Eccentrically Braced Frames with Removable Links – Design Example

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Date: 16th January 2013
Ref.: EQK1007

Key Words
Eccentrically Braced Frame, EBF, Removable Links

Introduction
Steel eccentrically braced frames are expected to sustain damage during a design level earthquake through repeated inelastic deformation of the active link. A method for assessment of the extent of inelastic strain in the active link is currently being developed. Repair is therefore expected to be costly and disruptive, even if the structure has met its goal of providing life safety during an earthquake. The replaceable or removable active link concept addresses these drawbacks. It allows for quick inspection and replacement of damaged links following a major earthquake, significantly minimising the disruption of the structure.

The development, research, design methodology and detailing considerations of eccentrically braced frames with removable links are covered in Steel Advisor articles EQK1005 and EQK1006 (the former is in preparation at the time of writing this article). This article illustrates the application of the proposed eccentrically braced frame with removable link design procedure by way of a design example. Understanding of the design of conventional eccentrically braced frames has been assumed. For those not familiar with the seismic behaviour and design of eccentrically braced frames reference should be made to HERA Report R4-76.

Design Example Description
The design example covers the design of the bottom frame of an eight storey EBF frame. The bottom frame detail is shown in figure 1. The removable active link is a hot rolled universal column (UC) section. The collector beams are a universal beam (UB) section with a depth greater than the removable link section to facilitate ease of removal. The removable link is connected with an extended end plate bolted connection to a flush end plate on the collector beams. The UC braces are welded to the collector beam. Note that one flange of the UC brace is connected to the flush end plate on the collector beam.

The design example in this article covers the member design of the removable link, collector beam and braces. The detailing of the connection between the removable link /collector beam/brace is also covered.

The bottom frame width (centreline to centreline) is 9m. The frame height (centreline to centreline) is 4.5m.

The eccentrically braced frame has been designed as a seismic category 2 frame for a ductility (μ) of 3. The steel grade for the UC removable link members is taken as grade 300S0 to AS/NZ3679.1. Therefore the overstrength factor as per NZS3404 Table 12.2.8(2) is $\phi_{\text{ens}} = 1.3$. 

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In conventional EBFs where the collector beam and active link is the same member, the collector beam may be designed for some inelastic action (NZS 3404 cl12.11.7.2) and must be considered a primary member. This is not the case for EBFs with removable links, where the collector beam is protected from inelastic demand by a capacity design approach. This allows for some relaxation in the material and section geometry requirements. The member seismic designations for EBFs with removable links are:

1. Link: primary member
2. Brace/collector beam: secondary members

For the sake of clarity, gravity loads (dead and live) on the beams will be ignored. Where dead and live loads may need to be considered in the design of a member, mention of this has been made in the calculations.

It has been assumed that the interstorey drifts are within the bounds outlined in NZS1170.5 Section 7 and that serviceability criteria are met.

**Notation**
The notation used in the example calculations are as follows. Member and section capacity notation is taken from NZS3404.

**Geometry notation:**

- L: Frame width (centreline to centreline)
- e: Active link clear length
- Hs: Frame height (centreline to centreline)
- L_{brace}: Length of brace (centreline to centreline)
- L_{beam}: Length of collector beam (centreline to centreline)

**Load demand notation:**

- N^*: Design load demand for a member
- N^*_e: Load demand from design level earthquake analysis
- N^*_{G+\psi Q}: Load demand from dead and live loads
- N^*_C: Capacity design derived design actions

**Factoring notation:**

- \(\phi_{oms}\): Overstrength factor from NZS3404
- \(\phi_o\): Ratio between overstrength actions and the actions induced by the design level earthquake.
- \(\phi_{emax}\): Ratio between analysis actions (\(\mu=3\)) and upper bound (\(\mu=1.25\)) actions
**Figure 1: EBF Bottom Frame Link Region**

**Member design of bottom frame**

**Removable Active Link**

Design action $V^*_{e,i} = 500kN$

(Half the lateral force is assumed to be applied to both sides of the frame, and thus no axial load is transferred through the replaceable link)

Try 310UC118

$\phi M_{ss} = 494$ kNm

$\phi V_w = 607$ kN

Check active link length

$$e \leq \frac{1.6 \times M_{ss}}{V_w}$$

$$e \leq \frac{1.6 \times 494}{607} \quad e \leq 1.3m \quad e = 0.6m \rightarrow ok$$

Check the inelastic rotation angle between the active link and the adjacent beam.

