Web Side Plate Rotation Capacity

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Introduction
The Steel Construction New Zealand publication *Steel Connect* (SCNZ 14.1 and SCNZ 14.2) provides structural engineers with a rapid and cost-effective way to specify the majority of structural steelwork connections, in accordance with accepted fabrication industry norms. Specification of these connections also facilitates the development of reliable cost estimates by designers, fabricators, consulting quantity surveyors and constructors.

Steel Connect contains bolted web side plate (WP) connections. An example of a web plate connection is shown in figure 1. A number of limit states are checked. The connections are specifically designed to accommodate rotational demands on the connection. Designers of non-standard WP connections must ensure rotational demands are checked. This is especially an important check to be carried out for long span beams with precamber. There has been an instance of a construction project involving long span precambered beams where a number of bolts were sheared off a non-standard WP connection. Carrying out the rotational checks in *Steel Connect* for this project would have alerted the designer to the potential of bolts shearing off. This article summarizes the rotational checks from *Steel Connect*.

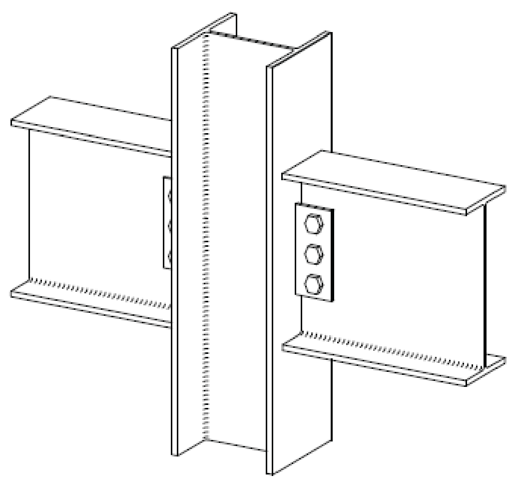

Figure 1: Web Plate Connection (Hyland et al, 2008)

Rotation Capacity Checks
For beams connected to columns with web side plates the following is used in *Steel Connect* to ensure that the connection can meet the rotational demand. These are:

1) The plate or web thickness shall not exceed half the bolt diameter
2) The bolt group design flexural capacity must exceed the minimum design flexural strength limit of the plate or section web.
3) There is adequate end gap between the support and the beam to accommodate 0.030 radians of relative rotations.

An example is used to illustrate these three checks.

Example

Check the rotational demands on the connection in figure 2 can be met. Notation is as per *Steel Connect*.

