

Section H

BEAM—COLUMN JOINTS

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This paper is the result of deliberations of the Society's
Study Group for the Seismic Design of STEEL STRUCTURES

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2. INTRODUCTION

The principal requirements for connections are ease of fabrication and erection, sufficient strength, stiffness, adequate rotation capacity and the ability to accept adjacent large repeated reversing plastic deformations where required.

Beam-column joints can be of many types and are dependent on the type of framing chosen.

This paper discusses general principles and the design of components rather than the detailed rules for any particular type of joint.

Capacity design methods will be required for the design of ductile frames. That is energy-dissipating elements should be chosen, and suitably proportioned and detailed, while all other structural elements (including connections) should be provided with sufficient reserve strength

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to ensure that the chosen energy dissipating mechanisms would be maintained throughout the deformations that may occur.

The design rules for the energy-dissipating elements and their adjacent connections should be derived from the results of simulated and real earthquakes where random cyclic loading occurs. This should verify the amount of energy dissipation available, the stiffness, ductility and the significance of any strength degradation.

The strength of non-energy-dissipating elements should generally be adequately predicted from static tests under monotonic loading. These elements will merely be required to accept the actions from the energy-dissipating elements. These actions should be derived from ideal strengths and specified material sizes and strengths increased by an overstrength factor.

For multi-storey moment-resisting steel frames it may be more difficult to meet stiffness requirements, e.g. Cl. 3.8.3 of NZS 4203:1984 (1), than strength requirements, e.g. Cl. 3.4.2. In this case the designer will be trying to achieve rigid joints. Methods of increasing joint rigidity are discussed in Section 6. This will probably also mean that the joint will be designed so that large plastic deformations will not take place in the joint region. The rules in this paper are designed to achieve the above two objectives.

For low-rise structures it may be acceptable to allow large plastic deformations to occur in the joint components, and research by Krawinkler and Popov (2) and McAteer and Walpole (3) has indicated that energy may be absorbed by shear yielding inside the panel zone. It should be noted that any joint deformation will significantly increase the flexibility of the frame.

Further research is required before guidance can be given on this approach, but meanwhile it should be accepted with special study of a particular application for structures less than 10 metres in height.

Although high ductility may be required at certain locations in a steel frame, at other locations in the same frame only limited or low ductility may be required.

3. APPLICATION

The design recommendations given in this paper are intended to apply to beam-column joints made between the members of rectangular multi-storey steel frames, which have been designed in accordance with NZS 4203(1) or with SDPP(4).

No allowance is made in this paper for any composite action with concrete slabs or casing.

3.1 Category One : Ductile Frames.

Members designed to form plastic hinges which are expected to suffer several cycles of plastic deformation of magnitude greater than that corresponding to a member displacement ductility ratio of four would come into this category.

It was considered that it would generally be conservative to assume that structures designed using a structural type factor S factor of less than two (1); or a ductility capability factor μ of greater than two(4); would come into category one.

3.2 Category Two : Limited Ductility Frames

Members designed to form plastic hinges which are expected to suffer either a few cycles of plastic deformation of large magnitude or several cycles of plastic deformation of magnitude less than that corresponding to a member displacement ductility ratio of four would come into this category.

It was expected that structures designed with an S factor of between two and six or with a ductility capability factor μ between one and two would come into category two.

3.3 Category Three : Elastically Responding Frames

Structures designed using a structural type factor S of six or using a ductility capability factor μ of one would come into this category.

It should be appreciated that there is not a particularly direct relationship between the S factor in NZS 4203 and the ductility demand required to withstand particular earthquakes. There is likely to be a more direct relationship between the μ factor in SDPP and ductility demand, but the characteristics of large New Zealand earthquakes are not well defined yet.

4. MATERIAL

This topic is covered more fully by McKay (5) however some problems of material selection and specification which are important in beam-column joints are mentioned here. Most specifications are written so that satisfactory performance will be obtained with non-seismic loading. Test samples are usually specified to be

taken parallel to the direction of rolling. This direction often corresponds to the maximum stress in a member. However in joints there may be significant stresses in other directions, for example in the through-thickness direction. In this case tensile tests should also be made on samples cut parallel to the other directions of stress.