$\gamma_p \leq 0.08$ rad

For this building example the inelastic storey rotation angle, $\theta_p$, was found to be 0.0040 radians.

$$\gamma_p = \frac{L}{e} \theta_p$$

$$\gamma_p = \frac{9}{0.6} \times 0.0040 \quad \gamma_p = 0.06 \text{ rad} \rightarrow ok$$

Check $\phi V_w \geq V^*_{e,i}$

$\phi V_w = 607kN \geq V^*_{e,i} = 500kN \rightarrow ok$
Find $\phi_o$

$$\phi_o = \frac{V_w \times \phi_{oms}}{V_e}$$

$$\phi_o = \frac{(607 / 0.9) \times 1.3}{500} \quad \phi_o = 1.75 \quad \text{N.B: use } V_w \text{ here not } \phi V_w \text{ in conjunction with } \phi_{oms}$$

$(\mu=3.0, \text{table 12.2.8(2)})$

Compare with the ratio between limited ductile and elastic actions.

$$\phi_{e_{\max}} = \frac{k_{\mu=3}}{k_{\mu=1.25}}$$

$$\phi_{e_{\max}} = \frac{3}{1.25} \quad \phi_{e_{\max}} = 2.4$$

$\phi_o < \phi_{e_{\max}}$ Use overstrength design actions

Check moment in link

$$M_{sai} = 0.75M_s \quad \text{NZ3404 12.11.5 (b)}$$

$$M^* = V_e \times e / 2 \leq \phi M_{sai}$$

$$M^* = 500 \times \frac{0.6}{2} \leq \frac{0.75 \times 494}{2} \quad 150 \leq 371 \quad \rightarrow \text{ok}$$

310UC118 L01 link $\rightarrow$ ok

**Brace**

$\phi_o \leq \phi_{e_{\max}} \quad \text{NZS3404 12.11.3.9}$

Find the axial load in the brace from the shear force in the active link.

$$N_{brace, e}^* = V_e \times \frac{L/2}{L_{beam}} \times \frac{L_{brace}}{H_s}$$

(centreline to centreline dimensions are used for determining loads)

$$L_{brace} = \sqrt{4.2^2 + 4.5^2} \quad L_{brace} = 6.16m$$

$$N_{brace, e}^* = 500 \times \frac{9/2}{4.2} \times \frac{6.16}{4.5} \quad N_{brace, e}^* = 733kN$$

Multiply by the overstrength factor and add the gravity and live load actions to obtain the axial load demand in the brace.

$$N_{brace}^e = N_{brace, e}^* \times \phi_o + N_{brace G+V, Q}^*$$

NZS3404 12.11.3.9
\[ N_c^{\text{brace}} = 733 \times 1.75 + 0 \quad N^*_{\text{brace}} = 1283kN \]

(For simplicity assume \( N^*_{\text{brace}G+Q, Q} = 0 \))

Check capacity (250UC89.5)

\[ \phi N_c \geq N_c^{\text{brace}} \]

Assume effective length of brace is based on centreline dimensions.

\[ L_{\text{brace}} = 6.16m \]

From ASI Design Capacity Tables

\[ \phi N_{cy} = 1572kN \geq N_c^{\text{brace}} = 1283kN \quad \rightarrow \text{ok} \]

Check axial force limit \( \text{NZ3404 12.8.3.1, Table 12.8.1.} \)

\[ 0.8 \phi N_s \geq N_c^{\text{brace}} \]

\[ 0.8 \phi N_s = 0.8 \times 2870 = 2296kN \geq N_c^{\text{brace}} = 1283kN \quad \rightarrow \text{ok} \]

The brace will be welded to the active link/collector beam junction. In accordance with NZS 3404 12.11.7.7 this connection must be analyzed as rigid and the braces designed for the capacity design actions.

Find the moment in the brace from the shear force in the active link.

First determine the overstrength moment in the link

\[ M_{\text{link}}^* = V_e^* \times \phi_o \times e / 2 = 500 \times 1.75 \times 0.6 / 2 = 263kNm \]

The moment in the brace is determined by the ratio of stiffness of collector beam and brace

\[ M_{\text{brace}}^* = \left( \frac{I_{\text{brace}}}{I_{\text{collector beam}}} \frac{L_{\text{brace}}}{L_{\text{collector beam}} + \frac{I_{\text{brace}}}{I_{\text{brace}}}} \right) M_{\text{link}}^* \]

\[ M_{\text{brace}}^* = \left( \frac{143}{761} \frac{6.16}{4.2} + \frac{143}{6.16} \right) \times 263 = 30kNm \]

Check combined actions

First determine if brace has full lateral restraint.

