Proyecto: **CUBIERTA IGAC**
Proyecto nº:
Autor:

Datos del proyecto

Nombre del proyecto	CUBIERTA IGAC
Número del Proyecto	
Autor	
Descripción	
Fecha	14/04/2025
Código	AISC/ACI

Material

Acero	A572 Gr.50

Ítem del proyecto CON12

Diseño

Nombre	CON12
Descripción	
Análisis	Tensión, deformación/ Cargas en equilibrio
Normativa de cálculo	AISC - LRFD (2022)

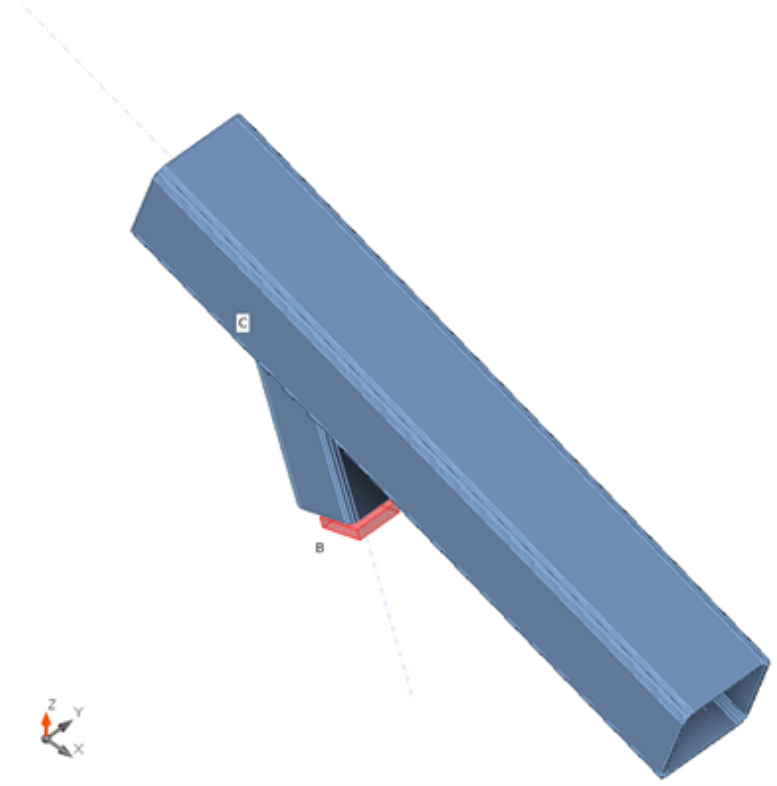
Elementos

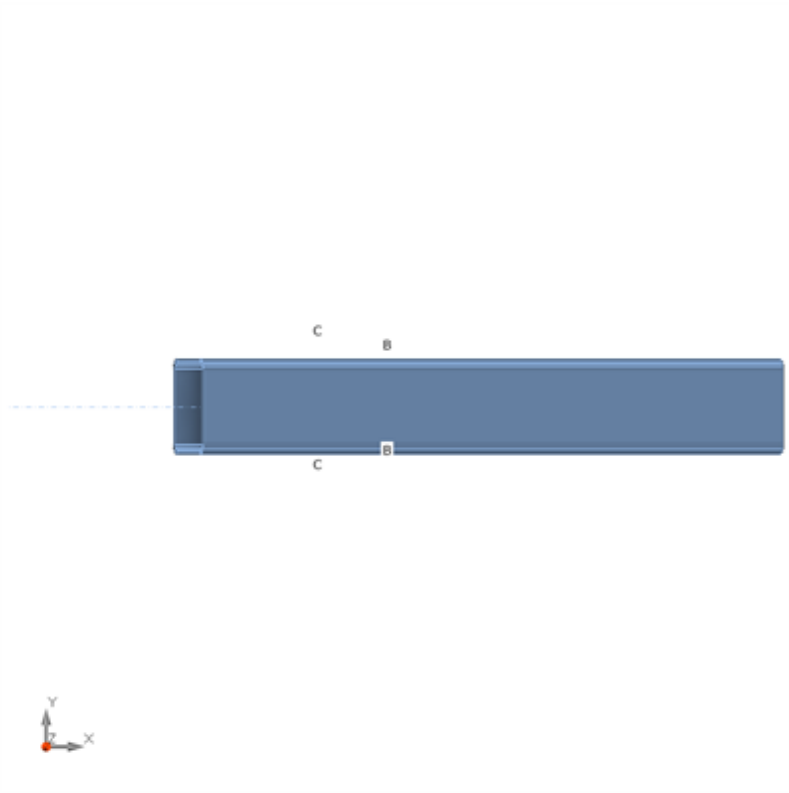
Geometría

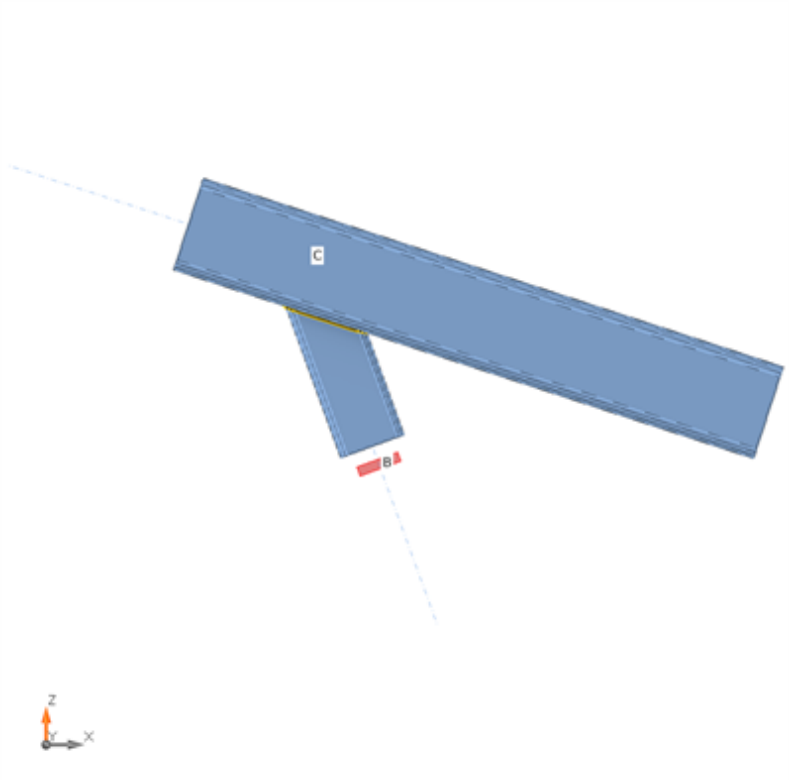
Nombre	Sección transversal	β – Dirección [°]	γ - Inclinación [°]	α - Rotación [°]	Desplazamiento ex [mm]	Desplazamiento ey [mm]	Desplazamiento ez [mm]
C	3 - RHS100x100	0.0	-18.0	0.0	0	0	0