In addition the Charpy impact tests should be carried out on specimens cut so that the energy absorption is not less than 27J, at the minimum expected service or erection temperature, for notches cut perpendicular to the expected direction of tensile stress.

Laminations may be formed during the rolling process and also lamellar tearing may occur near the surface of the steel after welding. Often these major defects will be revealed during fabrication, but inspection procedures must ensure that any defects present are acceptable. Clause 12.7 of NZS 3404:1977 (6) gives some guidance here.

Steel-making processes have been improved in recent years partly because of the requirements of offshore steel platforms. Current structural steel specifications will not ensure that the better steel properties available are provided. Some specifications do not require impact tests on material above a certain thickness, even for samples cut in the direction of rolling.

More effort is required to transfer existing knowledge between the fields of structural engineering, metallurgy and fracture mechanics.

5. DESIGN OF COMPONENTS

5.1 General Principles

Structural steel is generally a very ductile material, but this ductility can be significantly reduced through poorly detailed connections. Many connections rely on the ductility of the steel to redistribute the stress distribution and this practice has been proven over many years. However there are limits to the amount of strain which may be tolerated by structural steel before fracture.

Stress raisers such as sharp re-entrant corners, notches, etc, greatly increase the local strain and can lead to premature fracture. It is desirable that the components and welds or fasteners are arranged in such a way that a smooth stress flow may take place between them.

In situations where biaxial or triaxial stresses occur, the loading and factors specified for the strength method by NZS 4203 shall be used to ensure the equivalent uniaxial stress f_{eq} calculated using the von-Mises-Henky criterion is less than the specified yield stress, F_y , viz.

$$f_{eq} = \sqrt{\frac{1}{2} \left[(f_1 - f_2)^2 + (f_2 - f_3)^2 + (f_3 - f_1)^2 \right]}$$

for triaxial stresses

$$f_{eq} = \sqrt{f_1^2 - f_1 f_2 + f_2^2}$$

for biaxial stresses

$$f_{eq} < F_y$$

where f_1, f_2, f_3 are the principal stresses

Undrill (7) has shown how the connection plate between an I-beam and a double I-column gave better ductility when a curved transition plate was used and the ends of the connecting fillet welds ground to a taper. This delayed the formation of cracks in the beam flanges.

Driscoll and Beedle (8) have given an example of how premature fracture can occur where restraint creates a triaxial stress situation. They studied beam-column connections where the I beams were connected to the webs of the I columns. They suggested the various methods shown in Figure 1 to reduce the magnitude of the non-linear bending stress which occurs as the beam flange enters the I section, as shown in Figure 2(a). When the connection plate is the same thickness as the beam flange, fracture occurs near the tip of the column flange.

Premature local or overall buckling is undesirable, as with further cycles of loading the curvature of the buckle increases which may lead eventually to fracture, or loss of strength. Fracture may occur more quickly if stress raisers such as tack welds, stud welds, transverse fillet welds, threaded holes etc are present within the plastic zone.

5.2 Reduction of Section

Where the net area A_n is less than the gross area A_g then the average stress at the net area will reach the yield stress before the gross area. When the actual non-uniform stress distribution is considered then it will be realised that yielding can occur at the net section before the gross section, even when the net section equals the gross area in area because of stress raiser effects. Because of this effect, some codes require the net to be greater than the gross area for pin connections.