Take triangular moment distribution

\[ \alpha_m = 1.75 \quad \text{NZS 3404 Table 5.6.1} \]

Using ASI Design Capacity Tables or Memdes for 250UC90 and Le = 6.16m

\[ \alpha_s = 0.73 \]

\[ \alpha_m \alpha_s = 1.3 > 1.0 : \text{Full Lateral Restraint} \]
As there is full lateral restraint the combined action check that will govern is NZS 3404 8.4.2.2

\[
\phi M_{brace} = \phi M_{sx,brace} \left( 1 - \frac{N^*_{brace}}{\phi N_{c,brace}} \right) = 369 \left( 1 - \frac{1283}{2344} \right) = 167 \text{kNm} > 30 \text{kNm} \quad \rightarrow \text{ok}
\]

\[250UC90 \ L01 \ brace \quad \rightarrow \text{ok}\]

**Collector beam**

Required depth of collector beam:

Depth of active link plus allowance for extended endplate (this includes sufficient gap for the timber packer to support the deck during construction)

\[
= 315 + 240 = 555 \text{mm}
\]

Try 610UB101

For EBFs with removable links, the collector beam overstrength design actions may not be reduced to 80\% as per clause 12.11.7.2 to avoid inelastic demand.

Find the design axial and bending actions.

Find overstrength axial demand in beam.

\[
N^c_{\text{beam}} = 1.0 \times N^c_{\text{brace}} \times \frac{L_{\text{beam}}}{L_{\text{brace}}} \quad \text{NZS3404 12.11.7.2}
\]

\[
N^c_{\text{beam}} = 1.0 \times 1283 \times \frac{4.2}{6.16} \quad N^c_{\text{beam}} = 875 \text{kN}
\]

Find overstrength bending demand in the beam.

\[
M^{*}_{\text{beam}} = 1.0 \times V^*_{e} \times \phi_o \times e / 2 + M^{*}_{\text{beam}(G+\psi,Q)}
\]

(For simplicity assume \(M^{*}_{\text{beam}(G+\psi,Q)} = 0\) and ignore any reduction due to moment carried by 250UC brace)

\[
M^{*}_{\text{beam}} = 1.0 \times 500 \times 1.75 \times 0.6 / 2 + 0 \quad M^{c}_{\text{beam}} = 263 \text{kNm}
\]

Check axial capacity

Effective length for axial load

\[
L_{\text{beam,e}} = 4.2 - D_{\text{col}} / 2 = 4.2 - 0.355 / 2 = 4.02 \text{m}
\]

\[
\phi N_{c_y} = 1960 \text{kN} \geq N^*_{\text{beam}} = 875 \text{kN} \quad \rightarrow \text{ok}
\]

Second order effects. \quad \text{NZS3404 4.4.3.2}

This has not been specifically checked in this example but assumed to be \(\delta_o = 1\). This will be the case unless there is significant gravity loading on the collector beam, such as that from a major secondary beam near the centre of the collector beam. This check is only required for case 2.

**Case 1.** The collector beam is in tension. The compression flange from bending is the bottom flange. No collector beam second order effect check is required.
First determine if collector beam has full lateral restraint
Using ASI Design Capacity Tables or Memdes for 610UB101
Effective length for bending \( L_{beam,e} = 4.2 - D_{col} / 2 = 4.2 - 0.355 / 2 = 4.02 \text{m} \)
\( \alpha_s = 0.68 \)
\( \alpha_m = 1.75 \)

\( \alpha_m \alpha_s = 1.19 > 1.0 : \text{Full Lateral Restraint} \)
As there is full lateral restraint the combine action check that will govern is NZS 3404 8.3.2
Check the conservative formula first.
\[
\phi M_{beam} = \phi M_{xx} \left( 1 - \frac{N_{beam}^*}{\phi N_{t,beam}} \right) = 782 \left( 1 - \frac{875}{3510} \right) = 587 \text{kNm} > 263 \text{kNm} \rightarrow \text{ok}
\]
No need to check alternative provision as capacity is sufficient using the conservative method.

