Semi-Rigid Sliding Hinge Joint

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Semi-rigid connections, sliding hinge joint, bolted connection

Introduction

The Sliding Hinge Joint (SHJ) is a new semi-rigid joint system developed for moment resisting steel frames. It has the ability to remain rigid under in-service conditions or ultimate state wind loading, and to rotate under severe earthquake loadings, returning to the rigid state when the severe earthquake stops.

The joint is designed and detailed such that there is negligible damage to the frame or slabs. The joint has a similar cost to conventional construction.

The SHJ was first developed by Auckland University in conjunction with HERA. A full design procedure and detailing requirements for the joint was developed and published in 2005. (Clifton, 2005) Since 2005 there has been further research undertaken by Canterbury University. A number of building projects have used the SHJ. The outcome of the research and practical applications has lead to modifications of the original design procedure and detailing requirements. (Clifton, 2007) A simplification of the design procedure is being developed and is currently being reviewed. (MacRae et al, 2009)

This article examines the methodology of the design procedure, identifies issues to be considered in the design and presents an example of how to apply the principles.

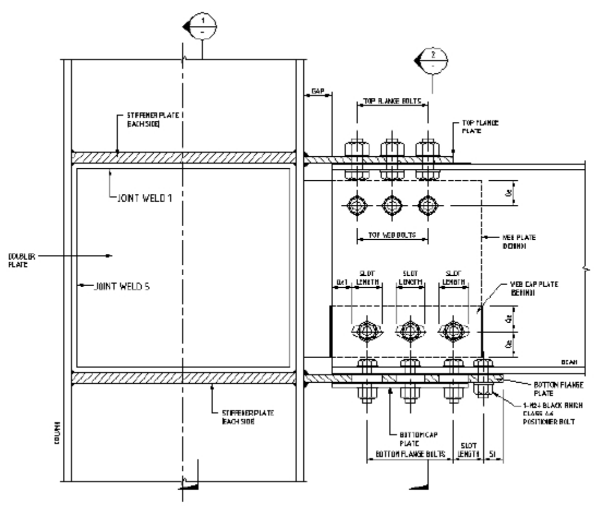

Figure 1: An example of a simple semi-rigid sliding hinge joint (Gledhill et al, 2008)

Description of Joint

The sliding hinge joint is illustrated in figure 1. The beam is pinned laterally at the top flange level. This detail is similar to the semi-rigid flange bolted joint (FBJ). At the top flange level nominal sized bolts holes are used. There is no sliding between the beam, top flange plate and column and thus slab damage is minimised.

Joint rotation is achieved by sliding at the bottom flange bolts and web bottom bolts. Holes for these bolts in the flange plate and web plate are slotted to allow sliding to occur. The slotted holes are sized to accommodate a joint rotation of ±30 mrad multiplied by an over-rotation factor of 1.25. If the inelastic rotation demand exceeds this, the joint undergoes further inelastic behaviour through flange plate yielding, in the same manner as for the FBJ. Below the bottom flange plate is the bottom flange cap plate. A web cap plate over the bottom web bolts is placed on the web plate. The cap plates have no physical connection to the plates apart from through the bolts. The cap plates provide the support to the bolt end remote from the beam of the siding bolt groups. Shims are placed on surfaces where sliding may occur and facilitate smooth sliding between the steel surfaces at a near constant level of shear friction.

The shear force on in the beam is carried by the web top bolts.

A positioner bolt is used for the bottom flange plate and has three roles:

- It acts as a stability bolt for erection purposes, making the joint rigid for erection by developing moment resistance in conjunction with the top flange bolts
- It functions as a locater bolt for the sliding bolts, ensuring that they are located in the middle of the slotted holes in the erected joint
- It provides a rapid visual indicator as to whether the joint has gone into the sliding mode following a severe earthquake.

The mechanism for energy dissipation is by sliding along the bottom flange plate and bottom of the web plate. This is illustrated in figure 2. For the sake of clarity the web plate has not been shown.

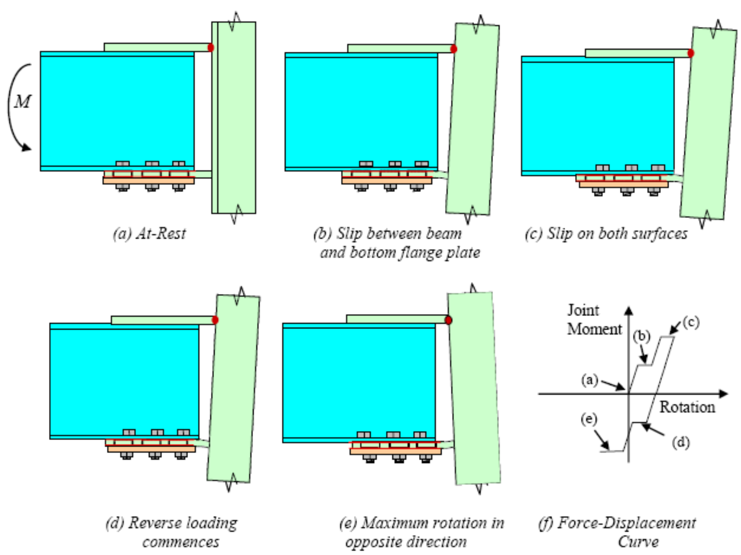

Figure 2: Sliding of Plates Below Beam During Cyclic Deformation (MacRae et al, 2009)

Design Philosophy

The beam is sized to resist the maximum gravity actions as simply supported.