For non-seismic bolted connections most codes require that the connection will yield at the gross section before failure would occur by fracture at the net section. As the brittle failure is less desirable an additional small penalty factor of 0.85 is used. This rule occurs in Cl. 12.4.8.2 of NZS 3404:1977. The factor is further reduced when $A_n/A_g < 0.85$ by the Canadian standard (9) to allow for stress raiser effects. Limits are also placed on the maximum net area which may be assumed;

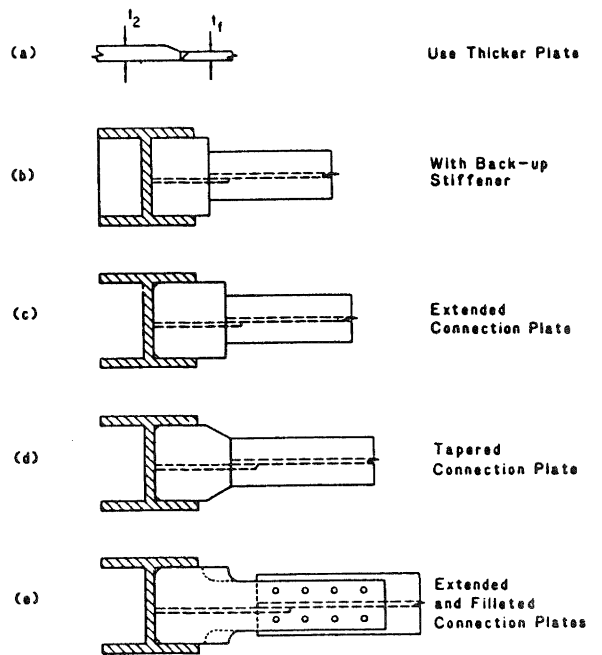


Figure 1.

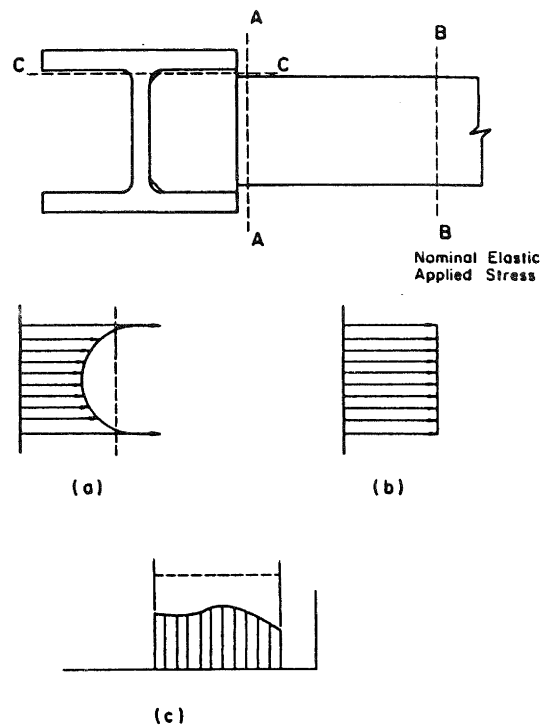


Figure 2

- (a) Longitudinal stresses on Section A-A
- (b) Longitudinal stresses on Section B-B
- (c) Shear stresses on Section C-C

viz.

- 0.85 A_g when $F_y/F_u \leq 0.75$
- 0.90 A_g when $0.75 < F_y/F_u \leq 0.85$
- 0.95 A_g when $0.85 < F_y/F_u$

where F_u is the ultimate stress.

Capacity design principles would require that NZS 3404 rule Cl. 12.4.8.2

$$0.85 F_u A_n > F_y A_g$$

be modified by the addition of an overstrength factor, ϕ_{os} where the connection is adjacent to an area designated to dissipate energy.

$$0.85 F_u A_n > \phi_{os} F_y A_g$$

If ϕ_{os} is taken as 1.35 for a tension member and $F_u/F_y = 1.6$ then the rule becomes

$$A_n \geq A_g$$

Preliminary tests by Nakane and Walpole (10) on bolted lap joints, tested under cyclic tensile loading, indicate that this rule leads to excellent ductility with fracture eventually occurring in the gross section of the member.