Case 2. The collector beam is in compression. The compression flange from bending is the top flange. A second order effects check is required but as noted on the previous page will generally not result in first order magnification.

The beam has full lateral restraint as the compression flange is restrained by the slab.

\[
M_{beam}^* \leq \phi M_{xx} \left( 1 - \frac{N_{beam}^*}{\phi N_{c}^*} \right) \quad \text{NZS3404 8.4.2.2}
\]

\( \phi N_{c}^* = 3080 \text{kN} \)

\[
M_{beam}^* \leq 783(1 - \frac{875}{3080}) = 561 \text{kNm}
263 \text{kNm} \leq 561 \text{kNm} \rightarrow \text{ok}
\]

Check the limitations on axial force. \quad \text{NZS3404 12.8.3.1}

Limit (a) \( N^* < 0.8 \phi N_s \) \quad \text{Table 12.8.1}
\[ 875 \text{kNm} \leq 2488 \text{kN} \rightarrow \text{ok} \]
Limit (b) - Does not apply, no inelastic demand in the collector beam
Limit (c) - does not apply for the collector beam as there is no axial load due to gravity.

\rightarrow \text{Axial load limits ok}

The final check for the collector beam is the shear capacity of the beam/brace/column panel zone.
This panel zone check is similar to that shown on page 11.

\( 610UB101 \ L01 \ collector \ beam \rightarrow \text{ok} \)
Bottom Frame Detailing of Removable Active link Connection Region

*Removable Link End Plate Connection*

SCNZ Steel Connect is used as starting point for connection details.

Find the overstrength moment demand

\[ M_{\text{link}}^e = \phi_{\text{oms w}} V_w \frac{e}{2} \]

\[ M_{\text{link}}^e = 1.3 \times \frac{607}{0.9} \times \frac{0.6}{2} = 263 \text{kNm} \]

\[ \frac{M_{\text{link}}^e}{\phi M_{\text{sx}}} = \frac{263}{494} \times 100 = 53\% \]

Try MEP 50/25

- Moment Capacity \( \Phi M_{\text{con}} = 341 \text{kNm} \)
- End plate thickness \( t_p = 25\text{mm} \)
- End plate dimensions – 560mm long x 320mm wide
- Bolts - 8 M30
- Flange weld – BW
- Web Weld – 10mm

The welds in Steel Connect MEP connections are designed to develop the design tension capacity of the flange and web.

![Isometric drawing of a Moment End Plate (MEP) connection in SCNZ Steel Connect](image)

**Figure 2:** Isometric drawing of a Moment End Plate (MEP) connection in SCNZ Steel Connect

An additional requirement to ensure ductile behaviour is:

\[ t_p \leq 0.9 d_f \quad \text{where} \quad d_f \quad \text{is the fastener diameter} \]

*Check bolts have sufficient shear capacity*

Find the overstrength shear demand

\[ V^c = \phi_{\text{oms w}} V_w \]

\[ V^c = 1.3 \times \frac{607}{0.9} = 877 \text{kN} \]

As a starting point assume shear carried by bolts on compression side of connection.

Try 4 bolts to take shear demand

\[ \phi V_{fn} \geq \frac{V^*}{4} \]

\[ \phi V_{fn} \geq \frac{877}{4} = 219 \text{kN} \]
M30 $\phi V_{fn} = 214 \text{kN} $ Close – Consider excess capacity of bolts around tension flange to also take proportion of shear.
Additional shear to be taken each bolts around tension flange = $219 - 214 = 5 \text{kN}$

Find the overstrength beam flange tension

$$ N_{cf}^e = \frac{M^c}{d_h - t_f} $$

$$ N_{cf}^e = \frac{263}{315 - 18.7} \times 10^3 = 888 \text{kN} $$

There are 4 bolts to take tension

$$ \phi N_g \geq \frac{N_{cf}^e}{4} $$

$$ \phi N_g \geq \frac{888}{4} = 222 \text{kN} $$

M30 $\phi N_g = 373 \text{kN} \rightarrow OK$

Check shear-tension interaction, including the effects of bolt prying. This means taking $N^* = N_{ti} = 0.9 \phi N_g = 335 \text{kN}$

$$ \left( \frac{V^*}{\phi V_{fn}} \right)^2 + \left( \frac{N^*}{\phi N_g} \right)^2 \leq 1.0 $$

$$ \left( \frac{5}{214} \right)^2 + \left( \frac{335}{373} \right)^2 = 0.81 \leq 1.0 \rightarrow ok $$