The joint is sized for the moment generated by earthquake action. The joint remains rigid at the serviceability earthquake level, remains reasonably rigid above the serviceability limit state earthquake level and up to the design level ultimate state earthquake moment. The joint is designed to allow inelastic rotation between the beam and column to occur when the design ultimate limit stat earthquake moment is exceeded. The ultimate level earthquake rotation is expected to be accommodated within the slotted hole.

The joint was principally developed as a semi-rigid joint for seismic resisting systems. However the same principles may be used for joint design for the wind ultimate limit state. The SHJ must remain rigid at the wind serviceability limit state. The design rigid moment capacity is 75% of the design moment capacity determined for seismic actions. This is determined by comparing the equation used in determining the bolt serviceability limit shear in clause 9.3.3.1 of NZS3404 (SNZ, 2007) with the equation used for determining the bolt design sliding capacity. See (Clifton, 2005).

Determination of bolt sliding shear capacity

Effect of shim material

The determination of the bolt sliding shear capacity is critical to the design of the sliding hinge joint. The original design procedure developed was based on the use of brass shims. (Clifton, 2005). Testing at Canterbury University has shown that steel shims perform in a similar manner and gives very similar results. (MacRae et al, 2007) Steel shims also have the advantage of being cheaper and more readily available then brass, less potential for corrosion due to dissimilar metal contact and can be tack welded into position onto the steel beams and cap plates for ease of erection. For these reasons steel shims are now used in the design procedure and used in practice. (Sidwell et al, 2008)

Interaction of moment, shear and axial

The bolt sliding shear capacity is dependent on the interaction of moment, shear and axial load on the bolt. Figure 3 shows the bolt sliding shear model as applied to the beam bottom flange. On assembly, the bolt is fully tensioned to the part turn method of NZS3404 (SNZ,2007), clamping all plies rigidly together. When the beam is forced to slide relative to the sandwiched (flange) plate, the bolts must drag the cap plate along with the beam, also sliding relative to the sandwiched plate.

The bolt is already subject to plastic tension force through the full tensioning process, prior to sliding commencing. Once stable sliding occurs, the bolt is subject to combined bending, shear and axial force. The assumed bending moment diagram is shown in figure 4. The effect of the combined actions is to reduce the bolt tension force from that originally developed by the full tensioning process. For the joint to work successfully, the bolt must retain a sufficient percentage of the original installed tension force from that originally developed by the full tensioning process.

To obtain the nominal sliding shear capacity, V_{fss}, the shear, V*, and moment, M*, on the bolt must be expressed as a function of the axial force, N* and this is expressed in (Clifton, 2005) as follows:

$$V^* = N^* \mu_s \tag{1}$$

$$M^* = \frac{V^* d_{lever}}{2} \tag{2}$$

d_{lever} = distance between the centroids of the bearing areas

The bolt moment capacity reduced by axial force and the bolt shear capacity are then determined using the NZS 3404 (SNZ,2007) provisions and plastic theory, as follows:

$$V_{fn} = 0.62 f_{uf} A_c \quad \text{(see clause 9.3.2.1 of NZS 3404)} \tag{3}$$

$$M_{rfn} = S_{fn}\left(1 - \frac{N}{N_{tf}}\right) f_{yf} \tag{4}$$

S_{fn} = plastic section modulus for bolt, based on bolt tensile stress area, A_s

$$N_{tf} = A_s f_{uf} \quad \text{(see clause 9.3.2.2 of NZS 3404)} \tag{5}$$

The interaction of moment, shear and axial force in the bolt

$$\left(\frac{M^*}{M_{rfn}}\right)^2 + \left(\frac{V^*}{V_{fn}}\right)^2 = 1 \tag{6}$$

By substituting equations 1 through 5 into equation 6, an equation can be expressed in terms of N* from which the bolt sliding shear capacities is obtained.

Testing done at Canterbury University has shown that interaction of moment, shear and axial force in the bolt is better represented by a linear relationship. (MacRae et al, 2007) And therefore equation 6 should be replaced with

$$\left(\frac{M^*}{M_{rfn}}\right)+\left(\frac{V^*}{V_{fn}}\right)\leq 1 \qquad (7)$$

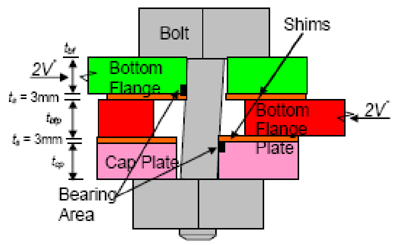

Figure 3: Idealized bolt deformation (MacRae, 2009)

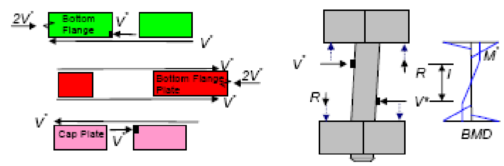

Figure 4: External forces on components

The original SHJ design procedure used a brass shim and therefore the coefficient of friction used between brass and steel was 0.29. The coefficient of friction for steel against steel is in region of 0.30 - 0.35.