B	6 - RHS70x70	0.0	-70.0	0.0	0	0	-150

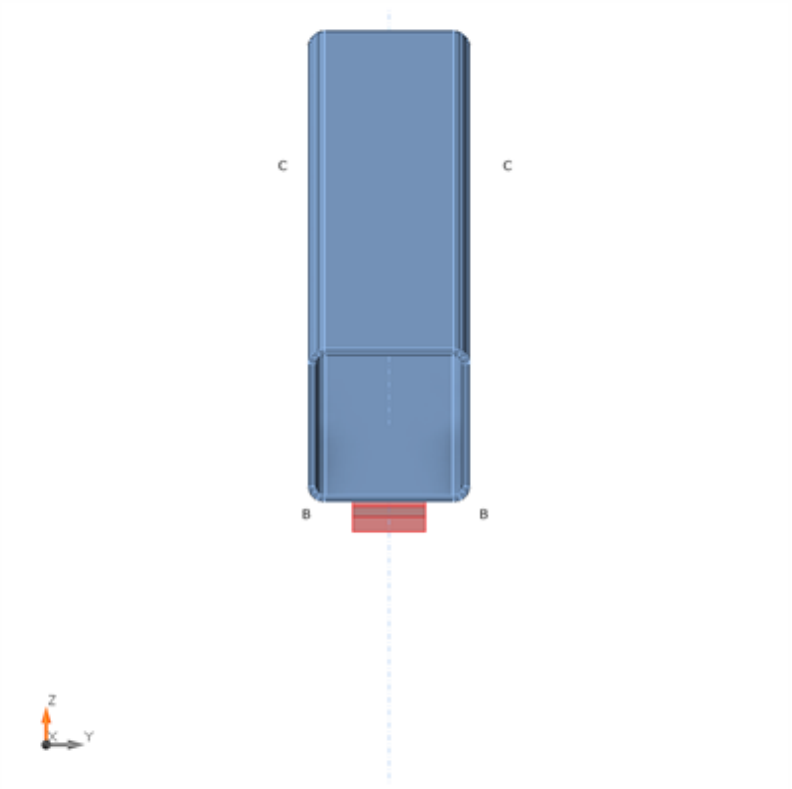
Apoyos y fuerzas

Nombre	Apoyo	Fuerzas en	X [mm]
C / inicial		Nodo	0
C / final		Nodo	0
B / final	N-Vy-Vz-Mx-My-Mz	Nodo	0




C
B
B
C
Y
Z
X

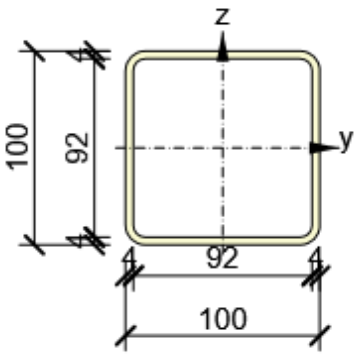
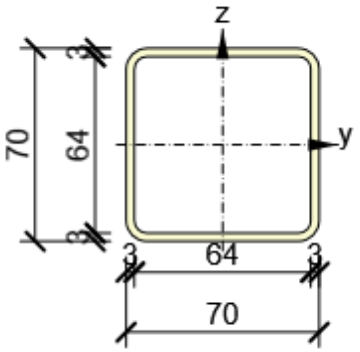

C
B
A
Z
Y
X



Secciones

Nombre	Material
3 - RHS100x100	A572 Gr.50
6 - RHS70x70	A572 Gr.50

Secciones

Nombre	Material	Dibujo
3 - RHS100x100	A572 Gr.50	
6 - RHS70x70	A572 Gr.50	

Cargas (Fuerzas en equilibrio)

Nombre	Elemento	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE1	C / Inicial	0.0	0.0	0.0	0.0	0.0	0.0
	C / Finalizar	0.0	0.0	0.0	0.0	0.0	0.0
	B / Finalizar	48.3	-0.1	-0.3	0.0	0.4	0.1

