. Linear material, geometrically nonlinear using imperfections
5. Nonlinear material, geometrically nonlinear using imperfections

A design procedure which combines approaches 2 and 3 – material nonlinearity and stability analysis – is mentioned in Chapter 8 of EN 1993-1-6. The verification of buckling resistance based on the obtained FEM results is described in Annex B of EN 1993-1-5. This procedure is used for wide range of structures except for very slender shells, where geometrically nonlinear analysis with initial imperfections is more suitable (4 and 5).

The procedure uses load amplifiers α which are obtained as the results of FEM analysis and allows to predict the post-buckling resistance of the joints.

The load coefficient, $\alpha_{ult,k}$, is determined by reaching the plastic capacity without considering the geometrical nonlinearity. The check of plastic capacity and the general automatic determination of $\alpha_{ult,k}$ is implemented into the developed software.

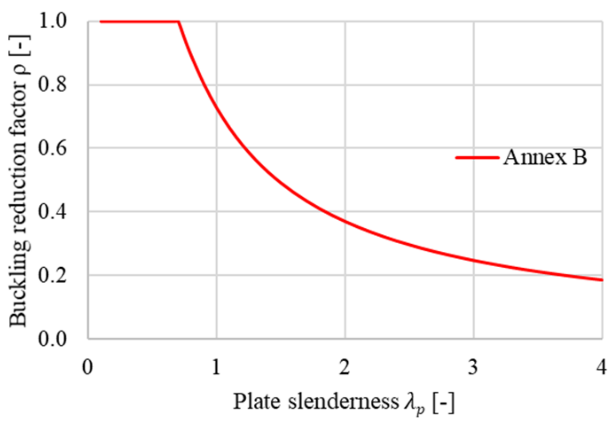
The critical buckling factor, α_{cr}, is determined, which is obtained using FEM analysis of linear stability. It is determined automatically in the software using the same FEM model as for calculation of $\alpha_{ult,k}$. It should be noted that the critical point in terms of the plastic resistance is not necessarily assessed in the first critical buckling mode. More buckling modes need to be assessed in a complex joint because they are related to different parts of the joint.

The non-dimensional plate slenderness, $\bar{\lambda}_p$, of the examined buckling mode is determined:

$$\bar{\lambda}_p = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr}}}$$

The reduction buckling factor ρ is determined according to Annex B of EN 1993-1-5. The reduction factor depends on the plate slenderness. The used buckling curve shows the influence of reduction factor on the plate slenderness. The provided buckling factor applicable to non-uniform members is based on the buckling curves of a beam. The verification is based on the von Mises yield criterion and the reduced stress method. Buckling resistance is assessed as

$$\frac{\alpha_{ult,k}\rho}{\gamma_{M2}} \geq 1$$



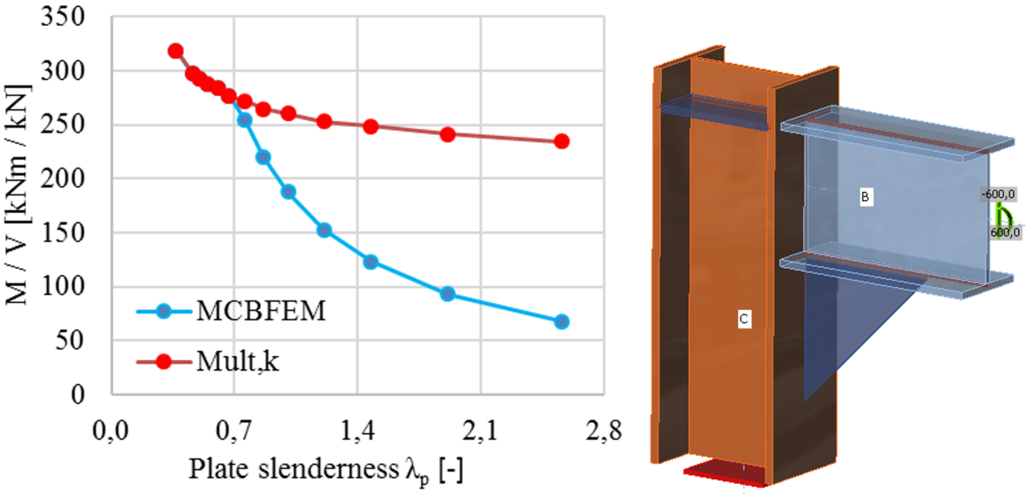
Buckling reduction factor ρ according to EN 1993-1-5 Annex B

Although the process seems trivial, it is general, robust and easily automated. The advantage of the procedure is the advanced FEM analysis of the whole joint which can be applied to general geometry. Moreover, it is included in the valid Eurocode standards. The advanced numerical analysis gives a quick overview of the global behaviour of the structure and its critical parts and allows fast stiffening to prevent instabilities.

The limit slenderness, λ_p, is provided in Annex B of EN 1993-1-5 and sets all cases which must be assessed according to the previous procedure. The resistance is limited by buckling for plate slenderness higher than 0.7. With the decreasing slenderness, the resistance is governed by plastic strain. The limit critical buckling factor for plate slenderness equals to 0.7 and buckling resistance equals to the plastic resistance may be obtained as follows

$$\alpha_{cr} = \frac{\alpha_{ult,k}}{\bar{\lambda}_p^2} = \frac{1}{0.7^2} = 2.04$$

The influence of the plate slenderness on the plastic resistance, $M_{ult,k}$, and buckling resistance, M_{CBFEM}, is shown in the figure bellow. The diagram shows the results of a numerical study of a triangular stiffener in a portal frame joint.



The influence of plate slenderness on the resistance of portal frame joint with slender stiffener

4.10 Deformation capacity

The prediction of the deformation capacity, δ_{Cd}, of connections is currently studied by the Component Method (CM) but is not offered as the standardized procedure. Compared to the well-accepted methods for determination of the initial stiffness and resistance of many types of structural joints, there are no generally accepted standardized procedures for the determination of the rotation capacity. The criteria which should be satisfied are selected to help engineers in Cl 6.4 of EN1993-1-8:2006.

A beam-to-column joint in which the design moment resistance of the joint, $M_{j,Rd}$, is governed by the design resistance of the column web panel in shear may be assumed to have adequate rotation capacity for plastic global analysis, provided that

$d / t_w \leq 69\,\varepsilon$

where d is the column web panel width, t_w is the web thickness and $\epsilon \leq \sqrt{235/f_y}$ is the steel yield strength ratio.

In Cl 6.4.2(2), the plastic distribution between the bolt rows is limited for joints with a bolted end-plate connection provided that the design moment resistance of the joint is governed by the design resistance of the column flange or the beam end-plate or tension flange cleat in bending. The thickness t of either the column flange or the beam end-plate or tension flange cleat should satisfy:

$$t \leq 0.36d\sqrt{\frac{f_{ub}}{f_y}}$$

where d and f_{ub} are the diameter and strength of the bolt and f_y is the yield strength of the relevant plate.

Clause 6.4.3 states that the rotation capacity, φ_{Cd}, of a welded beam-to-column connection may be assumed to be not less than the value given by the following expression provided that its column web is stiffened in compression but unstiffened in tension and its design moment resistance is not governed by the design shear resistance of the column web panel, see 6.4.2(1):

$\varphi_{Cd} = 0.025\, h_c / h_b$

where h_b is the depth of the beam and h_c is the depth of the column. An unstiffened welded beam-to-column joint designed in conformity with the provisions of this section may be assumed to have a rotation capacity φ_{Cd} of at least 0.015 radians.

4.11 Detailing

Detailing checks of bolts is performed if the option is selected in Code setup. Dimensions from bolt centre to plate edges and between bolts is checked. Edge distance e = 1.2 and spacing between bolts p = 2.2 are recommended in Table 3.3 in EN 1993-1-8. A user can modify both values in Code setup.

5 Check of components according to AISC

CBFEM method combines the advantages of general Finite Element Method and standard Component Method. The stresses and internal forces calculated on the accurate CBFEM model are used in checks of all components.

Individual components are checked according to American Institute of Steel Construction (AISC) 360-16.

5.1 Plates

The resulting equivalent stress (HMH, von Mises) and plastic strain are calculated on plates. When the yield strength (in LRFD multiplied by material resistance factor ϕ = 0.9, in ASD divided by material safety factor Ω = 1.67, which are editable in Code setup) on the bilinear material diagram is reached, the check of the equivalent plastic strain is performed. The limit value of 5 % is suggested in Eurocode (EN1993-1-5 App. C, Par. C8, Note 1). This value can be modified in Code setup but verification studies were made for this recommended value.

Plate element is divided into 5 layers and elastic/plastic behavior is investigated in each of them. The program shows the worst result from all of them.