From experimental tests the bearing area on a M24 bolts was observed to be 2mm. (Clifton, 2005). For general application expressing the bearing area as function of bolt diameter is more suitable. Therefore the bearing equals 0.1 times the bolt diameter.

Construction Tolerances

Because there are rolling tolerances on beams and plates, the top and bottom flange plates connected to the column flange must be offset vertically to allow for these. However it is also essential that these plates not be placed too far apart. If they are, then even when the bolts are tightened, these plates may not come into full contact with the beams thereby reducing the shear friction force that can be transmitted across the interface

(Clifton, 2005) proposes construction tolerances such that the total unfilled gap between the beam top and bottom flange plates should not exceed 2mm. Finite element studies and discussion in (MacRae, 20009) show the loss of strength is small enough to be ignored for beams detailed to maintain the maximum tolerances gap at around 3mm.

Tolerances may be controlled by ensuring that the beam is bolted to the column plates when the welding of these plates to the column flange is carried out in the factory.

Durability

The design procedure has been developing assuming that the sliding surfaces are between bare steel. Testing has only been done for bare steel. The steel surfaces must be clean and free of any surface coatings, loose scale, loose rust, visible grease or oil marks. Because of these restrictions on the surface condition of the sliding surfaces, the SHJ is principally intended for application in very low corrosion atmosphere, typically found inside heated or air conditioned buildings with clean atmospheres. There is concern that over a period of time if corrosion occurs that the joint may 'lock up' the sliding shear interface.

For more severe atmospheric conditions the contact surfaces for the bottom flange bolts and web bottom bolts must be as specified above. However the non contact surfaces could be protected with an appropriate surface treatment and the edges of the contact surfaces sealed against water ingress. The positioner bolt will need to be painted in these applications.

Overstrength Factor

The column must be designed to resist the overstrength joint moments. The overstrength factor is determined on the basis that the columns will be protected from inelastic demand until the joint has reached stable sliding conditions under each direction of rotation, i.e. both sliding surfaces are actively sliding. The overstrength factor is found based on experimental testing. In (Clifton, 2005) the overstrength factor was found to be approximately 1.4.

Bolt Sliding Capacity Values

Table 1 gives the suggested bolt sliding capacities from (Clifton, 2007). As discussed above there are a number of factors that influence the bolt sliding capacities. Testing may be required to show that the sliding capacities can be achieved and to determine the overstrength factor to use.

Table 1: Suggested Bolt Sliding Capacities (Clifton, 2007)

Bolt Designation	Plate Thickness (mm)	ΦV_{fss} kN Method 1
M16	10	28
M16	12	27
M16	16	24
M20	12	47
M20	16	43
M20	20	40
M24	12	74
M24	16	68
M24	20	63
M30	16	118
M30	20	110
M30	25	102
M36	16	186
M36	20	175
M36	25	162
M36	32	148

Design Example

This design example has been adapted from (Clifton,2005)

A 530UB82 beam is connected to a 610x229x171W column.

1. Design actions

   Design moment

   $M^* = M^*_{E\mu design} = 377\,kNm$

   Design shear

   $V^* = V_{GQu} + V^*_{E\mu design} = 299\,kN$

2. Determine bottom flange width and initial thickness

   M30 bolts are to be used.

2.1 Bottom flange plate width

   $s_g + 3.0 d_f \le b_{bfp} \le 1.05 b_{fc}$

   $120 + 3.0 \times 30 \le b_{bfp} \le 1.05 \times 229$

   $210\,mm \le b_{bfp} \le 240\,mm$

   Try b_{bfp} = 240mm

2.2 Initial estimate of bottom flange plate thickness

   $N^*_t = \dfrac{1.2M^*}{d}$

$$N_t^* = \frac{1.2 \times 377}{0.528} = 857\,kN$$

$$t_{bfp} \geq \frac{N_t^*}{0.9\left(b_{bfp} - 2d_f'\right)f_{ybfp}}$$

$$t_{bfp} \geq \frac{857}{0.9\left(240 - 2 \times 33\right) \times 250} = 21.9\,mm\,;\,use\,20\,mm$$

3. Determine sliding bolt size and numbers for moment adequacy
   Try 6 bottom flange bolts and 3 bottom web bolts

$$\phi M_{SHJ} = n_{bfb}\phi V_{fss}\,d + n_{wbb}\phi V_{fss}\,e_{wb}$$

$$\phi M_{SHJ} = 6 \times 102 \times 0.528 + 3 \times 102 \times 0.423 = 453\,kNm$$

$$\phi M_{SHJ} \geq M^* \;\therefore\; OK!$$

$$e_{wb} = d - t_f - 26.5 - 65$$

$$e_{wb} = 528 - 13.2 - 26.5 - 65 = 423\,mm$$

4. Design of bottom flange plate

4.1 Net tension yield

$$N_{ty}^* = n_{bfb}\phi V_{fss}$$

$$N_{ty}^* = 6 \times 102 = 612\,kN$$

$$\phi N_{tybfp} = \phi_s\left(b_{bfp} - 2d_f'\right)f_{ybfp}t_{bfp}$$

$$\phi N_{tybfp} = 0.9\left(240 - 2 \times 33\right)250 \times 20 \times 10^{-3} = 783\,kN$$