Fuerzas Desbalanceadas

Nombre	X [kN]	Y [kN]	Z [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE1	16.2	-0.1	-45.5	0.1	-6.8	0.0

Verificación

Resumen

Nombre	Valor	Estado de la verificación
Análisis	100.0%	OK
Placas	0.0 < 5.0%	OK
Deformación loc.	0.0 < 3%	OK
Soldaduras	0.0 < 100%	OK
Pandeo	No calculado	
GMNA	No calculado	

Placas

Nombre	t_p [mm]	Cargas	σ_{Ed} [MPa]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [MPa]	Estado
C	4.0	LE1	0.0	0.0	0.0	OK
B	3.0	LE1	0.0	0.0	0.0	OK

Datos de diseño

Material	F_y [MPa]	ε_{lim} [%]
A572 Gr.50	344.7	5.0

Explicación del símbolo

t_p	Espesor de placa
σ_{Ed}	Tensión equivalente
ε_{Pl}	Deformación plástica
$\sigma_{c,Ed}$	Tensiones de Contacto
F_y	Límite elástico
ε_{lim}	Límite de la deformación plástica

Resultado detallado para C

Design values used in the analysis

$\phi F_y =$ 310.3 MPa

Donde:

$F_y = 344.7$ MPa – characteristic yield strength

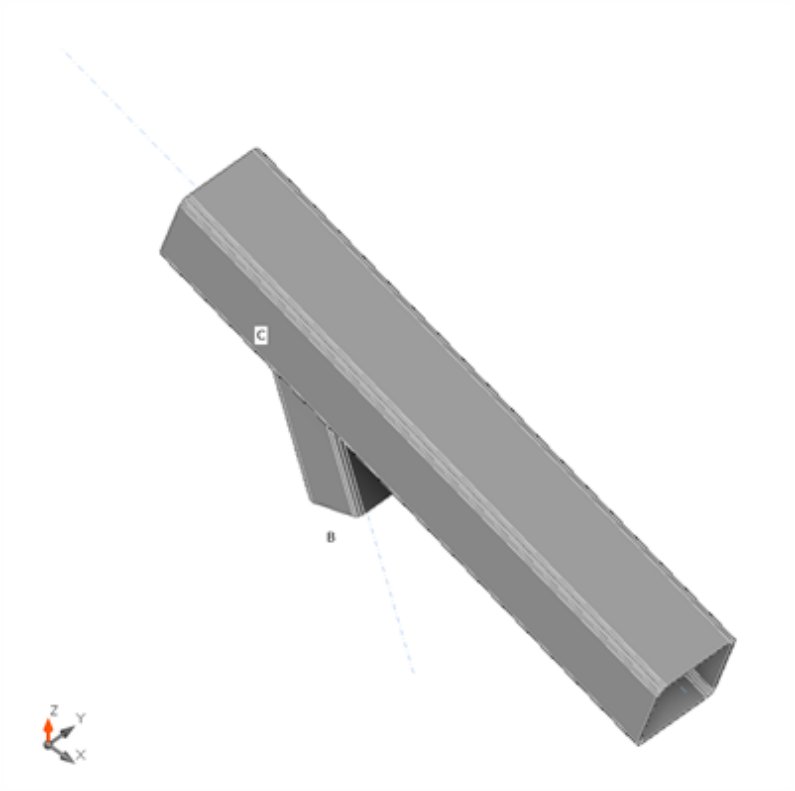
$\phi = 0.90$ – resistance factor for steel material AISC 360-22 – B3.1

Deformación loc.

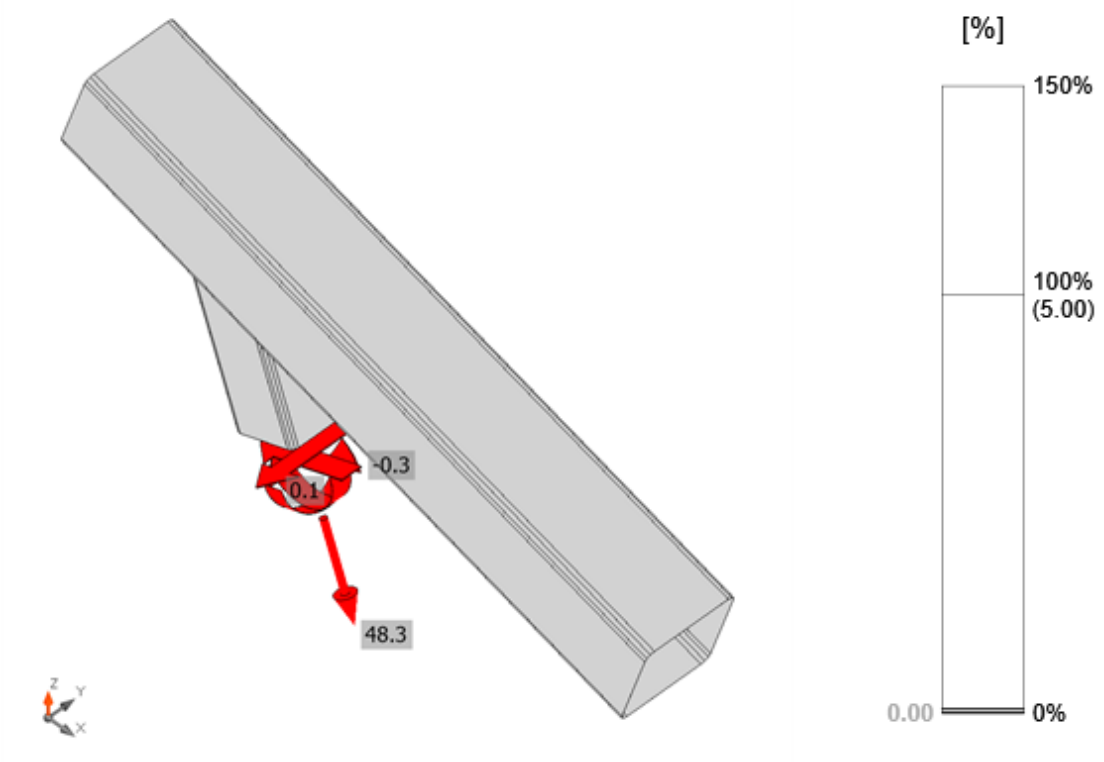
Nombre	d_0 [mm]	Cargas	δ [mm]	δ_{lim} [mm]	δ/d_0 [%]	Estado de la verificación
C	100	LE1	0	3	0.0	OK
B	70	LE1	0	2	0.0	OK