Check of members and steel plates for extreme load effect

	Status	Item	Th [in]	Loads	σ,Ed [ksi]	ε,Pl [%]
>	✓	C-bfl 1	9/16"	LE1	32.6	0.8
	✓	C-tfl 1	9/16"	LE1	32.9	1.8
	✓	C-w 1	9/16"	LE1	33.3	3.2
	✓	B-bfl 1	5/8"	LE1	32.5	0.4
	✓	B-tfl 1	5/8"	LE1	32.3	0.1
	✓	B-w 1	9/16"	LE1	32.5	0.2
	✓	STIFF1a	3/8"	LE1	32.6	0.6
	✓	STIFF1b	3/8"	LE1	32.6	0.6
	✓	STIFF1c	3/8"	LE1	32.8	1.3
	✓	STIFF1d	3/8"	LE1	32.8	1.3

Design data

	Grade	Fy [ksi]	ε,lim [%]
>	A36	36.0	5.0

The CBFEM method can provide stress a little bit higher than yield strength. The reason is the slight inclination of the plastic branch of the stress-strain diagram, which is used in the analysis to improve the stability of the interaction calculation. This is not a problem for practical design. The equivalent plastic strain is exceeded at higher stress and the joint does not satisfy anyway.

5.2 Welds

5.2.1 Fillet welds

The design strength, ϕR_n, and the allowable strength, R_n/Ω, of welded joints are evaluated in the connection weld check.

$\phi = 0.75$ (Load and Resistance Factor Design, LRFD, editable in Code setup)

$\Omega = 2.00$ (Allowable Strength Design, ASD, editable in Code setup)

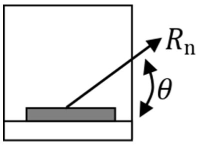
Available strength of welded joints is evaluated according to AISC 360-16 – J2.4

$R_n = F_{nw}\, A_{we}$

$F_{nw} = 0.6\, F_{EXX}\, (1.0 + 0.5 \sin^{1.5}\theta\,)$

where:

- F_{nw} – nominal stress of weld material
- A_{we} – effective area of the weld
- F_{EXX} – electrode classification number, i.e., minimum specified tensile strength
- θ – angle of loading measured from the weld longitudinal axis, degrees



For end-loaded fillet welds with a length up to 100 times the weld size, it is permitted to take the effective length equal to the actual length. When the length of the end-loaded fillet weld exceeds 100 times the weld size, the effective length shall be determined by multiplying the actual length by the reduction factor, β, determined as follows:

$\beta = 1.2 - 0.002\ (l\ /\ w)$

- l – weld length
- w – size of weld leg

When the length of the weld exceeds 300 times the leg size, w, the effective length is taken as 180 w.

Base metal strength is evaluated if the option is selected in Code setup (Base metal capacity at the fusion face).

$R_n = F_{nBM}\, A_{BM}$ – AISC 360-16 – J2.4 (J2-2)

where:

- $F_{nBM} = 0.6\, F_u$ – nominal strength of the base metal – AISC 360-16 – J4.2 (J4-4)
- $A_{BM} = A_{we}\sqrt{2}$ – cross-sectional area of the base metal
- F_u – specified minimum tensile strength

All values required for check are printed in tables.

Check of welds for extreme load effect (Plastic redistribution)

	Status	Item	Edge	Xu	Th [mm]	Ls [mm]	L [mm]	Lc [mm]	Loads	Fn [kN]	φRn [kN]	Ut [%]
−		B1-bfl 1	SPL1	E70xx	◢2,8	◢4,0	59	10	LE1	8,1	9,1	88,7
+		B2-bfl 1	SPL1	E70xx	◢2,8	◢4,0	59	10	LE1	8,1	9,1	88,7
+		B1-bfl 1	SPL3	E70xx	◢2,8	◢4,0	59	10	LE1	8,1	9,1	88,7
+		B2-bfl 1	SPL3	E70xx	◢2,8	◢4,0	59	10	LE1	8,1	9,1	88,7

Weld resistance check
AISC 360-16: J2-4
$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} =$ 9,1 kN $\geq$ $F_n =$ 8,1 kN
Where:
$F_{nw} =$ 434,4 MPa – nominal stress of weld material:

- $F_{nw} = 0,6 \cdot F_{EXX} \cdot (1 + 0,5 \cdot sin^{1,5}\theta)$, where:
  - $F_{EXX} =$ 482,6 MPa – electrode classification number, i.e. minimum s
  - $\theta =$ 89,4° – angle of loading measured from the weld longitudinal axi

$A_{we} =$ 28 mm² – effective area of weld's critical element

5.2.2 CJP groove welds

AISC Specification Table J2.5 identifies four loading conditions that might be associated with groove welds and shows that the strength of the joint is either controlled by the base metal or that the loads need not be considered in the design of the welds connecting the parts. Accordingly, when Complete Joint Penetration (CJP) groove welds are made with matching-strength filler metal, the strength of a connection is governed or controlled by the base metal and no checks on the weld strength are required.

5.3 Bolts

5.3.1 Tensile and shear strength of bolts

The design tensile or shear strength, ϕR_n, and the allowable tensile or shear strength, R_n/Ω of a snug-tightened bolt is determined according to the limit states of tension rupture and shear rupture as follows:

$R_n = F_n A_b$

$\phi = 0.75$ (LRFD, editable in Code setup)

$\Omega = 2.00$ (ASD, editable in Code setup)

where:

A_b – nominal unthreaded body area of bolt or threaded part

F_n – nominal tensile stress, F_{nt}, or shear stress, F_{nv}, from Table J3.2

The required tensile strength includes any tension resulting from prying action produced by the deformation of the connected parts.

5.3.2 Combined Tension and shear in bearing type connection

The available tensile strength of a bolt subjected to combined tension and shear is determined according to the limit states of tension and shear rupture as follows:

$R_n = F'_{nt} A_b$ (AISC 360-16 J3-2)

$\phi = 0.75$ (LRFD, editable in Code setup)

$\Omega = 2.00$ (ASD, editable in Code setup)

$$F'_{nt} = 1.3F_{nt} - \frac{f_{rv}F_{nt}}{\phi F_{nv}}$$ (AISC 360-16 J3-3a LRFD)

$$F'_{nt} = 1.3F_{nt} - \frac{f_{rv}\Omega F_{nt}}{F_{nv}}$$ (AISC 360-16 J3-3b ASD)

where:

- F'_{nt} – nominal tensile stress modified to include the effects of shear stress
- F_{nt} – nominal tensile stress from AISC 360-16 Table J3.2
- F_{nv} – nominal shear stress from AISC 360-16 Table J3.2
- f_{rv} – required shear stress using LRFD or ASD load combinations. The available shear stress of the fastener shall be equal or exceed the required shear stress, f_{rv}

5.3.3 Bearing strength in bolt holes

The available bearing strengths, ϕR_n and R_n/Ω, at bolt holes are determined for the limit state of bearing as follows:

$\phi = 0.75$ (LRFD, editable in Code setup)

$\Omega = 2.00$ (ASD, editable in Code setup)

The nominal bearing strength of the connected material, R_n, is determined as follows:

For a bolt in a connection with standard holes:

$R_n = 1.2\, l_c\, t\, F_u \le 2.4\, d\, t\, F_u$ (AISC 360-16 J3-6a, J3-6a, c)

For a bolt in a connection with slotted holes:

$R_n = 1.0\, l_c\, t\, F_u \le 2.0\, d\, t\, F_u$ (AISC 360-16 J3-6a, J3-6e, f)

where:

- F_u – specified minimum tensile strength of the connected material
- d – nominal bolt diameter
- l_c – clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
- t – thickness of the connected material

5.4 Preloaded bolts

The design slip resistance of preloaded class A325 or A490 bolt with the effect of tensile force Ft

Preloading force to be used AISC 360-10 tab. J3.1.

$T_b = 0.7\, f_{ub}\, A_s$

Design slip resistance per bolt AISC 360-10 par. J3.8

$R_n = k_{SC}\, \mu\, D_u\, h_f\, T_b\, n_s$

Utilization in shear [%]:

$U_{ts} = V / \phi R_n$ (LRFD)

$U_{ts} = \Omega\, V / R_n$ (ASD)

where:

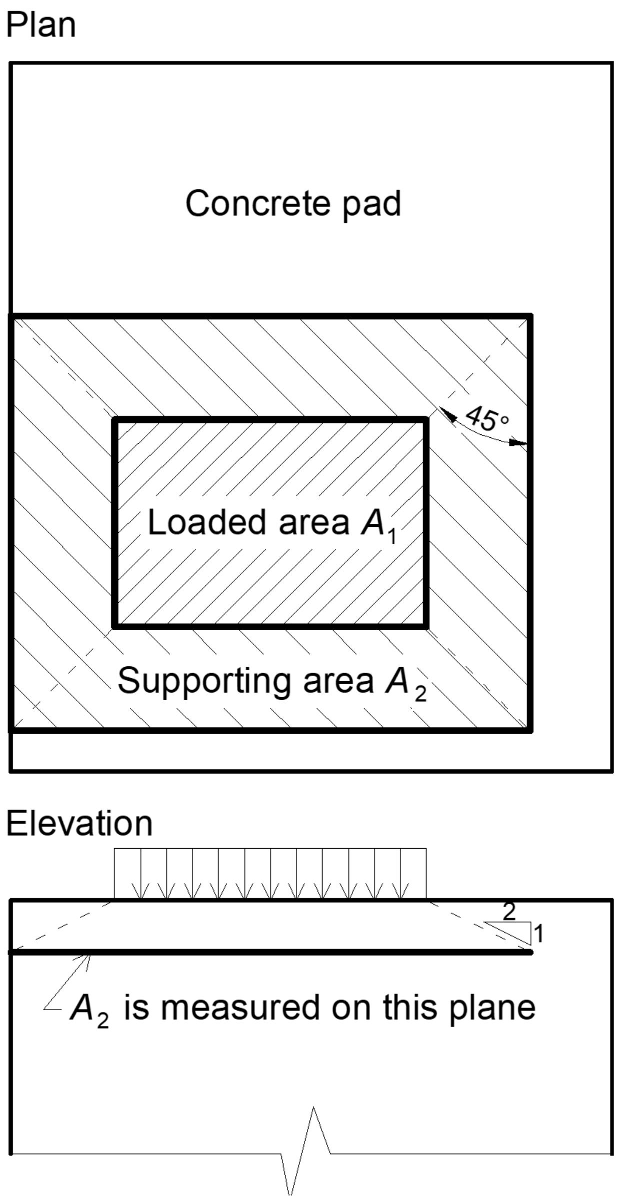
- A_s – tensile stress area of the bolt
- f_{ub} – ultimate tensile strength
- $k_{SC} = 1 - \frac{F_t}{D_u T_b n_b}$ – factor for combined tension and shear (LRFD) (J3-5a)
- $k_{SC} = 1 - \frac{1.5 F_t}{D_u T_b n_b}$ – factor for combined tension and shear (ASD) (J3-5b)
- μ – mean slip factor coefficient editable in Code setup
- $D_u = 1.13$ – multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension
- $h_f = 1.0$ – factor for fillers
- n_s – number of the friction surfaces; Check is calculated for each friction surface separately
- V – shear force acting on the bolt

- ϕ = 1.0 – resistance factor for standard size holes (LRFD) editable in Code setup
- ϕ = 0.7 – resistance factor for slotted holes (LRFD)
- Ω = 1.5 – resistance factor for standard size holes (ASD) editable in Code setup
- Ω = 2.14 – resistance factor for slotted holes (ASD)

5.5 Concrete in compression

Concrete design bearing strength in compression is designed according to AISC 360-16, Section J8. When the supported surface of the concrete is larger than the base plate the design bearing strength is defined as

$$f_{p(max)} = 0.85 f_c \sqrt{\frac{A_2}{A_1}} \leq 1.7 f_c'$$



where:

- f'_c – concrete compressive strength
- A_1 – base plate area in contact with concrete surface (upper surface area of the frustum)
- A_2 – concrete supporting surface (geometrically similar lower area of the frustum having its slopes of 1 vertical to 2 horizontal)

The assessment of concrete in bearing is as follows

$\sigma \leq \phi_c\, f_{p(max)}$ for LRFD

$\sigma \leq f_{p(max)} / \Omega_c$ for ASD

where:

- σ – average compressive stress under the base plate
- ϕ_c = 0.65 – resistance factor for concrete
- Ω_c = 2.31 – safety factor for concrete

5.6 Transfer of shear forces

Shear loads can be transferred via one of these options:

- Shear lug,
- Friction,
- Anchor bolts.

5.6.1 Shear lug

Only LFRD is available. The shear load is transferred via the shear lug. The concrete in bearing and, unless reinforcement is provided to develop the required strength, concrete breakout checks are necessary.

The **bearing capacity** of shear lug against concrete is determined according to ACI 349-01 – B.4.5 and ACI 349-01 RB11 as:

$$\phi P_{br} = \phi \, 1.3 \, f'_c \, A_1 + \phi \, K_c \, (N_y - P_a)$$

where:

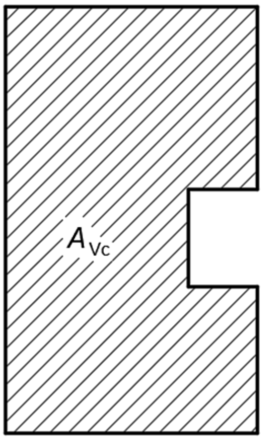
- $\phi = 0.7$ – strength reduction factor for bearing on concrete according to ACI 349
- f'_c – concrete compressive strength
- A_1 – projected area of the embedded shear lug in the direction of the force excluding the portion of the lug in contact with the grout above concrete member
- $K_c = 1.6$ – confinement coefficient
- $N_y = n \, A_{se} \, F_y$ – yield strength of tensioned anchors
- P_a – external axial load

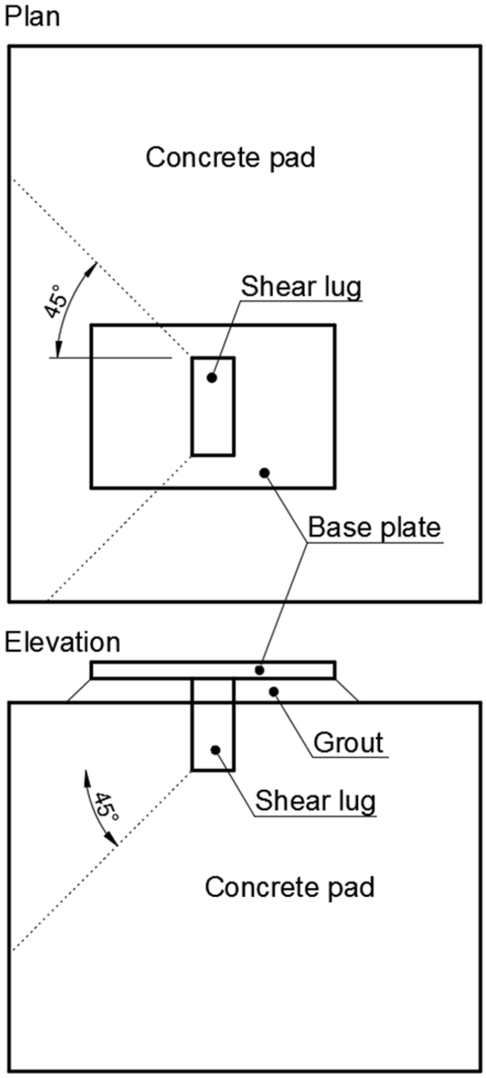
The **concrete breakout strength** of the shear lug according to ACI 349 – B11 is:

$$\phi V_{cb} = A_{Vc} 4 \phi \sqrt{f_c'}$$

where:

- ϕ = 0.85 – strength reduction factor for shear according to ACI 349
- A_{Vc} – effective stress area defined by projecting a 45° plane from the bearing edges of the shear lug to the free surface in the direction of the shear load. The bearing area of the shear lug is excluded from the projected area





If the concrete breakout resistance in Code setup is disabled, user is provided with the force that needs to be transferred via reinforced concrete.

5.6.2 Friction

The shear load is transferred via friction. The shear resistance is determined as:

$\phi_c V_r = \phi_c \mu C$ (LRFD)

$V_r / \Omega_c = \mu C / \Omega_c$ (ASD)

where:

- ϕ_c = 0.65 – resistance factor (LRFD)
- Ω_c = 2.31 – safety factor (ASD)

- μ = 0.4 – coefficient of friction between base plate and concrete (recommended value 0.4 in AISC Design guide 7 – 9.2 and ACI 349 – B.6.1.4, editable in Code setup)
- C – compressive force

5.6.3 Anchor bolts

If the shear load is transferred via anchor bolts only, the shear force acting on each anchor is determined by FEA and anchor bolts are assessed according to ACI 318-14 as described in following chapters.

5.7 Anchors

Only LFRD is available. Anchor rods are designed according to AISC 360-16 – J9 and ACI 318-14 – Chapter 17. The following resistances of anchor bolts are evaluated:

- Steel strength of anchor in tension ϕN_{sa},
- Concrete breakout strength in tension ϕN_{cbg},
- Concrete pullout strength ϕN_{p},
- Concrete side-face blowout strength ϕN_{sb},
- Steel strength of anchor in shear ϕV_{sa},
- Concrete breakout strength in shear ϕV_{cbg},
- Concrete pryout strength of anchor in shear ϕV_{cp}.

The user must choose the concrete condition (cracked or uncracked – with no cracks in service condition) and the type of anchors (cast-in headed, cast-in hooked or post-installed).