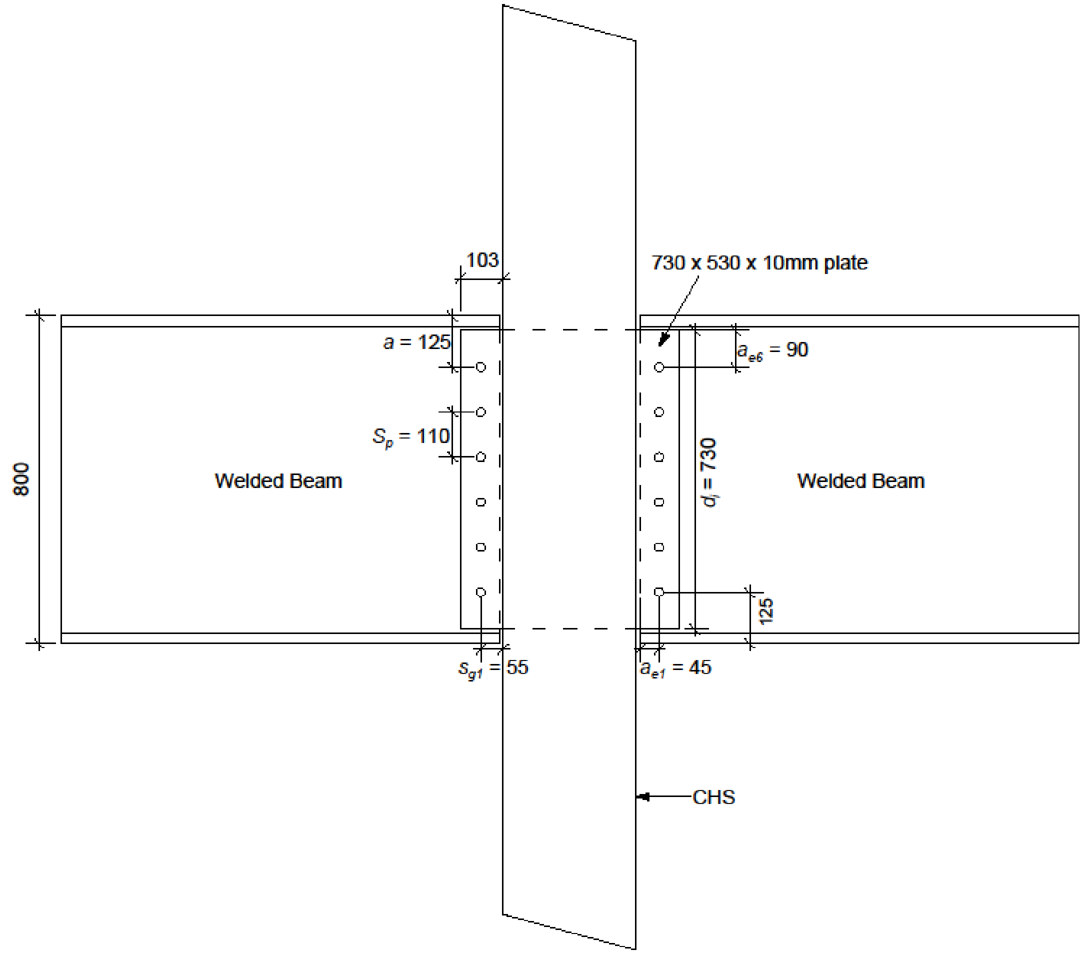

Figure 2: Example WP Connection to be Checked

Material Properties

Plate

Grade 250 to AS/NZS 3678:2011

$f_y = 260\,\text{MPa}$

$f_{ui} = 410\,\text{MPa}$

Check Plate or Web Thickness to Bolt Diameter

M20 bolts are used in the connection. The plate and section web thickness is 10mm.

$$t_i \le \frac{d_f}{2}$$

where:

t_i = cleat plate thickness = 10mm

d_f = fastener diameter = 20mm

$$\therefore 10 \le \frac{20}{2} = 10 \quad \rightarrow \text{OK!}$$

Flexural Capacity of the Bolt Group

$\phi M_b = n_p Z_e \phi V_f e$

where:

n_p = number of bolt rows = 6

Z_e = bolt group flexure factor $= \frac{s_p\ n_p + 1}{6e} = \frac{110 \times\ 6 + 1}{6 \times 55} = 2.33$

ϕV_f = Bolt design shear capacity = 92.6 kN, M20 bolts threads included in shear plane

e = reaction eccentricity = s_{g1} = 55 mm

$\therefore \phi M_b = 6 \times 2.33 \times 92.6 \times 55 / 1000 = 71$ kNm

Design Flexural Strength of Plate

Bolt hole 1st transverse tearing

$\phi V_{tti} = n_p \phi a_{eyi} t_i f_{ui}$

where:

a_{eyi} = edge dist an ce = $\min(a_{e3}, a_{e6}) = \min(99, 90) = 90$mm

$a_{e3} = s_p - \frac{d_h}{2} = 110 - \frac{22}{2} = 99$mm

d_h = hole diameter = $d_f + 2 = 20 + 2 = 22$mm

$\therefore \phi V_{tti} = 6 \times 0.9 \times 90 \times 10 \times 410 / 1000 = 1993$kN

Cleat gross flexure yield

$$\phi V_{gfi} = \phi \frac{t_i d_i^2}{4e} f_{yi}$$

$$\phi V_{gfi} = 0.9 \times \frac{10 \times 730^2}{4 \times 55} \times 260 = 5668\text{kN}$$

Cleat net flexure ultimate

$$\phi V_{nfi} = \phi \left(1 - \frac{n_p d_h}{d_i}\right) \frac{t_i d_i^2}{4e} 0.85 f_{ui}$$

$$\phi V_{nfi} = 0.9 \left(1 - \frac{6 \times 22}{730}\right) \frac{10 \times 730^2}{4 \times 55} \times 0.85 \times 410 = 6223\text{kN}$$

$\therefore$ Design flexural strength of plate

$$M^*_{drift} = e\left[\phi V_{tti}; \phi V_{gfi}; \phi V_{nfi}\right]_{min}$$

$$M^*_{drift} = 55 \times \left[1993; 5668; 6223\right]_{min} = 110\text{kNm}$$

Design Flexural Strength of Section Web

The welded beam is Grade 300. The web is 10mm. By inspection the design flexural strength of the section web is greater than the design flexural strength of the plate.

Compare Flexural Strength of Bolt Group with the Minimum Design Flexural Strength Limit of the Plate or Section Web

$\phi M_b = 71$ kNm

$M^*_{drift} = 110$kN

$\therefore M^*_{drift} > \phi M_b \quad \rightarrow$ Not OK!

Check Seismic End Gap

$$\frac{a_c}{s_{g1} - a_{e1}} = \frac{35}{55 - 45} = 3.5 \leq 33 \rightarrow \text{OK!}$$

where:

$$a_c = \text{end gap} = d - a + a_{e6} - d_i = 800 - 125 + 90 - 730 = 35\text{mm}$$

In summary this connection will not meet the rotational demand. The bolt pitch of 110mm is larger than that used in *Steel Connect*. Reducing the bolt pitch and increasing the number of bolts will ensure that the rotational demand can be achieved. Alternatively larger size bolts may also achieve the required rotational ductility.

There are other limit states that require checking. Only the checks for rotation capacity are shown.

Conclusion

It is important that the rotational capacity of non standard web plate connections for long span beams with precamber be checked. The approach given in SCNZ *Steel Connect* is illustrated by the way of a design example. For the complete design of web plate connections other limits must also be checked.

References

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