Explicación del símbolo

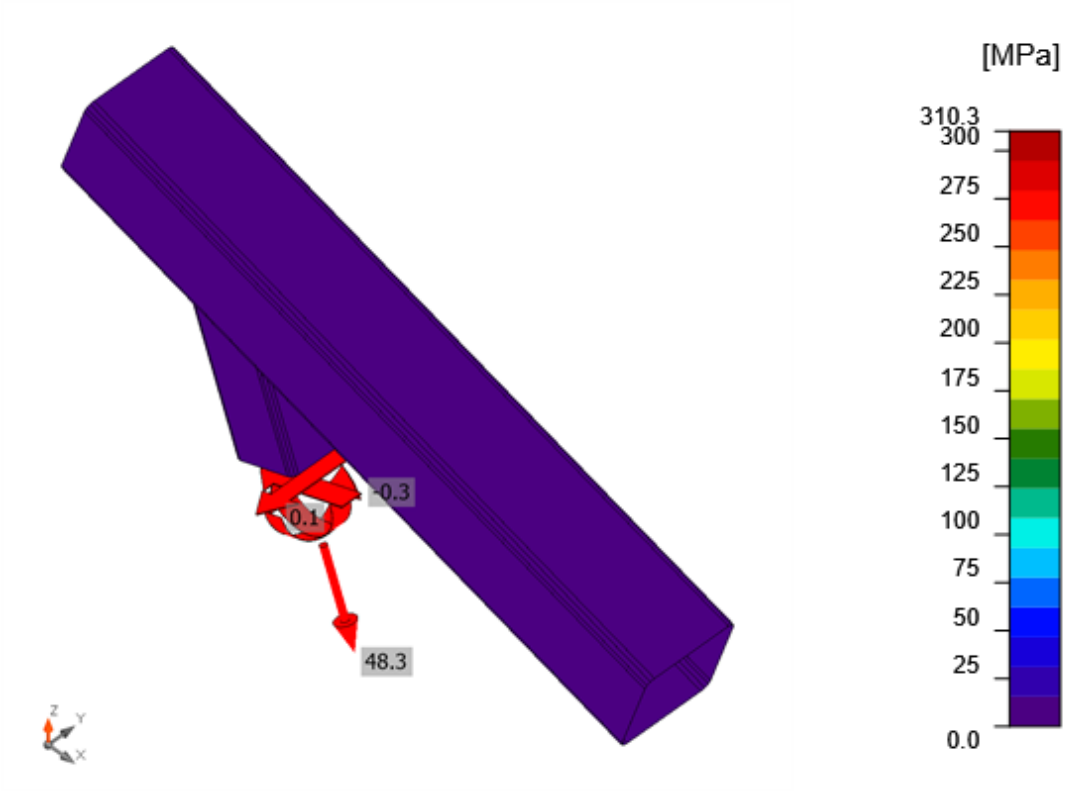
d_0	Tamaño de la sección
δ	Deformación local de la sección
δ_{lim}	Deformación permitida



Verificación general, LE1



Verificación de deformación, LE1



Tensión equivalente, LE1

Soldaduras

Ítem	Borde	Xu	t_w [mm]	w [mm]	L [mm]	L_c [mm]	Cargas	F_n [kN]	ϕR_n [kN]	Ut [%]	Detallado	Estado
C-w 1	B	E70xx	◢ 2.2	◢ 3.2	290	8	LE1	0.0	3.9	0.0	Aceptar	OK

Datos de diseño

Material	F_{exx} [MPa]
E70xx	482.6

Explicación del símbolo

t_w	Espesor de garganta de soldadura
w	Tamaño de la soldadura
L	Longitud de la soldadura
L_c	Longitud del elemento crítico de la soldadura
F_n	Fuerza en el elemento crítico de la soldadura
ϕR_n	Resistencia de la soldadura - AISC 360-22 – J2-4
Ut	Utilización
◢	Cordón de soldadura
F_{exx}	Ultimate strength as rated by electrode classification number

Resultado detallado para C-w 1 / B

Comprobación de la resistencia de la soldadura (AISC 360-22 – J2-4)

$$\phi R_n = \phi \cdot F_{nw} \cdot A_{we} = \quad 3.9 \quad \text{kN} \quad \geq \quad F_n = \quad 0.0 \quad \text{kN}$$

Donde:

$F_{nw} =$ 301.3 MPa – resistencia nominal del material de aportación:

- $F_{nw} = 0.6 \cdot F_{EXX} \cdot (1 + 0.5 \cdot sin^{1.5}\theta)$, donde:
  - $F_{EXX} =$ 482.6 MPa – número de clasificación del electrodo, p.e. mínima resistencia a tracción especificada
  - $\theta =$ 10.8° – ángulo de actuación de la carga medido desde el eje de la soldadura

$A_{we} =$ 17 mm^2 – área eficaz del elemento de soldadura crítico

$\phi =$ 0.75 – factor de resistencia para conexiones soldadas

Pandeo

El análisis de pandeo no se ha calculado.