Collector Beam End Plate Connection

End plate thickness
It is important to avoid excessive deformation of the collector beam end plate to facilitate ease of removal of the active link. This is achieved by ensuring the tension capacity of the collector beam end plate exceeds that of the active link end plate. This is achieved by:

1) Making the collector beam end plate thicker than the active link end plate or
2) Adding a 12mm backing plate to the second row of bolts (between the flanges)

End plate thickness $\rightarrow 25\text{mm with backing plates for row 2 bolts}$

Horizontal stiffener
The stiffener plate should be selected to have similar thickness as the active link beam flange thickness.
Active link beam flange thickness $=18.7\text{mm}$, select 20mm thickness for stiffener.

The weld of the stiffener to the end plate is to develop the design tension capacity of the stiffener. A full strength butt weld is detailed.

Check fillet welds from horizontal stiffener to beam web.

The horizontal stiffeners welds are sized to transfer the active beam flange overstrength tension force to the collector beam web.
Weld demand
\[ V_w^* = N_{wf}^* \]
\[ V_w^* = 888 kN \]

Try 5mm fillet weld. Stiffener is on both sides so weld length is 4 times the stiffeners length.

Stiffener length
\[ L_{stiffener} = d_{brace} \times \frac{6.16}{4.2} = 225 \times \frac{6.16}{4.2} = 330 mm \]
Length of weld = 4\times(330) = 1320mm

Using Australian Steel Institute Capacity Tables
\[ \phi N_{ww} = 0.815 \times 1320 = 1076 kN > 888 kN \rightarrow ok \]

Check horizontal tearout of beam web
\[ \phi N_{horiz} = 0.9 \times 0.6 \times 330 \times 10.6 \times 320 \times 10^{-3} = 1209 kN > 888 kN \rightarrow ok \]

**Horizontal stiffener** → 20mm thickness with full strength butt weld to end plate and 5mm fillet weld to collector beam web

**Weld collector beam to end plate**
Check weld of beam web to end plate
Weld to develop the design tension capacity of the web
\[ N_{ww}^* = 0.9 t_w f_{yw} \]
\[ N_{ww}^* = 0.9 \times 10.6 \times 320 / 1000 = 3.1 kN / mm \]
SP weld, \( f_{uw} = 480 \) MPa
10mm fillet weld both side
\[ \phi N_{ww} = \phi N_{w} \times 2 \]
Using Australian Steel Institute Capacity Tables
\[ \phi N_{ww} = 1.63 \times 2 = 3.26 kN / mm \cdot OK \]

**Brace connection to collector beam**
A conservative force distribution between the brace flanges and webs will be assumed for the design of each component. The greater of two distributions will be used for the design of any component. The two distributions are, \( \frac{1}{2} \) the load to each of the flanges and \( 1/3 \) of the load to the web and each of the flanges.

**Check transfer of load from brace to end plate**
Outside brace flange is to have complete penetration butt weld to end plate.

Weld demand
\[ N^* = \frac{N_{brace}}{2} = \frac{1283}{2} = 642 kN \]

\( (1/2 \) force through each brace flange)\)

The capacity of a complete penetration butt weld is to be equal to the capacity of the weaker part joined.

The thickness of the brace flange is 17.3mm<25mm thickness of the end plate.
The width of the brace flange is 256mm<320 mm width of the end plate.

Weld capacity \( \phi N_i = \phi b_{f,brace} t_{f,brace} f_{y,brace} = 0.9 \times 256 \times 17.3 \times 280 \times 10^{-3} = 1116kN > 642 \rightarrow \text{ok} \)

Check transfer of load from brace to collector beam flange and vertical stiffener
Inside brace flange will be butt weld to collector beam flange with 16mm stiffener butt weld to beam flange above.

Weld demand \( N^*_{brace \ flange} = \frac{N^c_{brace}}{2} + \frac{M^c_{brace}}{d_{brace} - t_{f,brace}} = \frac{1283}{2} + \frac{30}{0.26 - 0.0173} = 766kN \)

(1/2 axial force through brace flange plus flange force due to moment)

The capacity of a complete penetration butt weld is to be equal to the capacity of the weaker part joined.

The thickness of the stiffener is 16mm<17.3mm thickness of brace flange.
The width of the collector beam flange is 228mm<256 mm width of the brace flange.