$$\phi N_{tybfp} \geq N_{ty}^* \;\therefore\; OK!$$

4.2 Net tension failure

$$N_{ubfp}^* = n_{bfb}\frac{\phi V_{fss}}{\phi}\phi_{oms}$$

$$N_{ubfp}^* = 6 \times \frac{102}{0.9} \times 1.4 = 952\,kN$$

$$\phi N_{tubfp} = \phi_s 0.85\left(b_{bfp} - 2d_f'\right)f_{ubfp}t_{bfp}$$

$$\phi N_{tubfp} = 0.9 \times 0.85\left(240 - 2 \times 33\right)410 \times 20 \times 10^{-3} = 1092\,kN$$

$$\phi N_{tubfp} \geq N_{ubfp}^* \;\therefore\; OK!$$

4.3 Compression Capacity

Clearance between beam face and column flange

$$f_{SHJ} \geq t_{wbfp} + 0.0375d + 2.5t_{bfp}$$

10mm FW to bottom flange plate has been assumed, to be confirmed later.

$$f_{SHJ} \geq 10 + 0.0375 \times 528 + 2.5 \times 20 \approx 80\,mm$$

Effective length

$$L_{ebfp} = 0.7\left(f_{SHJ} + 0.0375d\right)$$

$$L_{ebfp} = 0.7\left(80 + 0.0375 \times 528\right) = 70\,mm$$

Slenderness

$$\lambda_{nbfp} = \left(\frac{L_{ebfp}}{0.29t_{bfp}}\right)\left(\sqrt{\frac{f_{ybfp}}{250}}\right)$$

$$\lambda_{nbfp} = \left(\frac{70}{0.29 \times 20}\right)\left(\sqrt{\frac{250}{250}}\right) = 12.1$$

Design compression capacity

$$\phi N_{cbfp} = \phi_s \alpha_c b_{bfp} t_{bfp} f_{ybfp}$$

$$\alpha_b = 0.5$$

$\alpha_c = 0.997$ from table 6.3.3(2) NZS3404

$\phi N_{cbfp} = 0.9 \times 0.997 \times 240 \times 20 \times 250x10^{-3} = 1077kN$

$\phi N_{cbfp} \geq N^*_{ubfp} \therefore$ OK!

5. Design of web top bolts
   Try 3 M30 web top bolts to be consistent with the number of web bottom bolts

$V^*_{wv} = 299kN$

$\phi V_b = n_{wtb} \phi_b V_{fn}$

$\phi V_b = 3 \times 214 = 642kN$

$\phi V_b \geq V^*_{wv} \therefore$ OK!

6. Design of web plate

6.1 Check for vertical shear adequacy
At maximum moment

$\phi V_{wp} = 0.6\phi 0.5 d_{wp} f_{ywp} t_{wp}$

$\phi V_{wp} = 0.6\phi 0.5 \times 448 \times 250 \times 20 x10^{-3} = 672kN$

6.2 Check for moment adequacy

$M^*_{wv} = V^*_{wv} e$

$M^*_{wv} = 299 \times 0.235 = 70.3kNm$

$e = a_{e4} + c + 0.5(n_{wtb} - 1)s_{gw}$

$e = 65 + 80 + 0.5(3 - 1)90 = 235mm$

$$\phi M_{wp} = \left[\frac{\phi t_{wp}(d_{wp} - d_{wcp})^2 f_{ywp}}{4}; \frac{\phi t_{wp} d_{wp}^2 f_{ywp}}{6}\right]_{max}$$

$$\phi M_{wp} = \left[\frac{0.9 \times 20(448 - d_{wcp})^2 250}{4}; \frac{0.9 \times 20 \times 448^2 \times 250}{6}\right]_{max} = 150.5kNm$$

$\phi M_{wp} \geq M^*_{wv} \therefore$ OK!

6.3 Check for net tension yield

$N^*_{tywp} = n_{wtb} \phi V_{fss}$

$N^*_{tywp} = 3 \times 102 = 306kN$

$\phi N_{tywp} = \phi(d_{wcp} - d'_f)t_{wp} f_{ywp}$

$\phi N_{tywp} = 0.9(130 - 33)20 \times 250x10^{-3} = 437kN$

$\phi N_{tywp} \geq N^*_{tywp} \therefore$ OK!

6.4 Check for net tension failure

$$N^*_{tuwp} = n_{wtb} \frac{\phi V_{fss}}{\phi} \phi_{oms}$$

$$N^*_{tuwp} = 3 \times \frac{102}{0.9} \times 1.4 = 476kN$$

$\phi N_{tuwp} = \phi 0.85(1.5 d_{wcp} - d'_f)t_{wp} f_{uwp}$

This is a simplification of block shear provisions

$\phi N_{tuwp} = 0.9 \times 0.85(1.5 \times 130 - 33)20 \times 410 \times 10^{-3} = 1016kN$

$\phi N_{tuwp} \geq N^*_{tuwp} \therefore$ OK!