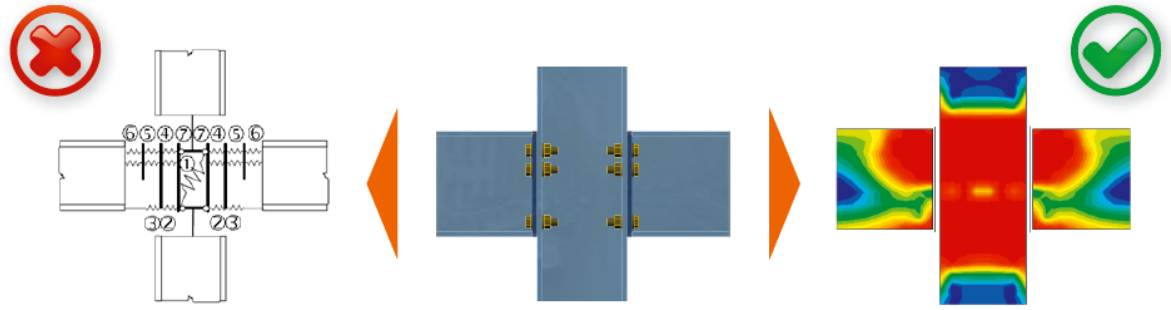
Configuración de la norma

Ítem	Valor	Unidad	Referencia
Coeficiente de fricción - hormigón	0.40	-	ACI 349-01 – B.6.1.4
Coeficiente de fricción en la resistencia a deslizamiento	0.30	-	AISC 360-22 – J3.9
Deformación plástica límite	0.05	-	
Detallado	Sí		
Distancia entre tornillos [d]	2.66	-	AISC 360-22 – J3.4
Distancia entre tornillos y el borde [d]	1.25	-	AISC 360-22 – J.3.5
Resistencia al arrancamiento del cono de hormigón	Ambos		
Resistencia del metal base en la cara de fusión de la soldadura	No		AISC 360-22 – J2-2
La deformación en el orificio del perno con la carga de servicio es una consideración de diseño	Sí		AISC 360-22 – J3.11
Hormigón fisurado	Sí		ACI 318-14 – 17
Comprobación de la deformación local	Sí		
Deformación límite local	0.03	-	CIDECT DG 1, 3 - 1.1
No linealidad geométrica (GMNA)	Sí		Grandes deformaciones para secciones huecas

Theoretical Background

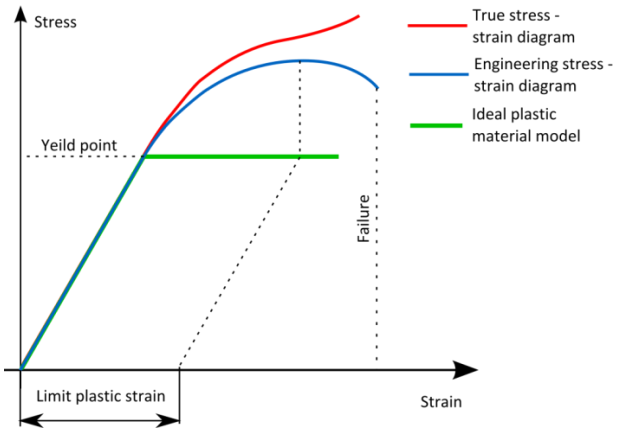
CBFEM versus AISC 360

The weak point of standard design method is in analyzing of internal forces and stress in a joint. CBFEM replaces specific analysis of internal forces in joint with general FEA.



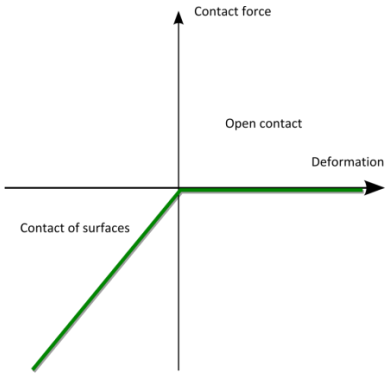
Component model *Bolted joint* *CBFEM model*

Check methods of specific components like bolts or welds are done according to standard AISC 360.
For the fasteners – bolts and welds – special FEM components had to be developed to model the welds and bolts behaviour in the connection. All parts of 1D members and all additional plates are modeled as plate/walls. These elements are made of steel (metal in general) and the behaviour of this material is significantly nonlinear.
The real stress-strain diagram of steel is replaced by the ideal plastic material for design purposes in building practice. The advantage of ideal plastic material is, that only yield strength and modulus of elasticity must be known to describe the material curve. The yield strength is multiplied by resistance factor (LRFD) or divided by safety factor (ASD) – AISC 360, Appendix 1. The granted ductility of construction steel is 15 %. The real usable value of limit plastic strain is 5% for ordinary design (EN 1993-1-5 appendix C paragraph C.8 note 1).
The stress in steel cannot exceed the yield strength when using the ideal elastic-plastic stress-strain diagram.