5.7.1 Steel strength of anchor in tension

Steel strength of anchor in tension is determined according to ACI 318-14 – 17.4.1 as

$\phi N_{sa} = \phi\, A_{se,N}\, f_{uta}$

where:

- ϕ = 0.7 – strength reduction factor for anchors in tension according to ACI 318-14 – 17.3.3, the factor is editable in Code setup
- $A_{se,N}$ – tensile stress area

- f_{uta} – specified tensile strength of anchor steel and shall not be greater than 1.9 f_{ya} and 120 ksi

5.7.2 Concrete breakout strength

Concrete breakout strength is designed according to the Concrete Capacity Design (CCD) in ACI 318-14 – Chapter 17. In the CCD method, the concrete cone is considered to be formed at an angle of approximately 34° (1 vertical to 1.5 horizontal slope). For simplification, the cone is considered to be square rather than round in plan. The concrete breakout stress in the CCD method is considered to decrease with an increase in size of the breakout surface. Consequently, the increase in strength of the breakout in the CCD method is proportional to the embedment depth to the power of 1.5. Anchors whose concrete cones overlap create a group of anchors which create a common concrete cone. Note that no equivalent ASD solution exists for concrete capacity design.

$$\phi N_{cbg} = \phi \frac{A_{Nc}}{A_{Nco}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$$

where:

- ϕ = 0.7 – strength reduction factor for anchors in tension according to ACI 318-14 – 17.3.3, the factor is editable in Code setup
- A_{Nc} – actual concrete breakout cone area for a group of anchors that create common concrete cone
- $A_{Nco} = 9\ h_{ef}^2$ – concrete breakout cone area for single anchor not influenced by edges
- $\psi_{ec,N} = \frac{1}{1+\frac{2e'_N}{3h_{ef}}}$ – modification factor for anchor groups loaded eccentrically in tension; in the case where eccentric loading exists about two axes, the modification factor $\Psi_{ec,N}$ is calculated for each axis individually and the product of these factors is used
- $\psi_{ed,N} = \min\left(0.7 + \frac{0.3c_{a,min}}{1.5h_{ef}}, 1\right)$ – modification factor for edge distance
- $c_{a,min}$ – smallest distance from the anchor to the edge
- $\Psi_{c,N}$ – modification factor for concrete conditions ($\Psi_{c,N}$ =1 for cracked concrete),

- $\Psi_{cp,N}$ = min ($c_{a,min}$ / c_{ac},1) – modification factor for splitting for post-installed anchors designed for uncracked concrete without supplementary reinforcement to control splitting; $\Psi_{cp,N}$ = 1 for all other cases
- $N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ – basic concrete breakout strength of a single anchor in tension in cracked concrete; for cast-in anchors and 11 in. ≤ h_{ef} ≤ 25 in. $N_b = 16 \lambda_a \sqrt{f'_c} h_{ef}^{5/3}$
- k_c = 24 for cast-in anchors
- h_{ef} – depth of embedment; according to Chapter 17.4.2.3 in ACI 318-14, the effective embedment depth h_{ef} is reduced to $h_{ef} = \max\left(\frac{c_{a,max}}{1.5}, \frac{s}{3}\right)$ if anchors are located less than 1.5 h_{ef} from three or more edges
- s – spacing between anchors
- $c_{a,max}$ – maximum distance from an anchor to one of the three close edges
- λ_a = 1 – modification factor for lightweight concrete
- f'_c – concrete compressive strength [psi]

According to ACI 318-14 – 17.4.2.8, in case of headed anchors, the projected surface area A_{Nc} is determined from the effective perimeter of the washer plate, which is the lesser value of d_a + 2 t_{wp} or d_{wp}, where:

- d_a – anchor diameter
- d_{wp} – washer plate diameter or edge size
- t_{wp} – washer plate thickness

The group of anchors is checked against the sum of tensile forces in anchors loaded in tension and creating a common concrete cone.

According to ACI 318-14 – 17.4.2.9, where anchor reinforcement is developed in accordance with ACI 318-14 – 25 on both sides of the breakout surface, the anchor reinforcement is presumed to transfer the tension forces and concrete breakout strength is not evaluated.

5.7.3 Concrete pullout strength

Concrete pullout strength of an anchor is defined in ACI 318-14 – 17.4.3 as

$\phi N_{pn} = \phi \Psi_{c,P} N_p$

where:

- ϕ = 0.7 – strength reduction factor for anchors in tension according to ACI 318-14 – 17.3.3, editable in Code setup
- $\Psi_{c,P}$ – modification factor for concrete condition, $\Psi_{c,P}$ = 1.0 for cracked concrete
- $N_P = 8\ A_{brg}\ f'_c$ for headed anchor
- A_{brg} – bearing area of the head of stud or anchor bolt
- f'_c – concrete compressive strength

Concrete pullout strength for other types of anchors than headed is not evaluated in the software and has to be specified by the manufacturer.

5.7.4 Concrete side-face blowout strength

Concrete side-face blowout strength of headed anchor in tension is defined in ACI 318-14 – 17.4.4 as

$$\phi N_{sb} = \phi 160 c_{a1} \sqrt{A_{brg}} \sqrt{f'_c}$$

The concrete side-face blowout strength is multiplied by one of reduction factors:

- $\frac{1+\frac{c_{a2}}{c_{a1}}}{4} \leq 1$
- $\frac{1+\frac{s}{6c_{a1}}}{2} \leq 1$

where:

- ϕ = 0.7 – strength reduction factor for anchors in tension according to ACI 318-14 – 17.3.3, editable in Code setup
- c_{a1} – shorter distance from the centreline of an anchor to an edge
- c_{a2} – longer distance, perpendicular to c_{a1}, from the centreline of an anchor to an edge
- A_{brg} – bearing area of the head of stud or anchor bolt
- f'_c – concrete compressive strength
- h_{ef} – depth of embedment; according to Chapter 17.4.2.3 in ACI 318-14, the effective embedment depth h_{ef} is reduced to

$h_{ef} = \max\left(\frac{c_{a,max}}{1.5}, \frac{s}{3}\right)$ if anchors are located less than 1.5 h_{ef} from three or more edges

- s – spacing between two adjacent anchors near one edge

5.7.5 Steel strength in shear

The steel strength in shear is determined according to ACI 318-14 – 17.5.1 as

$\phi V_{sa} = \phi\ 0.6\ A_{se,V}\ f_{uta}$

where:

- ϕ = 0.65 – strength reduction factor for anchors in tension according to ACI 318-14 – 17.3.3, editable in Code setup
- $A_{se,V}$ – tensile stress area
- f_{uta} – specified tensile strength of anchor steel and shall not be greater than 1.9 f_{ya} and 120 ksi

If mortar joint is selected, steel strength in shear V_{sa} is multiplied by 0.8 (ACI 318-14 – 17.5.1.3).

The shear on lever arm, which is present in case of base plate with oversized holes and washers or plates added to the top of the base plate to transmit the shear force, is not considered.

5.7.6 Concrete breakout strength of anchor in shear

Concrete breakout strength of an anchor or anchor group in shear is designed according to ACI 318 14 – 17.5.2.

$$\phi V_{cbg} = \phi \frac{A_V}{A_{Vo}} \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{h,V} \psi_{\alpha,V} V_b$$

where:

- ϕ = 0.65 – strength reduction factor for anchors in shear according to ACI 318-14 – 17.3.3, editable in Code setup
- A_V – projected concrete failure area of an anchor or group of anchors
- A_{Vo} – projected concrete failure area of one anchor when not limited by corner influences, spacing or member thickness

- $\psi_{ec,V} = \frac{1}{1+\frac{2e'_V}{3c_{a1}}}$ – modification factor for anchor groups loaded eccentrically in shear
- $\psi_{ed,V} = 0.7 + 0.3\frac{c_{a2}}{1.5c_{a1}} \leq 1.0$ – modification factor for edge effect
- $\Psi_{c,V}$ – modification factor for concrete condition; $\Psi_{c,V}$ = 1.0 for cracked concrete
- $\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1$ – modification factor for anchors located in a concrete member where $h_a < 1.5\ c_{a1}$
- $\psi_{\alpha,V} = \sqrt{\frac{1}{(\cos\alpha_V)^2+(0.5\sin\alpha_V)^2}}$ – modification factor for anchors loaded at an angle $90° - \alpha_V$ with the concrete edge; in ACI 318-14 – 17.5.2.1 are only discrete values, equation is taken from FIB bulletin 58 – Design of anchorages in concrete (2011)
- h_a – height of a failure surface on the concrete side
- $V_b = \min\left(7\left(\frac{l_e}{d_a}\right)^{0.2}\lambda_a\sqrt{d_a}\sqrt{f'_c}c_{a1}^{1.5}, 9\lambda_a\sqrt{d_a}\sqrt{f'_c}c_{a1}^{1.5}\right)$
- $l_e = h_{ef} \leq 8\ d_a$ – load-bearing length of the anchor in shear
- d_a – anchor diameter
- f'_c – concrete compressive strength
- c_{a1} – edge distance in the direction of load
- c_{a2} – edge distance in the direction perpendicular to load

If $c_{a2} \leq 1.5\ c_{a1}$ and $h_a \leq 1.5\ c_{a1}$, $c_{a1} = \max\left(\frac{c_{a2}}{1.5}, \frac{h_a}{1.5}, \frac{s}{3}\right)$, where s is the maximum spacing perpendicular to direction of shear, between anchors within a group.

According to ACI 318-14 – 17-5.2.9, where anchor reinforcement is developed in accordance with ACI 318-14 – 25 on both sides of the breakout surface, the anchor reinforcement is presumed to transfer the shear forces and concrete breakout strength is not evaluated.

5.7.7 Concrete pryout strength of anchor in shear

Concrete pryout strength is designed according to ACI 318-14 – 17.5.3.

