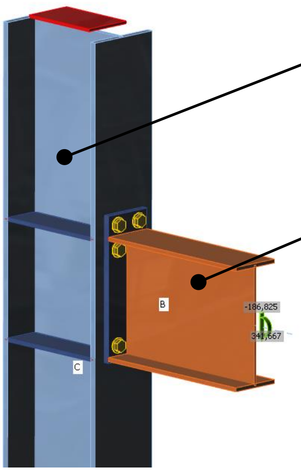
Verification example Extended Moment End-Plate

Type of connection:	Extended Moment End-Plate connection
Unit system:	Imperial (converted to metric)
Designed acc. to:	AISC 360-10 - Design Guide 16 – LRFD
Investigated:	Bolts, End-plate
Materials:	Steel A36, Bolts A325

Geometry:

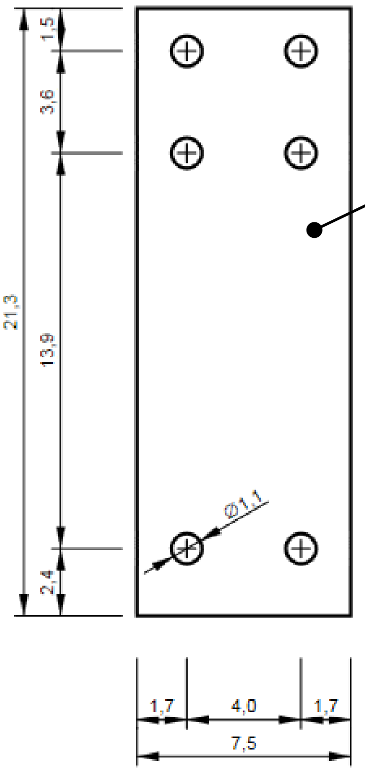


Structural shape:

W 18x97

Structural shape:

W 18x50



End-Plate thickness:

t_p = 7/8 in.

Bolts:

M 1 in. – A325

Applied forces:

M = 252 kip-ft

V = 0 kips

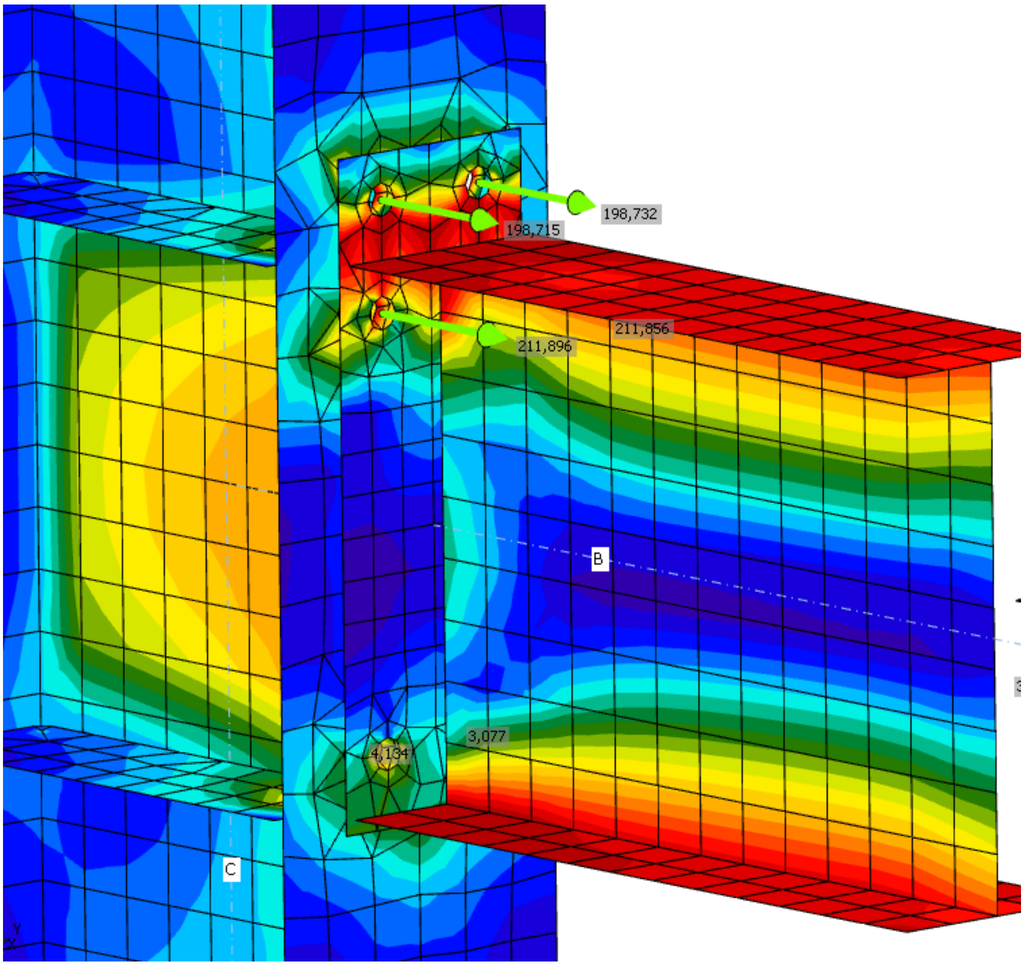
N = 0 kips

Procedure:

For the purpose of verification, it is considered that the design is determined by bolt rupture with prying actions. Therefore the design procedure with prying actions (acc. DG16) is used.

Example is based on an example in AISC Design Examples v14.1 - EXAMPLE II.B-4 FOUR-BOLT UNSTIFFENED EXTENDED END-PLATE FR MOMENT -CONNECTION (BEAM-TO-COLUMN FLANGE)