Real tension curve and the ideal elastic-plastic diagram of material

CBFEM method aims to model the real state precisely. Meshes of plates / walls are not merged, no intersections are generated between them, unlike it is used to when modeling structures and buildings. Mesh of finite elements is generated on each individual plate independently on mesh of other plates.
Between the meshes, special massless force interpolation constraints are added. They ensure the connection between the edge of one plate and the surface or edge of the other plate.
This unique calculation model provides very good results – both for the point of view of precision and of the analysis speed. The method is protected by patent.
The steel base plate is placed loosely on the concrete foundation. It is a contact element in the analysis model – the connection resists compression fully, but does not resist tension.



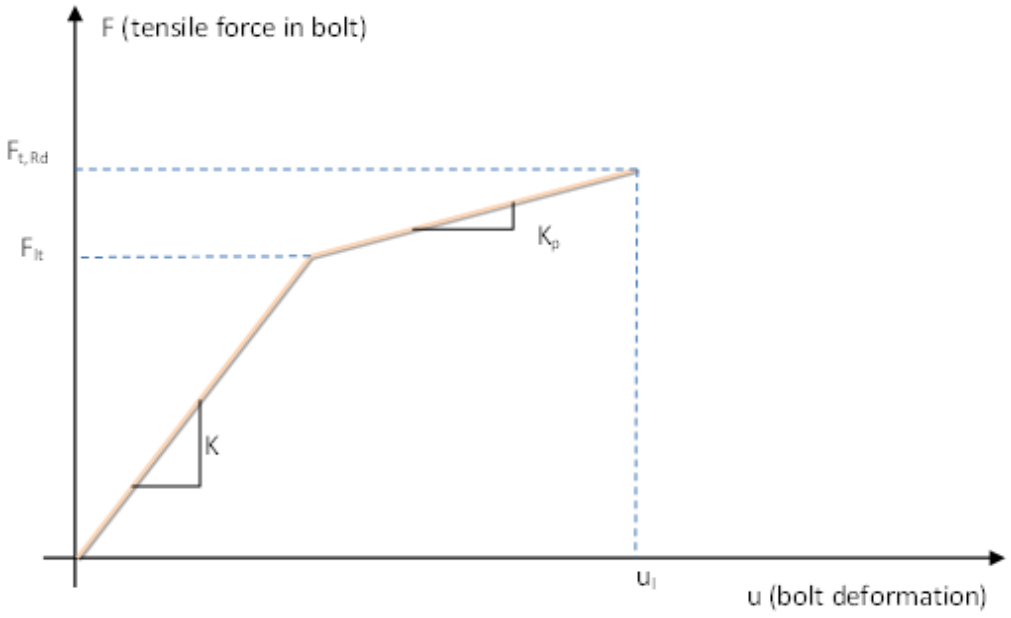
Stress-strain diagram of contact between the concrete block and the base plate

The concrete block in CBFEM is modeled using Winkler-Pasternak subsoil model. The stiffness of subsoil is determined using modulus of elasticity of concrete and effective height of subsoil. The concrete block is not designed by CBFEM method.

Welds are modeled using a special elastoplastic element, which is added to the interpolation links between the plates. The element respects the weld throat thickness, position and orientation. The plasticity state is controlled by stresses in the weld throat section. The plastic redistribution of stress in welds allows for stress peaks to be redistributed along the longer part of the weld.

Bolted connection consists of two or more clasped plates and one or more bolts. Plates are placed loosely on each other.
A contact element is inserted between plates in the analysis model, which acts only in compression. No forces are carried in tension.