7. Design of top flange bolts and plate

7.1 Number of bolts required

Sufficient top flange bolts are provided to carry the overstrength horizontal force from the bottom flange bolts and web bottom bolts

$$N^*_{tufp} = \left(n_{bfb} + n_{wbb}\right)\frac{\phi V_{fss}}{\phi}\phi_{oms}$$

$$N^*_{tfp} = \left(6 + 3\right)\times\frac{102}{0.9}\times 1.4 = 1428\text{kN}$$

$$\phi V_{btfp} = n_{tfb}\phi_b V_{fn}$$

$$\phi V_{btfp} = 8\times 214 = 1712\text{kN}$$

7.2 Determine top flange plate width
Use $b_{tfp} = b_{bfp} = 240$mm

7.3 Determine top flange thickness
This is sized so that the plate can develop the sliding shear capacity of the bottom flange and web bottom bolts, without tension yielding.

$$N^*_{tfp} = \left(n_{bfb} + n_{wbb}\right)\phi V_{fss}$$

$$N^*_{tfp} = \left(6 + 3\right)\times 102 = 918\text{kN}$$

$$\phi N_{tytfp} = \phi_s\left(b_{tfp} - 2d'_f\right)f_{ytfp}t_{tfp}$$

$$\phi N_{tytfp} = 0.9\left(240 - 2\times 33\right)250\times 25\times 10^{-3} = 979\text{kN}$$

7.4 Check top flange plate and bolt adequacy for the ULS condition

$$\phi N_{tutfp} = \phi_s 0.85\left(b_{tfp} - 2d'_f\right)f_{utfp}t_{tfp}$$

$$\phi N_{tutfp} = 0.9\times 0.85\left(240 - 2\times 33\right)410\times 25\times 10^{-3} = 1365\text{kN}$$

Within 5% accept
Effective length

$$L_{etfp} = 0.7\left(s_{SHJ} + a_{eptfb}\right)$$

$$L_{ebfp} = 0.7\left(80 + 65\right) = 101.5\,\text{mm}$$

Slenderness

$$\lambda_{ntfp} = \left(\frac{L_{etfp}}{0.29t_{tfp}}\right)\left(\sqrt{\frac{f_{ytfp}}{250}}\right)$$

$$\lambda_{nbfp} = \left(\frac{101.5}{0.29\times 25}\right)\left(\sqrt{\frac{250}{250}}\right) = 14$$

Design compression capacity

$$\phi N_{ctfp} = \phi_s\alpha_c b_{bfp}t_{bfp}f_{ybfp}$$

$$\alpha_b = 0.5$$

$$\alpha_c = 0.994$$ from table 6.3.3(2) NZS3404

$$\phi N_{cubfp} = 0.9\times 0.994\times 240\times 25\times 250\text{x}10^{-3} = 1342\text{kN}$$

8. Check beam tension adequacy in the connection region
Check is to suppress yielding of the beam cross section during the sliding phase.
On tension half of beam

$$N^*_{tb} = 0.5\frac{\phi M_{SHJ}}{\phi M_{sx}}N_t$$

$$N^*_{tb} = 0.5\times\frac{453}{558}\times\frac{2840}{0.9} = 1281\text{kN}$$

$$\phi N_{tb} = \left[0.5\phi 0.85A_n f_u; 0.5\phi A_g f_{yf}\right]_{min}$$

$$\phi N_{tb} = \left[0.5\times 0.9\times 0.85\times 8124\times 0.440; 0.5\times 0.9\times 10500\times 0.300\right]_{min}$$

$$\phi N_{tb} = \left[1367; 1417\right]_{min} = 1367\text{kN}$$

$\phi N_{tb} \geq N^*_{tb}$ ∴ OK!

9. Design of welds between column flange and bottom flange plate

$$\phi N_{wbfp} = 2\phi_w 0.6 f_{uw} \frac{t_{wbfp}}{\sqrt{2}} [b_{bfp}; b_{fc}]_{min}$$

$$\phi N_{wbfp} = 2 \times 0.8 \times 0.6 \times 480 \times \frac{15}{\sqrt{2}} [240; 230]_{min} = 1124\,kN$$

As fillet weld greater than 12mm use a CPBW.

10. Design of welds between column flange and bottom flange plate
    For consistency use a CPBW
11. Design of welds between column flange and web plate
    Design the welds to develop the web plate tension yield capacity

$$N^*_{wp} = \phi d_{wp} t_{wp} f_{ywp}$$

$$N^*_{wp} = 0.9 \times 450 \times 20 \times 250 \times 10^{-3} = 2025\,kN$$

$$\phi N_{wwp} = 2\phi_w 0.6 f_{uw} \frac{t_{wwp}}{\sqrt{2}} d_{wp}$$

$$\phi N_{wwp} = 2 \times 0.8 \times 0.6 \times 480 \times \frac{14}{\sqrt{2}} \times 450 = 2052\,kN$$