5.3 Bolted End-Plates

It is recommended that where the flanges are connected to the end plate by butt welds, the thickness of end plates, T_{ep} , be determined by the equation derived by Surtees and Mann (11) and modified by Whittaker and Walpole (12) to allow for the finite thickness of the beam web and flanges, for the splay provided by the welds, and for a variable depth, p , of yield line, as shown in Figure 3:

$$T_{ep} \geq \left[\frac{M_b}{F_{yp} d_f \left[\frac{2B_p}{c - T_b - 2w_f} + \frac{2p}{A - t_b - 2w_w} \right]} \right]^{1/2}$$

where

- A = the horizontal distance (gauge) between bolt holes
- B_p = the gross width of end-plate without reduction for bolt holes
- c = the vertical distance (pitch) between bolt holes
- d_f = the distance between the centres of the beam flanges
- F_{yp} = the specified yield stress of the end plate
- M_b = the overstrength moment provided by the beam
- p = $0.6 d_f$

- w_f = is the splay of the weld fillet on both sides of the flange
- w_w = is the splay of the weld fillet on both sides of the web
- T_b = the thickness of the beam flange
- t_b = the thickness of the beam web

This equation will also accurately predict the thickness of end-plate required when fillet welds are used. It has also been found (12,13) that adequate ductility can be obtained before failure by fracture of the fillet welds occurs, but the parameters which control the ductility that may be obtained with fillet welds are not well understood at present. It is recommended that a stiffer end-plate be used to reduce the high curvatures and strain that may occur at the interface between the fillet weld and the end-plate. The equation derived by Mann and Morris (14) using a T-stub analogy, as shown in Figure 4, which ignores the stiffening effect of the web of the beam, should be used when the flange is connected to the end-plate by fillet welds.

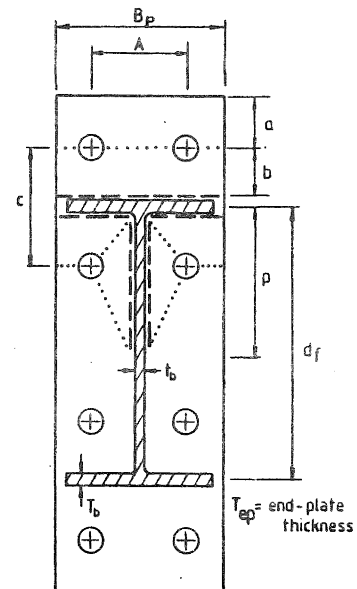
$$T_{ep} = \sqrt{\frac{M_b b}{d_f F_{yp} B_p}}$$

where

- b = the distance from the edge of the weld fillet to the centre of the bolt

5.4 Tee Stubs

The thickness of the tee-stub flange should comply with the Mann and Morris equation above. A recent trend in Japan



..... Hogging yield lines
 ----- Sagging yield lines

Figure 3

has been towards using cast tees with thick flanges. These reduce flange bending stresses and deformations and minimize bolt prying forces.

5.5 Bolts Forces

The strength of bolts, rivets and welds are given in Section G - Connections (15)

Where four bolts only are provided per flange in an end-plate connection complying with the equations above, then the bolts should be assumed to be equally loaded without allowance for prying. This procedure has been verified by testing (12,16).

$$P_f = \frac{M_b}{4 d_f}$$

Bolt forces have been found to increase less than 10% above the proof load and yet about 70% is available before the specified ultimate tensile load is reached. It is thought that compliance with the relevant bolt standards and will ensure an adequate reliability. The prestress of the bolts should not be exceeded under the more frequent code level earthquakes because of inclusion of an overstrength factor in M_b .

When eight bolts are provided per flange it is unlikely that bolts will be as evenly loaded. Test results are not available to give guidance to designers.

The following simplified formula for the additional bolt force, Q , due to prying, has been found to fit experimental results (17) for Tee-stubs attached by four bolts per flange, as shown in Figure 4:

$$Q = \left[\frac{3b^3}{8a} - \frac{T_b^3}{20} \right] \frac{M_b}{4d_f}$$

where
 a = the distance from the centre of the bolts to the top edge of the plate

Where bolts are subjected to tension and shear forces, these should satisfy the following interaction equation:

$$\left[\frac{P_f}{P_{df}} \right]^2 + \left[\frac{V_f}{V_{df}} \right]^2 < 1.0$$

where

- P_f = tensile force in the fastener
- P_{df} = dependable strength of a fastener in tension
- V_f = shear force in the fastener
- V_{df} = dependable strength of a fastener in shear