$\phi V_{cp} = \phi k_{cp}\ N_{cp}$

where:

- ϕ = 0.65 – strength reduction factor for anchors in shear according to ACI 318-14 – 17.3.3, editable in Code setup
- k_{cp} = 1.0 for h_{ef} < 2.5 in., k_{cp} = 2.0 for $h_{ef} \geq$ 2.5 in
- N_{cp} = N_{cb} (concrete breakout strength – all anchors are assumed in tension) in case of cast-in anchors

According to ACI 318-14 – 17.4.2.9, where anchor reinforcement is developed in accordance with ACI 318-14 – 25 on both sides of the breakout surface, the anchor reinforcement is presumed to transfer the tension forces and concrete breakout strength is not evaluated.

5.7.8 Interaction of tensile and shear forces

Interaction of tensile and shear forces is assessed according to ACI 318-14 – R17.6.

$$\left(\frac{N_{ua}}{N_n}\right)^{\zeta} + \left(\frac{V_{ua}}{V_n}\right)^{\zeta} \leq 1.0$$

where:

- N_{ua} and V_{ua} – design forces acting on an anchor
- N_n and V_n – the lowest design strengths determined from all appropriate failure modes
- ς = 5 / 3

5.7.9 Anchors with stand-off

The bar element is designed according to AISC 360-16. Interaction of shear force is neglected because the minimum length of the anchor to fit the nut under the base plate ensures that the anchor fails in bending before the shear force reaches half the shear resistance and the shear interaction is negligible (up to 7 %). Interaction of bending moment and compressive or tensile force is conservatively assumed as linear. Second order effects are not taken into account.

Shear resistance (AISC 360-16 – G):

$$V_n = \frac{0.6 A_V F_y}{\Omega_V} \quad \text{(ASD)}$$

$$V_n = \phi_V 0.6 A_V F_y \quad \text{(LRFD)}$$

- $A_V = 0.844 \cdot A_s$ – the shear area
- A_s – bolt area reduced by threads
- F_y – bolt yield strength
- Ω_V – safety factor, recommended value is 1.67
- ϕ_V – resistance factor, recommended value is 0.9

Tensile resistance (AISC 360-16 – D2):

$P_n = \frac{A_s F_y}{\Omega_t}$ (ASD)

$P_n = \phi_t A_s F_y$ (LRFD)

- Ω_t – safety factor, recommended value is 1.67
- ϕ_t – resistance factor, recommended value is 0.9

Compressive resistance (AISC 360-16 – E3)

$P_n = \frac{F_{cr} A_s}{\Omega_c}$ (ASD)

$P_n = \phi_c F_{cr} A_s$ (LRFD)

- $F_{cr} = 0.658^{\frac{F_y}{F_e}} F_y$ for $\frac{L_c}{r} \leq 4.74\sqrt{\frac{E}{F_y}}$, $F_{cr} = 0.877 F_e$ for $\frac{L_c}{r} > 4.74\sqrt{\frac{E}{F_y}}$ – critical stress
- $F_e = \frac{\pi^2 E}{\left(\frac{L_c}{r}\right)^2}$ – elastic buckling stress
- $L_c = 2 \cdot l$ – buckling length
- l – length of the bolt element equal to half the base plate thickness + gap + half the bolt diameter
- $r = \sqrt{\frac{I}{A_s}}$ – radius of gyration of the anchor bolt
- $I = \frac{\pi d_s^4}{64}$ – moment of inertia of the bolt,
- Ω_c – safety factor, recommended value is 1.67,
- ϕ_c – resistance factor, recommended value is 0.9.

Bending resistance (AISC 360-16 – F11):

$$M_n = \frac{ZF_y}{\Omega_b} \le \frac{1.6 S_x F_y}{\Omega_b} \quad \text{(ASD)}$$

$$M_n = \phi_b Z F_y \le 1.6 \phi_b S_x F_y \quad \text{(ASD)}$$

- $Z = \frac{d_s^3}{6}$ – plastic section modulus of the bolt
- $S_x = \frac{2I}{d_s}$ – elastic section modulus of the bolt
- Ω_c – safety factor, recommended value is 1.67
- ϕ_c – resistance factor, recommended value is 0.9

Linear interaction:

$$\frac{N}{P_n} + \frac{M}{M_n} \le 1$$

- N – the tensile (positive) or compressive (negative sign) factored force
- P_n – the tensile (positive) or compressive (negative sign) design or allowable strength
- M – the factored bending moment
- M_n – the design or allowable bending resistance

5.8 Member capacity design

Member capacity design is performed according to AISC 341-10

$M_{pe} = 1.1\ R_y\ F_y\ Z_x$

where:

- M_{pe} – the expected moment at the plastic hinge
- R_y – ratio of the expected yield stress to the specified minimum yield (Table A3.1)
- F_y – yield strength
- Z_x – the plastic section modulus

5.9 Detailing

The minimum spacing between **bolts** and distance to the bolt centre to an edge of a connected part are checked. The minimum spacing 2.66 times (editable in Code setup) the nominal bolt diameter between centres of bolts is checked according to AISC 360-16 – J.3.3. The minimum distance to the bolt centre to an edge of a connected part is checked according to AISC 360-16 – J.3.4; the values are in Table J3.4 and J3.4M.

The minimal and maximal **weld** size and the sufficient length of the weld are checked.

The maximal weld size is checked according to AISC 360-16 – J2.2b:

- For thinner plate thickness up to 3/16 in the weld size should be no bigger than plate thickness.
- For thinner plate thickness higher than 3/16 in and smaller than 1/4 in the weld size should be no bigger than 3/16 in.
- For thinner plate thickness higher than 1/4 in the weld size should be no bigger than 1/4−1/16 in.

The minimal weld size is checked according to Table J2.4:

- For thinner plate thickness to 1/4 in the weld size should be higher than or equal to 1/8 in.
- For thinner plate thickness over 1/4 in to 1/2 in the weld size should be higher than or equal to 3/16 in.
- For thinner plate thickness over 1/2 in to 3/4 in the weld size should be higher than or equal to 1/4 in.
- For thinner plate thickness over 3/4 in the weld size should be higher than or equal to 5/16 in.

The minimum length of fillet welds should not be less than four times the weld size according to J2.2b (c).

The spacing between **anchors** should be greater than 4 times anchor diameter according to ACI 318-14 – 17.7.1..

6 Check of components according to CISC

CBFEM method combines advantages of general finite element method and standard component method. The stresses and internal forces calculated on the accurate CBFEM model are used in checks of all components.

Components are designed according to Canadian standard (Canadian Institute of Steel Construction, CISC) S16-14 Design of steel structures and CSA A23.3 Design of concrete structures.

6.1 Plates

The resulting equivalent stress (HMH, von Mises) and plastic strain are calculated on plates. When the yield strength on the bilinear material diagram is reached, the check of the equivalent plastic strain is performed. The limit value of 5 % is suggested in Eurocode (EN1993-1-5 App. C, Par. C8, Note 1), this value can be modified in Code setup but verifications were done for the recommended value.

Plate element is divided into 5 layers and elastic/plastic behavior is investigated in each of them. The program shows the worst result from all of them.

6.2 Welds

6.2.1 Fillet welds

The resistance for direct shear and tension or compression induced shear is designed according to S16-14 – 13.13.2.2. Plastic redistribution in weld material is applied in Finite Element Modelling.

$$V_r = 0.67\phi_w A_w X_u (1 + 0.5\sin^{1.5}\theta) M_w$$

where:

- ϕ_w = 0.67 – resistance factor for weld metal, editable in Code setup
- A_w – area of effective weld throat
- X_u – ultimate strength as rated by the electrode classification number
- θ – angle of axis of weld segment with respect to the line of action of applied force (e.g., 0° for a longitudinal weld and 90° for a transverse weld)
- $M_w = \frac{0.85+\theta_1/600}{0.85+\theta_2/600}$ – strength reduction factor for multi-orientation fillet welds; equals to 1.0 in IDEA and the resistance of multi-orientation welds is determined by FEA where the most stressed element is assessed
- θ_1 – orientation of the weld segment under consideration
- θ_2 – orientation of the weld segment in the joint that is nearest to 90°

Base metal capacity at the fusion face:

$$V_r = 0.67\phi_w A_m F_u$$

where:

- $A_m = z\,L$ – area of the fusion face
- z – leg size of the weld
- L – length of the weld
- F_u – specified tensile strength

6.2.2 CJP groove welds

The resistance of Complete Joint Penetration (CJP) groove welds is assumed as that of the base metal.

6.3 Bolts

6.3.1 Tensile strength of bolts

The tensile resistance of a bolt is assessed according to Clause 13.12.1.3 and taken as:

$$T_r = 0.75\phi_b A_b F_u$$

where:

- ϕ_b = 0.8 – resistance factor for bolts, editable in Code setup
- A_b – cross-sectional area of a bolt based on its nominal diameter
- F_u – specified minimum tensile strength for a bolt

When the bolt threads are intercepted by a shear plane, the shear resistance is taken as 0.7 V_r.