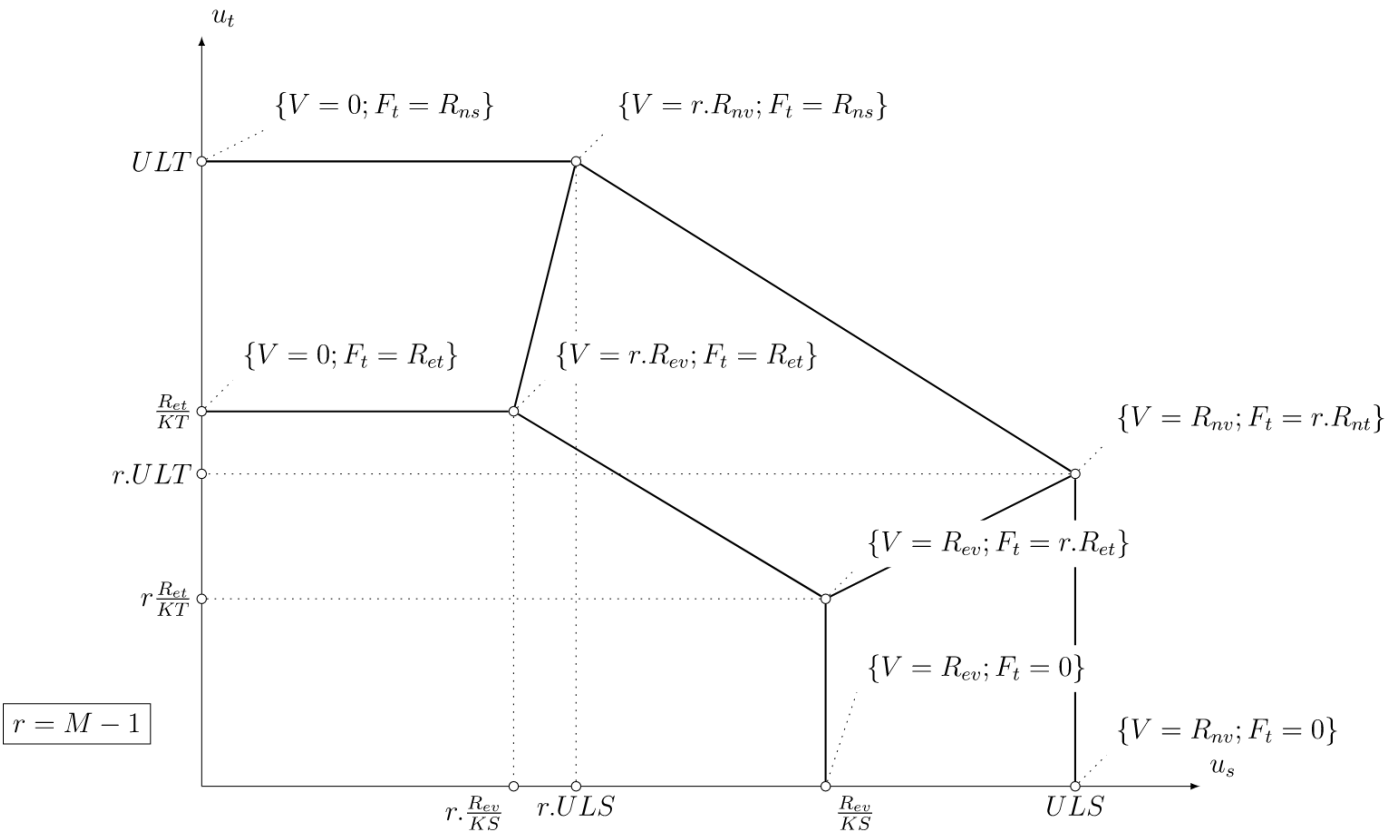
Shear force is taken by bearing. Special model for its transferring in the force direction only is implemented. IDEA StatiCa Connection can check bolts for interaction of shear and tension. The bolt behavior is implemented according to the following picture.



Bolt – tension

Symbols explanation:

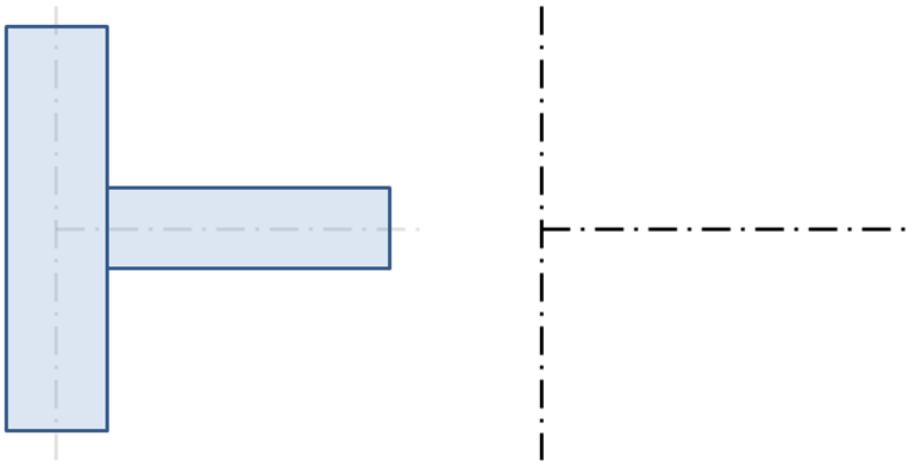
- K – linear stiffness of bolt,
- K_p – stiffness of bolt at plastic branch,
- F_{lt} – limit force for linear behaviour of bolt,
- $F_{t,Rd}$ – limit bolt resistance,
- u_l – limit deformation of bolt.



Bolt – interaction of shear and tension

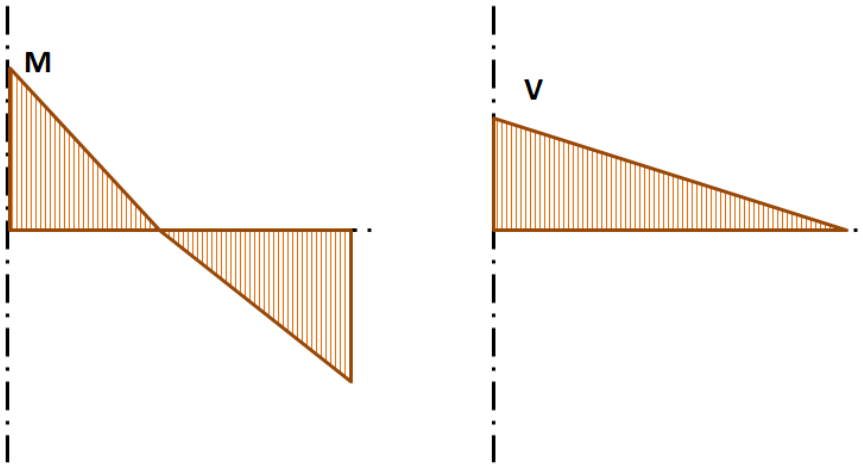
Loads

End forces of member of the frame analysis model are transferred to the ends of member segments. Eccentricities of members caused by the joint design are respected during load transfer.
The analysis model created by CBFEM method corresponds to the real joint very precisely, whereas the analysis of internal forces is performed on very idealised 3D FEM bar model, where individual beams are modeled using centrelines and the joints are modeled using immaterial nodes.