12. Determine area of tension/compression column continuity stiffeners required
    Area to be equivalent to bottom flange plate area

$$A_{spair} \geq (b_{bfp} t_{bfp} - t_{wc} t_{bfp}) \left(\frac{f_{ybfp}}{f_{ys}}\right)$$

$$A_{spair} \geq (240 \times 20 - 15.5 \times 20) \left(\frac{250}{250}\right) = 4490\,mm^2$$

$$\frac{0.9 b_{bfp} - t_{wc}}{2} \leq b_s \leq \frac{b_{fc} - t_{wc}}{2}$$

$$\frac{0.9 \times 240 - 15.5}{2} \leq b_s \leq \frac{230 - 15.5}{2}$$

$$100 \leq b_s \leq\approx 110$$

Use 2 110 x 20 flats for stiffeners

13. Welds between stiffeners and column flange adjacent to incoming beam
    Weld to be design for tension yield capacity

$$N^*_{ws} = \phi b_s t_s f_{ys}$$

$$N^*_{ws} = 0.9 \times 110 \times 20 \times 250 \times 10^{-3} = 495\,kN$$

Use 14mm leg length, category SP welds, E48 filler material

$$\phi N_{wscf} = 2\phi_w 0.6 f_{uw} \frac{t_{wscf}}{\sqrt{2}} b_s$$

$$\phi N_{wscf} = 2 \times 0.8 \times 0.6 \times 480 \times \frac{14}{\sqrt{2}} \times 110 = 502\,kN$$

14. Welds between stiffeners and column web
    Weld to be design for tension yield capacity

Use 5mm leg length, category SP welds, E48 filler material

$$\phi N_{wscw} = 2\phi_w 0.6 f_{uw} \frac{t_{wscf}}{\sqrt{2}} d_{1c}$$

$$\phi N_{wscw} = 2 \times 0.8 \times 0.6 \times 480 \times \frac{5}{\sqrt{2}} \times 573 = 934\,kN$$

15. Overstrength moment

$$\phi M^o_{SHJ} = \frac{\phi M_{SHJ}}{\phi} \phi_{oms}$$

$$\phi M^{o}_{SHJ} = \frac{453}{0.9} \times 1.4 = 705\,\text{kNm}$$

16. Design shear action on panel zone

$$V^{*}_{PSHJ} = \left(\frac{\phi M^{o}_{SHJ}}{d + t_{bfp}}\right)_L + \left(\frac{\phi M^{o}_{SHJ}}{d + t_{bfp}}\right)_R - V_{COL}$$

Approximately

$$V_{COL} = \frac{(\phi M_{SHJ})_L + (\phi M_{SHJ})_R}{h_c}$$

$$V_{COL} = \frac{0.5 \times 453}{3.5} = 64.7\,\text{kN}$$

$$V^{*}_{PSHJ} = \frac{705}{0.528 + 0.020} - 64.7 = 1222\,\text{kNm}$$

17. Design shear capacity of panel zone
No doubler plate

$$\phi V_c = 0.6\phi f_{ywc} d_c t_{wc} \left[1 + \frac{3 b_c t_{fc}^2}{d d_c t_{wc}}\right]$$

$$\phi V_c = 0.6 \times 0.9 \times 275 \times 629 \times 15.5 \left[1 + \frac{3 \times 230 \times 27.9^2}{528 \times 629 \times 15.5}\right] \times 10^{-3}$$

$$\phi V_c = 0.6 \times 0.9 \times 275 \times 629 \times 15.5\,[1.10] \times 10^{-3} = 1599\,\text{kN}$$

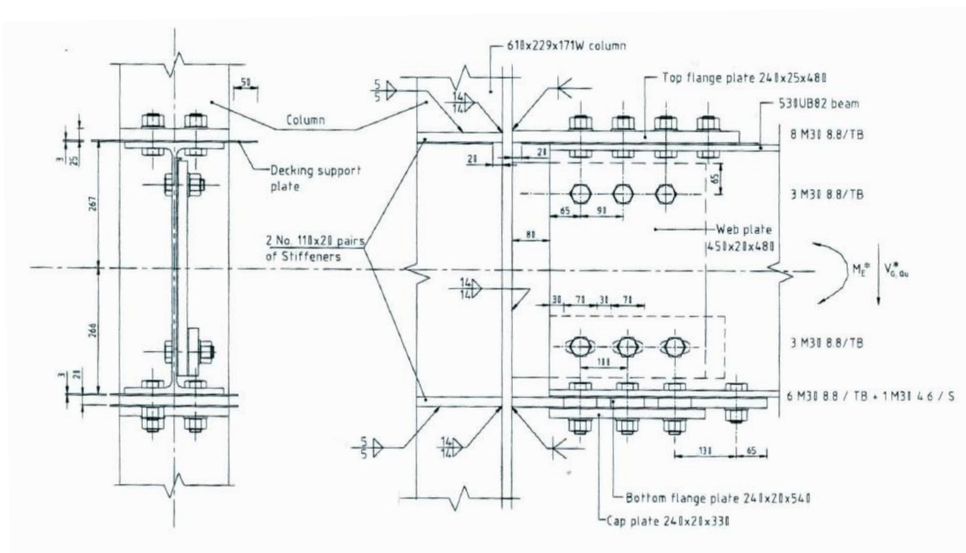

Figure 5: Design Example

References

Clifton C., Semi-Rigid Joints for Moment-Resisting Steel Framed Seismic-Resisting Systems, PhD Thesis, Department of Civil and Environmental Engineering, University of Auckland, 2005 (Also available as a HERA document)