At present dependable values equal ideal values, which may be found from allowable stresses or strengths divided by 0.6 (6)

5.6 Local Flange Bending

5.6.1 Tensile flange loading

Graham et al(18) have for non-seismic connections shown that a flange may be left unstiffened under a tensile flange load, as shown in Figure 5, provided that:

$$T_c \geq 0.4 \sqrt{B_b T_b}$$

For seismic connections this should be modified to allow for overstrength in the beam flange, if this is chosen to yield:

$$T_c \geq 0.4 \phi_{os} \sqrt{B_b T_b F_{yb}/F_{yc}}$$

- where
- T_c = the thickness of the column flange
- B_b = the width of the beam flange
- F_{yb} = the specified yield stress of the beam
- F_{yc} = the specified yield stress of the column

5.6.2 Tensile bolt loading

Bolt forces may be resisted by local bending of the flange of a member, for example where the end-plate on a beam is bolted to a column flange. It will generally be conservative to check the adequacy of the flange thickness by means of the equation above. If it does not

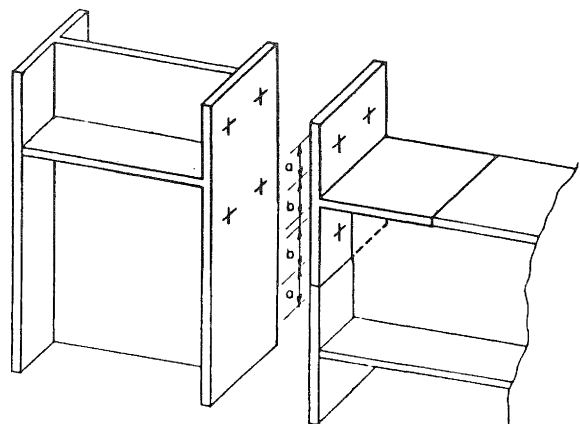


Figure 4

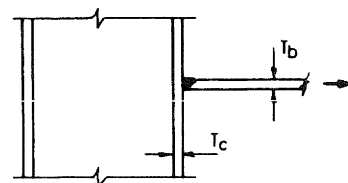


Figure 5

comply then the strength of the flange may be determined more accurately by equilibrium or yield line methods. The following strengths have been found by yield line methods ignoring any stresses in the flange caused by axial loads or overall member flexure. Although possibly theoretically unconservative, connections designed in this way have been found by test to behave very well (12,13,16).

The strength of the flange is not very sensitive, within reason, to the precise pattern of the yield lines. With steel flanges the thickness is not small compared to other dimensions and distinct yield lines cannot be found experimentally.

The flange tension force P_{ms} which can be applied through four bolts to a member with web stiffeners as shown in Figure 6(b) was calculated for the yield line pattern shown in Figure 7 by Packer and Morris (19) to be:

$$P_{ms} = F_{yc} T_c^2 \left[\left[\frac{1}{v} + \frac{1}{w} \right] (2m + 2n - D) + \frac{2v + 2w - D}{m} \right]$$

where

- w = $\sqrt{m(m + n - 0.5 D)}$
- v = the distance from the edge of the fillet to the centre of the bolt
- m = the distance from the edge of the fillet to the centre of the bolt
- n = the distance from the edge of the flange to the centre of the bolt
- D = the diameter of the bolt hole

For a member with doubler plates flange welded to the tips and flange reinforcing plates as shown in Figure 6(c) the tension force applied through four bolts was calculated by Whittaker and Walpole (12) for the mode I and II yield line pattern shown in Figure 8 to be:

Mode I:

$$P_{mp} = F_{yc} T_c^2 \left[\frac{2(m+n) - D}{g} + \frac{2g+c-D}{m} + \frac{(1+r)(2g+c) - 2D}{2n} \right] + F_{yr} T_r^2 \left[\frac{m+n-D}{g} + \frac{2g+c-2D}{2m} + \frac{2g+c-2D}{2n} \right]$$