6.3.2 Shear strength of bolts

The shear resistance of a bolt is assessed according to Clause 13.12.1.2. Each shear plane of a bolt is checked separately. It is taken as:

$$V_r = 0.6\phi_b A_b F_u$$

where:

- ϕ_b = 0.8 – the resistance factor for bolts, editable in Code setup
- A_b – cross-sectional area of a bolt based on its nominal diameter
- F_u – specified minimum tensile strength for a bolt

When the bolt threads are intercepted by a shear plane, the shear resistance is taken as 0.7 V_r.

6.3.3 Combined tension and shear in bearing type connection

The resistance of a bolt loaded by combined tension and shear is assessed according to Clause 13.12.1.4 and taken as:

$$\left(\frac{V_f}{V_r}\right)^2 + \left(\frac{T_f}{T_r}\right)^2 \leq 1$$

where:

- V_f and T_f are design shear force and tensile force acting on the bolt, respectively
- V_r and T_r are design shear resistance and tensile resistance of the bolt, respectively

6.3.4 Bearing strength in bolt holes

The resistance developed at the bolt in a bolted joint subjected to bearing and shear is assessed according to Clause 13.12.1.2 and taken as

$B_r = 3\ \phi_{br}\ t\ d\ F_u$ for regular bolt holes

$B_r = 2.4\ \phi_{br}\ t\ d\ F_u$ for slotted holes loaded perpendicular to these holes

where:

- $\phi_{br} = 0.8$ – resistance factor for bearing of bolts on steel
- t – thinner thickness of connected plates
- d – diameter of a bolt
- F_u – tensile strength of the connected material

6.3.5 Hole tear-out of a bolt

The resistance of hole tear-out of a bolt is checked for individual bolts according to Clause 13.11 as:

$$T_r = \phi_u 0.6 A_{gv} \frac{F_y + F_u}{2}$$

where:

- $\phi_u = 0.75$ – resistance factor for structural steel
- $A_{gv} = 2 \cdot l \cdot t$ – gross area in shear
- F_y – yield strength of the connected material
- F_u – tensile strength of the connected material
- l – distance from centreline of the bolt to the edge in the direction of the shear force
- t – thickness of the connected material

For steel grades with F_y > 460 MPa, $(F_y + F_u) / 2$ shall be replaced with F_y in the determination of T_r.

6.3.6 Bolts in slip-critical connections

The slip resistance of a bolted joint is assessed according to Clause 13.12.2 as

$V_s = 0.53\, c_s\, k_s\, A_b\, F_u$

where:

- c_s – coefficient determined according to k_s and bolt grade:
- for $k_s < 0.52$ class A $c_s = 1.00$ (A325) or 0.92 (A490) or 0.78 (other)
- for $k_s \geq 0.52$ class B $c_s = 1.04$ (A325) or 0.96 (A490) or 0.81 (other)
- k_s – friction coef. editable in Code setup which should be set according to Table 3 in S16-14; equals 0.3 for class A or 0.52 for class B
- A_b – cross-sectional area of a bolt based on its nominal diameter
- F_u – specified minimum tensile strength for a bolt

When slotted holes are used in slip-critical connections, $V_s = 0.75 \cdot 0.53\, c_s\, k_s\, A_b\, F_u$.

A bolt subjected to both tension and shear must satisfy the following relationship:

$$\frac{V_f}{V_s} + 1.9 \frac{T}{A_b F_u}$$

where:

- V_f and T_f are the design shear force and the tensile force acting on the bolt, respectively

Clause 13.12.2 states that the resistances of the connection as specified in Clause 13.12.1 shall be checked. The user should, therefore, check the state after slip occurs, i.e. change the shear force transfer of bolts from “Friction” to “Bearing – tension and shear interaction”.

6.4 Concrete in compression

The concrete design bearing strength in compression is determined in accordance with S16-14 – 25.3.1 and CSA A23.3 – 10.8. When the supported surface of the concrete is larger than the base plate the design bearing strength is defined as

$$f_{p,(max)} = 0.85\phi_c f_c' \sqrt{\frac{A_2}{A_1}} \leq 1.7\phi_c f_c'$$

where:

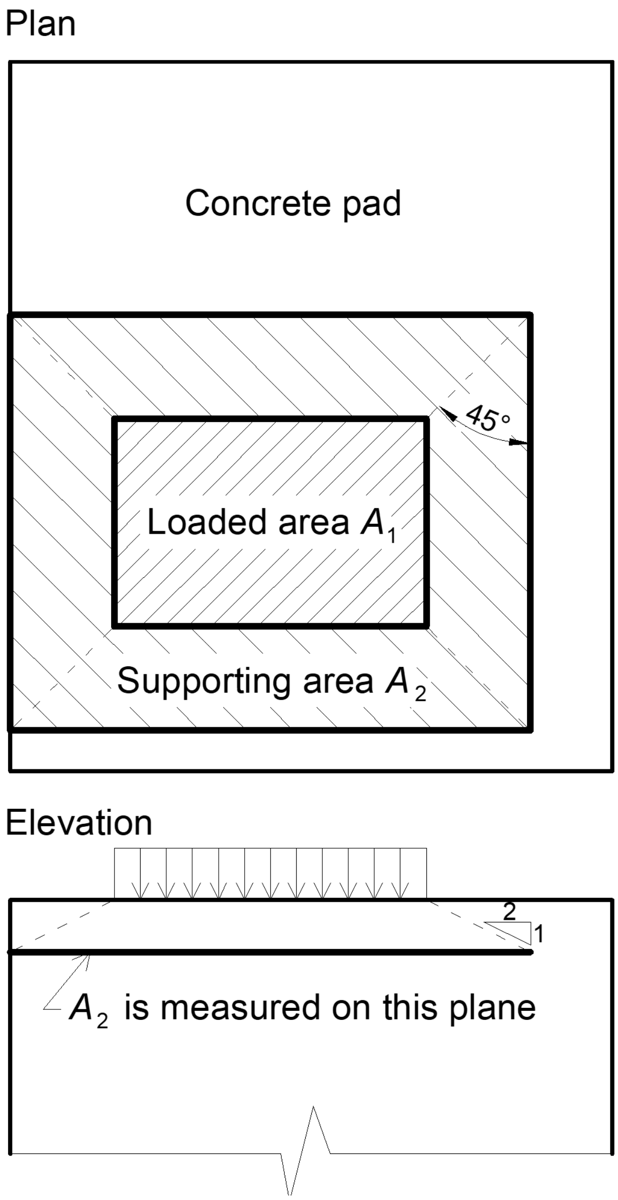
- ϕ_c=0.65 – resistance factor for concrete
- f'_c – concrete compressive strength
- A_1 – base plate area in contact with concrete surface (upper surface area of the frustum)
- A_2 – concrete supporting surface (geometrically similar lower area of the frustum having its slopes of 1 vertical to 2 horizontal)

The assessment of concrete in bearing is as follows:

$\sigma \leq f_{p(max)}$

where:

- σ – average compressive stress under the base plate



6.5 Transfer of shear forces

Shear loads can be transferred via one of these options:

- Shear lug,
- Friction,
- Anchor bolts.

6.5.1 Shear lug

Shear loads are considered to be transferred only via shear lug. Concrete bearing is not checked in software and should be checked by user elsewhere. Shear lug and welds are checked using FEM and weld components.

6.5.2 Friction

In case of compressive force, the shear loads can be transferred via friction between a concrete pad and a base plate. The friction coefficient is editable in Code setup.

6.5.3 Anchor bolts

If the shear load is transferred via anchor bolts only, the shear force acting on each anchor is determined by FEA and anchor bolts are assessed according to ACI 318-14 as described in following chapters.

6.6 Anchors

Anchor rods are designed according to A23.3-14 – Annex D. The following resistances of anchor bolts are evaluated:

- Steel strength of anchor in tension N_{sar},
- Concrete breakout strength in tension N_{cbr},
- Concrete pullout strength N_{pr},
- Concrete side-face blowout strength N_{sbr},
- Steel strength of anchor in shear V_{sar},
- Concrete breakout strength in shear V_{cbr},
- Concrete pryout strength of anchor in shear V_{cpr}.

The concrete condition is considered as cracked. The type of anchors (cast-in headed with circular or rectangular washers, straight anchors) is selected by user, the pullout strength and side-face blowout strength is checked in the software only for headed anchors.