IDEA StatiCa Connection – results

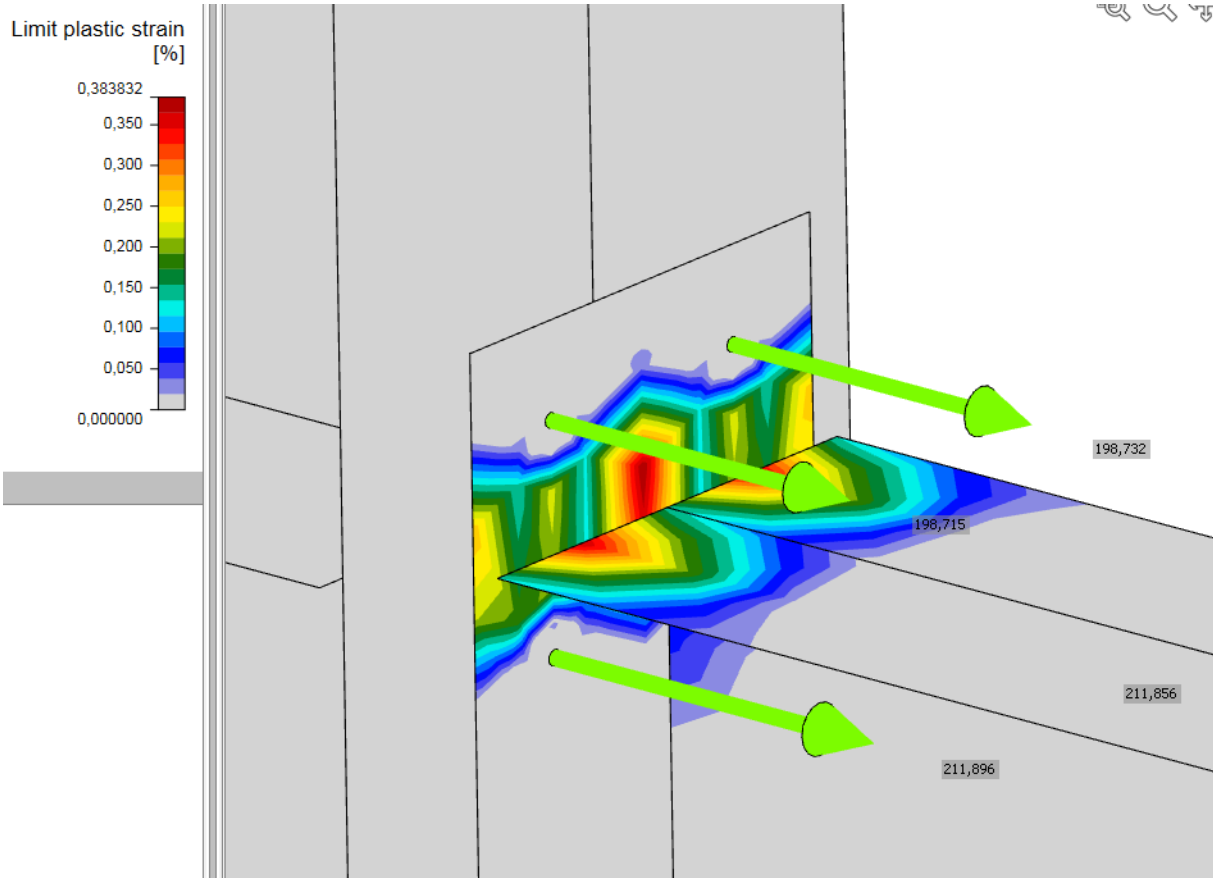


The extreme tension force in upper bolt is **Ft = 211.9 kN = 47.6 kips**

The ultimate tensile strength of 1 in. – A325 is **ϕFnt = 53 kips** *acc. Table J3-2 and Eq. J3-1*

The unit check: 47.6/53 = **0.90 = 90%**

Picture below shows that certain areas of connection overcome the yield strength. Therefore, the plastic strain limit must be taken into consideration:



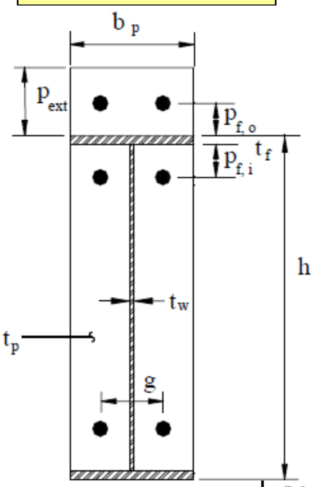
The maximal plastic strain is around 0.38%, which is significantly under the limit of 5%.

It's clear that the designed connection is close to its limit by both limit states of bolt rupture and by limit state of plate thickness. However the connection design is satisfactory, the unit check is then around 0.90.

AISC 360-10 and Design Guide 16 – results

Extended connection (AISC Steel design guide 16) - with prying actions

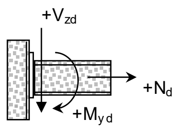
W18x50



h_{PROF} =	457,2	mm	= 18,00 in.
t_w =	9,017	mm	= 0,36 in.
t_f =	14,478	mm	= 0,57 in.
t_p =	22,225	mm	= 0,88 in.
bp =	190,373	mm	= 7,50 in.
g =	101,6	mm	= 4,00 in.
$p_{f,i}$=	38,1	mm	= 1,50 in.
$p_{f,o}$=	38,1	mm	= 1,50 in.
p_{ext}=	76,2	mm	= 3,00 in.
t_{ww} =	9,017	mm	[web + welds]
t_{FW} =	14,478	mm	[flange + welds]
h1 =	404,6	mm	
h0 =	495,3	mm	
b_f=	190,4	mm	
s=	69,5	mm	
Yp=	3612	mm	
d1=	397,383	mm	
d0=	488,061	mm	

Plate and beam steel: **A36**

E=	200	GPa	= 29 Msi
f_y =	250	MPa	= 36 ksi
f_U =	400	MPa	= 58 ksi
Φ_b =	0,9		
$\Phi_b f_y$ =	225,0	MPa	= 33 ksi
Φ =	0,75		
γ_r=	1,00		



Moment Myd =	*341,7*	*kNm*	= 252	kip-ft
Moment Mzd =	*0,0*	*kNm*	= 0	kip-ft
Axial force Nd =	*0,0*	*kN*	= 0	kips
Shear force Vzd=	*186,9*	*kN*	= 42	kips
$M_u = M_{yd}+0{,}5N_d(h_{prof}-t_f)+M_{zd}\,h_0/(b_p+g)$=	***341,71***	kNm	= 252	kip-ft