Clifton, C., Sliding Hinge Joint: Revisions to the Design Procedure, NZ Heavy Research Engineering Assn (Inc), Manukau City, 2007

Gledhill, S., Sidwell G., Bell D., The Damage Avoidance Design of Tall Steel Frame Buildings – Fairlie Terrace Student Accommodation Project, Victoria University of Wellington, 2008 NZSEE Conference, 2008

MacRae, G., Mackinven, H., Clifton, C., Pampanin, S., Walpole W., Butterworth J., Tests of Sliding Hinge Joints for Steel Moment Frames, Pacific Structural Steel Conference 2007, NZ Heavy Research Assn (Inc), Manukau, 2007

MacRae, G., Clifton, C., Mackinven, H., Mago, N., Butterworth J., Pampanin, S., The Sliding Hinge Joint Moment Connection, Bulletin of the New Zealand Society for Earthquake Engineering, Intended publication 2009

SNZ, Steel Structures Standard (Incorporating Amendments 1 and 2), NZS 3404:1997, Standards New Zealand, Wellington, 2007

Appendix

The following is the latest amendments to the Sliding Hinge Joint design described in (Clifton, 2005) and has been put together by Charles Clifton, previously of NZ Heavy Engineering Research Association.

Sliding Hinge Joint: Revisions to the Design Procedure

Revision No 2
Date of Revision: 29 August 2007.

The Sliding Hinge Joint (SHJ) is a new semi-rigid joint system developed for moment-resisting steel frames. It has the ability to remain rigid under in-service conditions or ultimate limit state wind loading, and to rotate under severe earthquake loading above a predetermined design level, returning to the rigid state when the severe shaking stops.

The joint is designed and detailed so that at the end of the severe earthquake the loss of strength and stiffness is minimal and known and so the need for repair is significantly reduced or eliminated.

A full design procedure and detailing requirements for the joint is given in HERA Report R4-134, Semi-Rigid Joints for Moment-Resisting Steel Framed Seismic-Resisting Systems. This was published in June 2005.

Since then there has been further research undertaken on the joint at the University of Canterbury and it has been used on at least 5 multi-storey buildings. From these research and design applications there have been some changes recommended to the 2005 procedures. These have been written up in a number of different places but no one source presents all the design changes.

The first edition was published in HERA News August 2007. This is the first revision which contains changes following review of the joints for application to the high shear application of a coupling beam between CBF frames which has shown changes required to some of the web plate design provisions.

These changes are as follows:

1. The original concept was based on the use of 3mm thick brass shims in the sliding surfaces. Testing at the University of Canterbury by McKinven et al has shown that these can be replaced by 3mm thick steel shims with the same design capacity achieved. See details in the paper *Tests of Sliding Hinge Joints for Steel Moment Frames* by MacRae, MacKinven, Clifton, Pampanin, Walpole and Butterworth presented at the 2007 PSSC

2. The thickness of the bottom flange cap plate and the web cap plate are given in R4-134 as equations 5.27 and 5.34 respectively. These are incorrect in that report and should read:
   t_{bcp} = max(t_{bfp}; 16mm) (repl to 5.27)
   t_{wcp} = max(t_{wp}; 16mm) (repl to 5.34)
   The requirement from the tests undertaken by Clifton was that the cap plate is not thinner than the flange or web plate containing the slotted holes but with a minimum thickness of 16mm. The equations correctly state this.

3. The bolt design model as presented in R4-134 uses a quadratic interaction equation between moment and shear; this is equation 5.49 in that report. Testing at the University of Canterbury has shown this is slightly unconservative and that a linear interaction should be used. Thus the replacement equation becomes:
   $$\frac{M^*}{M_{rfn}} + \frac{V^*}{V_{fn}} \le 1.0 \qquad \text{(repl to 5.49)}$$
   See details in the paper *Sliding Hinge Joints and Subassemblies for Steel Moment Frames* by Clifton, MacRae, MacKinven, Pampanin and Butterworth presented at the 2007 NZSEE Conference.

4. In section 5.9.5.2 reference to Table 5.10 should read Table 5.12

5. The bolt design sliding shear capacity has been calculated using a bolt strength reduction factor of 0.8 and this is also used in the calculation of the joint overstrength moment. See section 5.9.2 for the former and equation 5.106 for the latter.
   However the nature of the joint and sliding bolt behaviour in this joint are such that the use of the strength reduction factor for steel in bending, shear and axial load of
   ϕ = 0.9 is more appropriate. This important change is required in several places, namely:

In section 5.9.2 the strength reduction factor of 0.9 is used to go from nominal to design bolt sliding shear capacity. This will increase the design moment capacity of the joint given by equations 5.56 and 5.57 by 0.9/0.8 = 13% which is realistic given the test results from Clifton and then MacKinven et al. It also means that the design sliding shear bolt capacities currently given in Table 5.12 of the report, which are correct even with the change to the bolt design model given in 3 above for the reasons outlined in the June issue of HERA News, will need to be increased by the factor 0.9/0.8.