6.6.1 Steel resistance of anchor in tension

Steel strength of anchor in tension is determined according to CSA A23.3-14 – D.6.1 as

$N_{sar} = A_{se,N}\ \phi_s\ f_{uta}\ R$

where:

- ϕ_s = 0.85 – steel embedment material resistance factor for reinforcement
- $A_{se,N}$ – effective cross-sectional area of an anchor in tension
- $f_{uta} \leq$ min (860 MPa, 1.9 f_{ya}) – specified tensile strength of anchor steel
- f_{ya} – specified yield strength of anchor steel
- R = 0.8 – resistance modification factor as specified in CSA A23.3.-14 – D.5.3

6.6.2 Concrete breakout resistance of anchor in tension

Concrete breakout strength is designed according to the Concrete Capacity Design (CCD) in CSA A23.3-14 – D.6.2. In the CCD method, the concrete cone is considered to be formed at an angle of approximately 34° (1 vertical to 1.5 horizontal slope). For simplification, the cone is considered to be square rather than round in plan. The concrete breakout stress in the CCD method is considered to decrease with an increase in size of the breakout surface.

$$N_{cbrg} = \frac{A_{Nc}}{A_{Nco}} \psi_{ed,N} \psi_{ec,N} \psi_{c,N} N_{br}$$

where:

- A_{Nc} – concrete breakout cone area for group of anchors loaded by tension that create common concrete cone
- $A_{Nco} = 9\ h_{ef}^2$ – concrete breakout cone area for single anchor not influenced by concrete edges
- $\psi_{ed,N} = \min\left(0.7 + \frac{0.3 c_{a,min}}{1.5 h_{ef}},\ 1\right)$ – modification factor for edge distance
- $c_{a,min}$ – the smallest distance from the anchor to the edge
- h_{ef} – depth of embedment; according to A23.3-14 – D.6.2.3, the effective embedment depth h_{ef} is reduced to $h_{ef} = \max\left(\frac{c_{a,max}}{1.5},\ \frac{s}{3}\right)$ if anchors are located less than 1.5 h_{ef} from three or more edges

- $\psi_{ec,N} = \frac{1}{1+\frac{2e'_N}{3h_{ef}}}$ – modification factor for eccentrically loaded group of anchors
- e'_N – tension load eccentricity with respect to the center of gravity of anchors loaded by tension and creating a common concrete cone
- $\Psi_{c,N}$ – modification factor for concrete conditions ($\Psi_{c,N}$ = 1 for cracked concrete)
- $N_{br} = k_c \phi_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5} R$ – basic concrete breakout strength of a single anchor in tension in cracked concrete; for cast-in headed anchors and 275 mm ≤ h_{ef} ≤ 625 mm, $N_{br} = 3.9 \phi_c \lambda_a \sqrt{f'_c} h_{ef}^{5/3} R$
- ϕ_c=0.65 – resistance factor for concrete
- k_c=10 for cast-in anchors
- *s* – spacing between anchors
- $c_{a,max}$ – maximum distance from an anchor to one of the three close edges
- λ_a = 1 – is modification factor for lightweight concrete
- f'_c – concrete compressive strength [MPa]
- *R* = 1 – resistance modification factor as specified in CSA A23.3 – D.5.3

According to A23.3-14 – D.6.2.8, in case of headed anchors, the projected surface area A_{Nc} is determined from the effective perimeter of the washer plate, which is the lesser value of d_a + 2 t_{wp} or d_{wp}, where:

- d_a – anchor diameter
- d_{wp} – washer plate diameter or edge size
- t_{wp} – washer plate thickness

The group of anchors is checked against the sum of tensile forces in anchors loaded in tension and creating a common concrete cone.

According to CSA A23.3-14 – D.6.2.9, where anchor reinforcement is developed in accordance with Clause 12 of A23.3-14 on both sides of the breakout surface, the anchor reinforcement is presumed to transfer the tension forces and concrete breakout strength is not evaluated (can be set in Code setup).

6.6.3 Concrete pullout resistance of anchor in tension

Concrete pullout strength of a headed anchor is defined in CSA A23.3-14 – D.6.3 as

$N_{cpr} = \Psi_{c,P}\, N_{pr}$

where:

- $\Psi_{c,P}$ – modification factor for concrete condition, $\Psi_{c,P}$ = 1.0 for cracked concrete
- $N_{pr} = 8\, A_{brg}\, \phi_c\, f'_c\, R$ for headed anchor
- A_{brg} – bearing area of the head of stud or anchor bolt
- ϕ_c = 0.65 – resistance factor for concrete
- d_a – anchor diameter
- f'_c – concrete compressive strength
- R = 1 – resistance modification factor as specified in CSA A23.3 – D.5.3

Concrete pullout strength for other types of anchors than headed is not evaluated in the software and has to be specified by the manufacturer.

6.6.4 Concrete side-face blowout resistance

Concrete side-face blowout strength of headed anchor in tension is defined in CSA A23.3-14 – D.6.4 as:

$$N_{sbr} = 13.3 c_{a1} \sqrt{A_{brg}} \phi_c \lambda_a \sqrt{f'_c} R$$

If c_{a2} for the single anchor loaded in tension is less than 3 c_{a1}, the value of N_{sbr} is multiplied by the factor $0.5 \le (1 + c_{a2} / c_{a1}) / 4 \le 1$.

A group of headed anchors with deep embedment close to an edge ($h_{ef} > 2.5\, c_{a1}$) and spacing between anchors less than 6 c_{a1} has the strength:

$$N_{sbgr} = \left(1 + \frac{s}{6 c_{a1}}\right) N_{sbr}$$

Only one reduction factor at a time is applied.

where:

- c_{a1} – the shorter distance from an anchor to an edge
- c_{a2} – the longer distance, perpendicular to c_{a1}, from an anchor to an edge

- A_{brg} – a bearing area of the head of stud or anchor bolt
- ϕ_c – resistance factor for concrete editable in Code setup
- f'_c – concrete compressive strength
- h_{ef} – depth of embedment; according to A23.3-14 – D.6.2.3, the effective embedment depth h_{ef} is reduced to $h_{ef} = \max\left(\frac{c_{a,max}}{1.5}, \frac{s}{3}\right)$ if anchors are located less than 1.5 h_{ef} from three or more edges
- s – spacing between anchors
- R = 1 – resistance modification factor as specified in CSA A23.3 – D.5.3

6.6.5 Steel resistance of anchor in shear

The steel strength in shear is determined according to A23.3 – D.7.1 as

$V_{sar} = A_{se,V}\ \phi_s\ 0.6\ f_{uta}\ R$

where:

- ϕ_s = 0.85 – steel embedment material resistance factor for reinforcement
- $A_{se,V}$ – effective cross-sectional area of an anchor in shear
- f_{uta} – specified tensile strength of anchor steel but not greater than the smaller of 1.9 f_{ya} or 860 MPa
- R = 0.75 – resistance modification factor as specified in CSA A23.3 – D.5.3

If mortar joint is selected, steel strength in shear V_{sa} is multiplied by 0.8 (A23.3 –D.7.1.3).

The shear on lever arm, which is present in case of base plate with oversized holes and washers or plates added to the top of the base plate to transmit the shear force, is not considered.

6.6.6 Concrete breakout resistance of anchor in shear

Concrete breakout strength of an anchor in shear is designed according to A23.3 –D.7.2. The shear force acting on a base plate is assumed to be transferred by the anchors which are closest to the edge in the direction of the shear force. The direction of the shear force with respect to the concrete edge affects the concrete breakout strength according to FIB Bulletin 58 – Design of anchorages in concrete – Guide to good practice (2011). If concrete cones of anchors overlap, they create a common concrete cone. The eccentricity in shear is also taken into account.

$$V_{cbr} = \frac{A_{Vc}}{A_{Vco}} \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{h,V} \psi_{\alpha,V} V_{br}$$

where:

- A_{Vc} – projected concrete failure area of an anchor or group of anchors divided by number of anchors in this group
- $A_{Vco} = 4.5\ c_{a1}^2$ – projected concrete failure area of one anchor when not limited by corner influences, spacing or member thickness
- $\psi_{ec,V} = \frac{1}{1+\frac{2e'_V}{3c_{a1}}}$ – modification factor for group of anchors loaded eccentrically in shear
- $\psi_{ed,V} = 0.7 + 0.3\frac{c_{a2}}{1.5c_{a1}} \le 1.0$ – modification factor for edge effect
- $\Psi_{c,V}$ – modification factor for concrete condition; $\Psi_{c,V} = 1.0$ for cracked concrete
- $\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \ge 1$ – modification factor for anchors located in a concrete member where $h_a < 1.5\ c_{a1}$
- $\psi_{\alpha,V} = \sqrt{\frac{1}{(\cos\alpha_V)^2+(0.5\sin\alpha_V)^2}}$ – modification factor for anchors loaded at an angle with the concrete edge (FIB Bulletin 58 – Design of anchorages in concrete – Guide to good practice, 2011)
- h_a – height of a failure surface on the concrete side
- $V_{br} = \min\left(0.58\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\phi_c\lambda_a\sqrt{f'_c}c_{a1}^{1.5}R,\ 3.75\lambda_a\phi_c\sqrt{f'_c}c_{a1}^{1.5}R\right)$
- $l_e = h_{ef} \le 8\ d_a$ – load-bearing length of the anchor in shear
- d_a – anchor diameter
- f'_c – concrete compressive strength
- c_{a1} – edge distance in the direction of load
- c_{a2} – edge distance in the direction perpendicular to load

- ϕ_c = 0.65 – resistance factor for concrete
- R = 1 – resistance modification factor as specified in CSA A23.3 – D.5.3

If both edge distances $c_{a2} \leq 1.5 c_{a1}$ and $h_a \leq 1.5\ c_{a1}$, $c_{a1} = \max\left(\frac{c_{a2}}{1.5}, \frac{h_a}{1.5}, \frac{s}{3}\right)$, where s is the maximum spacing perpendicular to direction of shear, between anchors within a group.