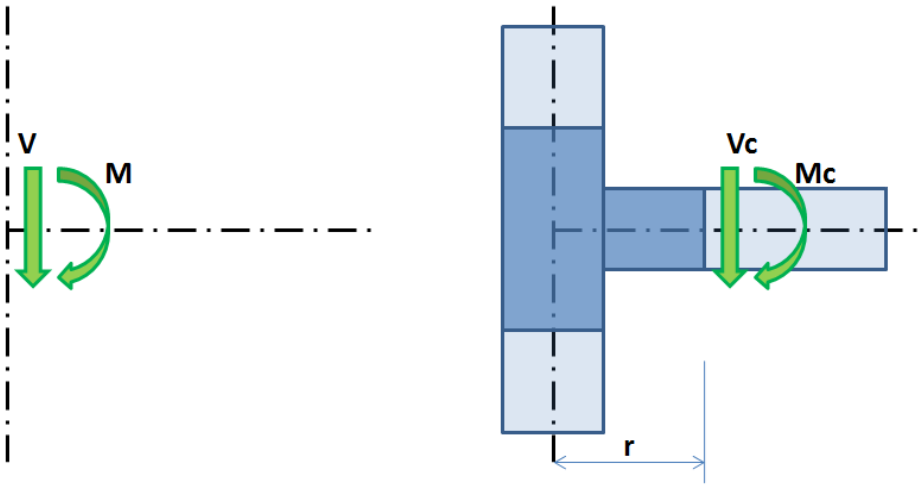
Joint of a vertical column and a horizontal beam

Internal forces are analysed using 1D members in 3D model. There is an example of courses of internal forces in the following picture.



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

The effects caused by member on the joint are important to design the connection. The effects are illustrated in the following picture.



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

Moment M and shear force V act in a theoretical joint. The point of theoretical joint does not exist in 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 CBFEM model, the end section of segment is loaded by moment M_c and force V_c.

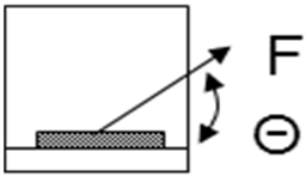
Welds

Fillet welds

The design strength, ϕR_n and the allowable strength, R_n/Ω of welded joints are evaluated in connection weld check.
$\phi = 0.75$ (LRFD)
$\Omega = 2.00$ (ASD)
Available strength of welded joints is evaluated according to AISC 360 – J2.4:
$R_n = F_{nw} A_{we}$
$F_{nw} = 0.60\, F_{EXX}\, (1.0 + 0.50 \sin^{1.5}\Theta)$
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.



For long welds and welding to unstiffened flanges or webs of rectangular hollow sections, the weld material model is fine-tuned so that no reduction factor is necessary. The weld resistance is governed by most stressed weld element.

CJP groove welds

AISC Specification Table J2.5 identifies four loading conditions that might be associated with JP 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 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.

Bolts

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)
$\Omega = 2.00$ (ASD)
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 tensile force, against which the required tensile strength is checked, includes any tension resulting from prying action produced by deformation of the connected parts.

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 J3-2)
$\phi = 0.75$ (LRFD)
$\Omega = 2.00$ (ASD)
$F'_{nt} = 1.3 F_{nt} - f_{rv} F_{nt} / \phi F_{nv}$ (AISC 360 J3-3a LRFD)
$F'_{nt} = 1.3 F_{nt} - f_{rv} \Omega F_{nt} / F_{nv}$ (AISC 360 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 – Tab. J3.2,
- F_{nv} – nominal shear stress from AISC 360 – Tab. 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}.

Bearing strength in bolt holes

The available bearing strength, ϕR_n and R_n/Ω at bolt holes is determined for the limit state of bearing 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 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 J3-6e, f)
$\phi = 0.75$ (LRFD)
$\Omega = 2.00$ (ASD)

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 connected material.

Preloaded bolts

The design slip resistance of a preloaded class A325 or A490 bolt with of effect of tensile force, $F_{t,Ed}$ according to AISC 360 – J3.9.
Preloading force to be used AISC 360 – Tab. J3.1.
$T_b = 0.7\, f_{ub} A_s$
Design slip resistance per bolt AISC 360 – J3.8
$R_n = 1.13\, \mu\, T_b N_s$
Utilisation in shear [%]:
$U_{ts} = V / R_n$
where

- A_s – tensile stress area of the bolt,
- f_{ub} – ultimate tensile strength,
- μ – mean slip factor coefficient,
- N_s – number of the friction surfaces. Check is calculated for each friction surface separately,
- V – shear force.

Anchors

The anchor bolt element is elastic-plastic with significant strain hardening. The maximum steel tensile resistance is expected at the strain which equals to 0.25 × guaranteed elongation. The failure mode due to concrete cracking may occur before the anchor steel tensile resistance is reached and is considered as a completely brittle failure.
Similarly, the steel components in shear (anchor bolt, base plate in bearing) are able to yield but failure modes connected with concrete cracking may occur suddenly as a brittle failure.
All standards use Concrete Capacity Design method developed by prof. R. Eligehausen at University of Stuttgart. The theory is based on vast experimental and numerical testing mostly on unreinforced concrete blocks and relatively short, often post-installed, anchors.
Anchorage is designed according to ACI 318-14 – Chapter 17. The design is available only for LRFD. Some failure modes (e.g. steel resistance) are evaluated for single anchors, others (e.g. concrete breakout) are checked for group of anchors.