In section 5.9.17 when calculating the joint overstrength moment from equation 5.106, ϕ = 0.9 is used in the denominator instead of ϕ = 0.8. The net result will be a reduction in the currently very large joint overstrength moment calculated by equation 5.106 which is also consistent with the test results from Clifton and MacKinven et al.

In sections 5.9.6.2 and 5.9.8.7, ϕ = 0.9 is used in the denominator instead of ϕ = 0.8.
The overstrength factors calculated in R4-134 are not altered.

In equations 5.60, 5.79, 5.86, 5.92 the value of 1.15 in the numerator becomes 1.0. This is explained in the changed definition for this factor given after equation 5.60 which would now note there is no difference in ϕ between bolt and plate.

The revised bolt sliding shear capacities are given by the table below:

Bolt Designation	Plate Thickness (mm)	ϕV_{fss} kN	$\phi V_{fss,\ bs}$ kN
M16	10	28	34
M16	12	27	33
M16	16	24	29
M20	12	47	57
M20	16	43	52
M20	20	40	48
M24	12	74	88
M24	16	68	82
M24	20	63	76
M30	16	18	141
M30	20	110	133
M30	25	102	123
M36	16	186	221
M36	20	175	209
M36	25	162	196
M36	32	148	179

6. For the bolts into the beam flanges, there is an issue with the minimum beam flange width which can be used for each bolt diameter. This is currently given in section 5.8.4 and is based on the edge distance to the flange edge being at least two times the bolt diameter. However, this limits the minimum beam flange width that can be used for an M30 bolt to 270mm, which is unnecessarily severe. NZS 3404 requires a minimum edge distance of 1.5d_f for bolts in this application and this can be used to develop both minimum edge distances and preferred edge distances from which minimum beam flange widths for a given bolt diameter can be established. These are given in the table below and the minimum dimensions allow larger bolt diameters to be used with the smaller beam sizes. They are based on the flange gauges, S_{gf}, of 70 for M16, 90 for M20 and M24, 120 for M30 and 140 for M36, which are required for constructability. However note the restriction on minimum flange width when using M30 bolts, even when the minimum edge distances are applied:

Bolt Designation	Edge Distance to Flange Edge		Minimum Flange Width	
	$a_{et, preferred}$ mm	$a_{et, minimum}$ mm	$b_{f,min, preferred\ aet}$ mm	$b_{f,min, minimum\ aet}$ mm
M16	35	25	140	120
M20	50	30	190	150
M24	50	40	190	170
M30	65	45	250	210
M36	65	55	270	250

7. When calculating the design vertical shear capacity of the web plate the current provisions are based on the clear depth of web plate given by ($d_{wp} - d_{wcp}$). This was done to avoid shear/axial load interaction in the bottom region of the plate covered by the web cap plate, so that all that region of plate could participate in the sliding shear action of the web bottom bolts without being affected by the vertical shear action. However, that is very conservative for two reasons. The first is that there is limited interation between vertical shear and transverse axial load and the latter is only reduced when the former is significant. This is already taken into account in the equation by limiting the maximum shear to 60% of the peak design capacity for shear alone. The second is the shear stress distribution is parabolic and therefore is low in the bottom quadrant of the web plate where the sliding action of the web bolts is concentrated. Thus the proposal is to allow the full depth of web plate to be used in calculating the design vertical shear capacity, thus giving:
$$\phi V_{vn,wp} = 0.27 d_{wp} f_{y,wp} t_{wp} \alpha_v \quad \text{(repl to 5.72)}$$

8. When calculating the design moment capacity of the web plate this is currently based on the effective depth of plate for resisting bending moment being given by
($d_{wp} - d_{wcp}$) and the moment capacity being based on the in-plane plastic moment of this effective depth of plate. This has been chosen to avoid interaction between bending and axial action from bolt sliding in the region of the web plate under the web cap plate.
However, in practice the whole depth of plate will resist the moment and the above is conservative, especially when the ratio of d_{wcp}/d_{wp} becomes greater than around 0.3. Given that there is already a net tension yield check on the portion of web plate which resists the sliding shear, given by section 5.9.8.6, if the design moment capacity of the web plate for vertical shear resistance is based on the full depth of plate in elastic bending, then there will be no interation in the joint sliding stage. If either of these criteria is satisfied then the web plate has adequate moment capacity. This requires section 5.9.8.4 equation 5.77 to be replaced by:
$$\phi M_{wp} = \max\left(0.225 t_{wp}(d_{wp} - d_{wcp})^2 f_{y,wp}; 0.15 t_{wp} d_{wp}^{\,2} f_{y,wp}\right) \quad \text{(repl to 5.77)}$$

9. Equation 5.30 gives the length of the web plate. This is taken as the maximum of two sets of dimensional variables. The first of these contains the dimension f_{SHJ} which is the clear distance from the column face to the end of the beam.
This term should also be in the second set of variables, thus eqn 5.30 should read:

$$L_{wp} \geq \max\left[f_{SHJ} + 2a_{ep} + (n_{wtb} - 1)s_{g,w}; \left(f_{SHJ} + a'_{ep} + n_{wbb}(s_{sh} + a'_{ep})\right)\right] \quad \text{(repl to 5.30)}$$