According to A23.3-14 – D.7.2.9, where anchor reinforcement is developed in accordance with A23.3-14 – Clause 12 on both sides of the breakout surface, the anchor reinforcement is presumed to transfer the shear forces and concrete breakout strength is not evaluated.

6.6.7 Concrete pryout resistance of an anchor in shear

Concrete pryout strength is designed according to A23.3 – D.7.3.

$V_{cpr} = k_{cp}\ N_{cpr}$

where:

- k_{cp} = 1.0 for h_{ef} < 65 mm, k_{cp} = 2.0 for $h_{ef} \geq$ 65 mm
- N_{cpr} – concrete breakout strength – all anchors are considered to be in tension

According to CSA A23.3-14 – D.6.2.9, where anchor reinforcement is developed in accordance with Clause 12 of A23.3-14 on both sides of the breakout surface, the anchor reinforcement is presumed to transfer the tension forces and concrete breakout strength is not evaluated (can be set in Code setup).

6.6.8 Interaction of tensile and shear forces

Interaction of tensile and shear forces is assessed according to A23.3 – Figure D.18.

$$\left(\frac{N_f}{N_r}\right)^{5/3} + \left(\frac{V_f}{V_r}\right)^{5/3} \leq 1.0$$

where:

- N_f and V_f – design forces acting on an anchor
- N_r and V_r – the lowest design strengths determined from all appropriate failure modes

6.6.9 Anchors with stand-off

Anchor with stand-off is designed as a bar element loaded by shear force, bending moment and compressive or tensile force. These internal forces are determined by finite element model. The anchor is fixed on both sides, one side is 0.5×d below the concrete level, the other side is in the middle of the thickness of the plate. The buckling length is conservatively assumed as twice the length of the bar element. Plastic section modulus is used. The bar element is designed according to S16-14. Interaction of shear force is neglected because the minimum length of the anchor to fit the nut under the base plate ensures that the anchor fails in bending before the shear force reaches half the shear resistance and the shear interaction is negligible (up to 7 %). Interaction of bending moment and compressive or tensile force is conservatively assumed as linear. Second order effects are not taken into account.

Shear resistance (CSA S16-14 – 13.4.4):

$V_r = \phi \cdot 0.66 \cdot A_v \cdot F_y$

- $A_v = 0.844 \cdot A_s$ – the shear area
- A_s – the bolt area reduced by threads
- F_y – bolt yield strength
- ϕ – the resistance factor, recommended value is 0.9

Tensile resistance (CSA S16-14 – 13.2)

$T_r = \phi \cdot A_s \cdot F_y$

Compressive resistance (CSA S16-14 – 13.3.1)

$$C_r = \frac{\phi A_s F_y}{\left(1+\lambda^{2n}\right)^{\frac{1}{n}}}$$

- $\lambda = \sqrt{\frac{F_y}{F_e}}$ – anchor bolt slenderness
- $F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$ – elastic buckling stress
- $KL = 2 \cdot l$ – buckling length
- l – length of the bolt element equal to half the base plate thickness + gap + half the bolt diameter
- $r = \sqrt{\frac{I}{A_s}}$ – radius of gyration of the anchor bolt

- $I = \frac{\pi d_s^4}{64}$ – moment of inertia of the bolt
- n = 1.34 – parameter for compressive resistance

Bending resistance (CSA S16-14 – 13.5):

$M_r = \phi \cdot Z \cdot F_y$

$Z = d_s^3 / 6$ – plastic section modulus of the bolt

Linear interaction:

$\frac{N}{C_r} + \frac{M}{M_r} \leq 1$ ... for compressive normal force

$\frac{N}{T_r} + \frac{M}{M_r} \leq 1$ ... for tensile normal force

- N – tensile (positive) or compressive (negative sign) factored force
- C_r – factored compressive (negative sign) resistance
- T_r – factored tensile (positive sign) resistance
- M – factored bending moment
- M_r – factored moment resistance

6.7 Member capacity design

Member capacity design is performed according to S16-14 – Cl. 27:

$R_d \cdot R_o = 1.3$

where:

- R_d – the ductility-related force modification factor that reflects the capability of a structure to dissipate energy through inelastic behavior
- R_o – the overstrength-related force modification factor that accounts for the dependable portion of reserve strength in a structure

6.8 Detailing

In the detailing of **bolted connections**, the minimum pitch and minimum edge distance are checked according to S16-14 – 22.3. Minimum pitch (2.7 d – editable in Code setup) and minimum edge distance (1.25 d) are checked.

The spacing between **anchors** should be greater than 4 times anchor diameter according to A23.3-14 – D.9.2.

References

A23.3-14, *Design of concrete structures*, CSA Group, 2014, revised in December 2015, 291 p.

ACI 318-14, *Building Code Requirements for Structural Concrete* and ACI 318R-14, *Commentary on Building Code Requirements for Structural Concrete*, ACI, 2014, 524 p.

ANSI/AISC 341-10, *Seismic Provisions for Structural Steel Buildings*, AISC, Chicago, 2010, 402 p.

ANSI/AISC 360-16, *An American National Standard – Specification for Structural Steel Buildings*, AISC, Chicago, 2016, 676 p.

Beg D., Zupančič E., Vayas I., On the rotation capacity of moment connections, *Journal of Constructional Steel Research*, 60 (3–5), 2004, 601–620.

Dvorkin E. N., Bathe K. J., Continue A., Mechanics Based Four Node Shell Element for General Nonlinear Analysis, *Engineering Computations*, 1, 1984.

EN 1992-1-1, Eurocode 2, *Design of concrete structures, Part 1-1, General rules and rules for buildings*, CEN, Brussels, 2005.

EN 1993-1-5, Eurocode 3, *Design of steel structures, Part 1-5, Plated Structural Elements*, CEN, Brussels, 2005.

EN 1993-1-6, Eurocode 3, *Design of steel structures, Part 1-6, Strength and Stability of Shell Structures*, CEN, Brussels, 2007.

EN 1993-1-8, Eurocode 3, *Design of steel structures, Part 1-8, Design of joints*, CEN, Brussels, 2006.

EN 1998-1, Eurocode 8, *Design of structures for earthquake resistance – Part 1: General rules, Seismic actions and rules for buildings*, CEN, Brussels, 2004.

ETAG 001, *Guideline for European Technical Approval of Metal Anchors for Use in Concrete – Annex C: Design Methods for Anchorages*, Brussels, EOTA, 2010.

Faella C, Piluso V, Rizzano G., Plastic deformation capacity of bolted T-stubs: theoretical analysis and testing, in *Moment resistant connections of steel frames in seismic area, design and reliability*, E&FN Spoon. 2000.

FIB bulletin 58 – *Design of anchorages in concrete – Guide to good practice*. Lausanne: Fédération Internationale du Béton fib / International Federation for Structural Concrete, 2011. ISBN 978-2-88394-098-7

Foley C.M., Vinnakota S., Toward design office moment–rotation curves for end-plate beam-to-column connections, *Journal of Constructional Steel Research*, 35, 1995, 217–53.

Gioncu V., Mazzolani F., *Ductility of seismic resistant steel structures*, Spon Press; 2002.

Girao A.M.C., Bijlaard F.S.K., da Silva L.S., Experimental assessment of the ductility of extended end plate connections, *Journal of Engineering Structures*, 26, 2004, 1185–1206.

Gödrich L., Wald F., Sokol Z., Advanced modelling of end plate, in *Eurosteel 2014*, Brussels, ECCS, 2014, 287-288.

Grecea D., Stratan A., Ciutina A., Dubina D., Rotation capacity of the beam-to-column joints under cyclic loading, in *Connections in Steel Structures V*, Amsterdam, 2004.

Ibrahimbegovic A., Taylor R.L., Wilson E. L., A robust quadrilateral membrane element with drilling degrees of freedom, *International Journal for Numerical Methods in Engineering*, 30/3, 1990, 445-457.

ISO 898-1, *Mechanical properties of fasteners made of carbon steel and alloy steel, Part 1, Bolts, screws and studs with specified, property classes, Coarse thread and fine pitch thread*, Geneva, 2009.

Kuhlmann U., Kuhnemund F., *Rotation capacity of steel joints, Verification Procedure and Component Tests*, 2000.

S16-14, *Design of steel structures*, CSA Group, 2014, revised in October 2015, 216 p.

Sherbourne A.N, Bahaari M.R., 3D simulation of end-plate bolted connections, *Journal of Structural Engineering*, 120, 1994, 3122–3136.

Sherbourne A.N., Bahaari M.R., Simulation of bolted connections to unstiffened columns T-stub connections, *Journal of Constructional Steel Research*, 40, 1996, 169–87.

VDI2230 *Systematic calculation of high duty bolted joints – Joints with one cylindrical bolt*, Association of German Engineers, Berlin, 2003.

Virdi K. S. et al, *Numerical Simulation of Semi Rigid Connections by the Finite Element Method*, Report of Working Group 6 Numerical, Simulation COST C1, Brussels Luxembourg, 1999.

Wald F., Sokol Z., Moal M., Mazura V., Muzeau J. P., Stiffness of cover plate connections with slotted holes, in *Eurosteel 2002*, Coimbra, 2002, 1007-1016