Max unit check: 0,92 OK

End-Plate Yield

$$\phi M_n = \phi_b M_{pl} = \phi_b F_{py} t_p^2 Y$$

$$Y = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{f,i}}+\frac{1}{s}\right)+h_0\left(\frac{1}{p_{f,o}}\right)-\frac{1}{2}\right]+\frac{2}{g}\left[h_1\left(p_{f,i}+s\right)\right]$$

Note: Use $p_{f,i} = s$, if $p_{f,i} > s$

$$s = \frac{1}{2}\sqrt{b_p g} \qquad \phi_b = 0.90$$

Bolt Rupture w/Prying Action

$$\phi M_n = \phi M_q = \max \begin{vmatrix} \phi\left[2(P_t - Q_{max,o})d_0 + 2(P_t - Q_{max,i})d_1\right] \\ \phi\left[2(P_t - Q_{max,o})d_0 + 2(T_b)(d_1)\right] \\ \phi\left[2(P_t - Q_{max,i})d_1 + 2(T_b)(d_0)\right] \\ \phi\left[2(T_b)(d_0 + d_1)\right] \end{vmatrix} \qquad \phi = 0.75$$

Plate thickness check

ΦM_n =	401 kNm	>	$\gamma_r M_u$=	342 kNm	0,85 OK
	= 3 553 kip-in.			= 3 024 kip-in.	

Bolts check

Bolts 6x 1 in. Class A325

d_{B0}= 1 1/8 in. (d_b+1/16 in.)

Shear plane in thread? Yes — Ft= 620 MPa — F_{nt}= 314 kN tension force per 1 bolt

Pretension (*fully tightend*): T_b= 267 kN — F_{nv}= 188 kN shear force per 1 bolt

$w = b_p/2 - (d_b + 1/16) =$ 2,62 in. = 66,61 mm

$a_i = 3.682\left(\frac{t_p}{d_b}\right)^3 - 0.085 =$ 2,4 in.= 60 mm

$$F_i' = \frac{t_p^2 F_y\left(0{,}85\frac{b_p}{2}+0{,}80w'\right)+\frac{\pi d_b^3 F_t}{8}}{4p_{f,i}} =$$ 135 kN = 30,3 kips

$$Q_{max,i} = \frac{w' t_p^2}{4a_i}\sqrt{F_y'^2 - 3\left(\frac{F_i'}{w' t_p}\right)^2} =$$ 26,4 kN = 5,9 kips

$$a_o = 3.682\left(\frac{t_p}{d_b}\right)^3 - 0.085 \le (p_{ext} - p_{f,o})$$ = 38 mm = *1,5 in.*

$$F_o' = \frac{t_p^2 F_y\left(0{,}85\frac{b_p}{2}+0{,}80w'\right)+\frac{\pi d_b^3 F_t}{8}}{4p_{f,o}} =$$ 135 kN = 30,3 kips

$$Q_{max,o} = \frac{w' t_p^2}{4a_o}\sqrt{F_y^2 - 3\left(\frac{F_o'}{w' t_p}\right)^2} =$$ 41,9 kN = 9,4 kips

ΦM_n =	371 kNm	>	M_u=	342 kNm	0,92	OK
	= 3 282 kip-in.			= 3 024 kip-in.		

The different stiffness of extended part (in CBFEM calc.) probably causes the non-linear force distribution in bolts. Therefore, the force in inner bolts is bigger than in outside ones. The standard approach in Design guide 16 is taken as the linear distribution along the height of connection, therefore the force in outside bolts is greater than in inner ones.

However we can compare the final bolts' capacity, which is around 0.92 = **92%**

Comparison:

The results show, that the force computed using CBFEM method by IDEA StatiCa is almost identical with the standard approach in AISC Design Guides.

The stress of end-plate in IDEA StatiCa calculation is beyond the yield strength and the end-plate undergoes a plastification. The strain limit is under the 5% limit. The check using AISC Design Guide 16 gives a unit ratio 0.85 which is quite close to the limit. The check of end-plate is satisfactory in both software calculation and DG calculation.