Weld capacity \( \phi N_i = \phi b_{f,beam} t_{f,beam} f_{y,beam} = 0.9 \times 228 \times 17.3 \times 280 = 994kN > 766 \rightarrow \text{ok} \)

Brace web is to have fillet welds to collector beam flange. Weld to develop the design tension capacity of the web
\( N^*_{ww} = 0.9 t_w f_{yw} \)
\( N^*_{ww} = 0.9 \times 10.5 \times 320 / 1000 = 3.0 kN / \text{mm} \)
SP weld, \( f_{yw}=480 \text{ MPa} \)
10mm fillet weld both side
\( \phi N_{ww} = \phi v_w \times 2 \)
Using Australian Steel Institute Capacity Tables
\( \phi N_{ww} = 1.63 \times 2 = 3.26 kN / \text{mm} > 3.0 \rightarrow \text{ok} \)

Check fillet welds from stiffener to beam web
The vertical component of the brace flange force needs to be transferred to the beam web.

Weld demand \( V^*_{w} = N^*_{brace \ flange} \times \frac{H_s}{L_{brace}} = 766 \times \frac{4.5}{6.16} = 560kN \)

Try 5mm fillet weld. Stiffener is on both sides so weld length is 4 times the beam depth between flanges.

Length of weld = \( 4 \times (602 - 2 \times 14.8) = 2290mm \)
Using Australian Steel Institute Capacity Tables
\( \phi N_{ww} = 0.815 \times 2290 = 1866kN > 560 \rightarrow \text{ok} \)

Collector Beam / Brace Panel Zone
Check shear capacity of panel zone
The panel is subject to axial load and shear. The design actions on the panel are:
1. Force couple due to \( M^c_{link} \)
2. Axial force in collector beam
3. Shear force from vertical component of brace force

Figure 3: Design actions in panel zone - Force couple due to $M_{\text{link}}$

Figure 4: Design actions in panel zone – axial force in collector beam

Figure 5: Design actions in panel zone - shear force from vertical component of brace force

Use parabolic shear/axial load interaction equation for panel zone.

Design actions in the upper half of the panel zone (zone 1) are critical as the design actions from the active link moment couple and the collector beam axial force are additive. The top half of the panel (zone 1 in figure 3) will therefore be checked for combined actions.

Axial load on panel

$$N_p^* = N_{c_t}^* + \frac{N_{\text{beam}}^*}{2} = 888 + \frac{875}{2} = 1326 \text{kN}$$
Shear load on panel zone for half the beam depth

Assuming load distribution is 1/3 from the brace web and 1/3 from the brace flange. One brace flange is welded directly to the collector beam end plate. Therefore a third of the link shear is transferred directly to this brace flange and not through the panel zone.

\[ V_p^* = V_{\text{link}} \times \frac{2/3}{2} = \frac{585}{2} = 293 \text{kN} \]

For the top half of the beam the axial action is distributed to the beam web, flange and effective stiffener width.

Axial capacity for half the beam depth

\[ \phi N_t = \phi f_{\text{beam}} f_{\text{yf,beam}} + \phi \left( \frac{d_{\text{beam}}}{2} - t_{f,\text{beam}} \right) t_{w,\text{beam}} f_{\text{yw,beam}} + 2 \phi t_{\text{stiffener}} b_{\text{effective, stiffener}} f_{\text{yw, stiffener}} \]

\[ \phi N_t = 0.9 \times 228 \times 14.8 \times 300 + 0.9 \left( \frac{602}{2} - 14.8 \right) 10.6 \times 320 + 2 \times 0.9 \times 20 \times 100 \times 250 = 2685 \text{kN} \]

Shear capacity for half the beam depth

\[ \phi V_p = \phi 0.6 \frac{d_{\text{beam}}}{2} t_{w,\text{beam}} f_{\text{yw,beam}} \]

\[ \phi V_p = 0.9 \times 0.6 \times \frac{602}{2} \times 10.6 \times 320 = 551 \text{kN} \]

Shear/tension interaction

\[ \left( \frac{V_p^*}{\phi V_p} \right)^2 + \left( \frac{N_t^*}{\phi N_t} \right)^2 = \left( \frac{293}{551} \right)^2 + \left( \frac{1326}{2685} \right)^2 = 0.53 \leq 1.0 \rightarrow \text{ok} \]

References