Mode II:

$$P_{mq} = F_{yc} T_c^2 \left[\frac{(1+r)(g+h) - D}{n} + \frac{2(g+h) - D}{m} + \frac{2(m+n) - D}{g} + \frac{2(m+n) - D}{h} \right] + F_{yr} T_r^2 \left[\frac{2(m+n) - D}{h} + \frac{m+n-D}{g} + \frac{g+h-D}{m} + \frac{g+h-D}{n} \right]$$

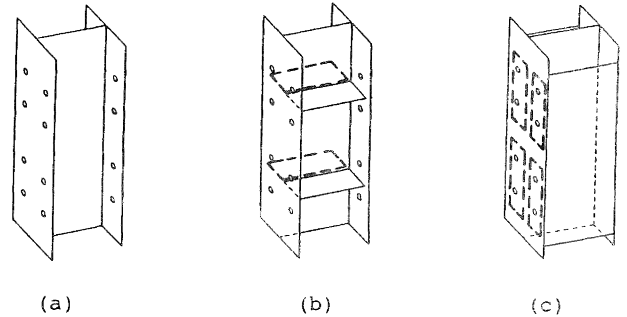


Figure 6

- (a) an unstiffened column
- (b) Web stiffeners, both sides
- (c) Doubler plates welded to flange tips both sides

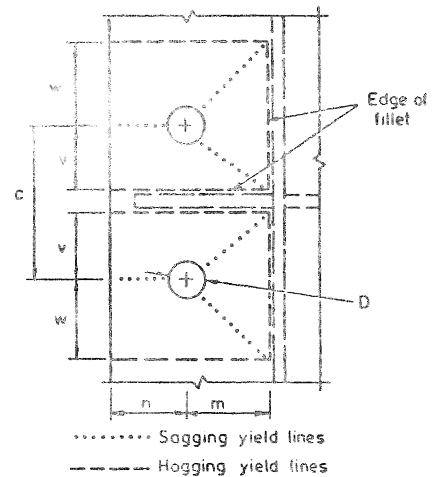


Figure 7

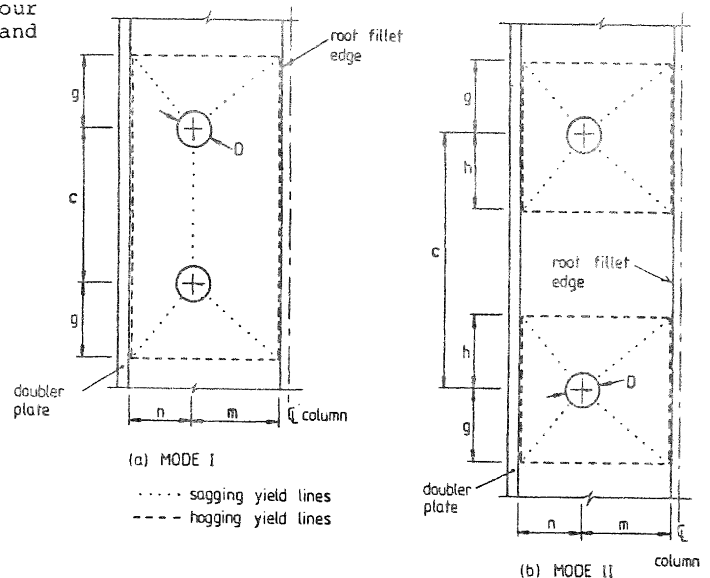


Figure 8

where

$$r = \frac{F_{yd} T_d^2}{F_{yc} T_c^2}$$

$$g = \left[\frac{((2+s)(m+n) - (1+s)D)mn}{(2+s)n + m(1+r) + ms} \right]^{1/2}$$

$$h = \left[\frac{(2(1+s)(m+n) - (1+s)D)mn}{(2+s)n + m(1+r) + ms} \right]^{1/2}$$

$$s = \frac{T_r^2}{T_c^2}$$

F_{Yr} = the specified yield stress of the column flange reinforcing plate

F_{Yd} = the specified yield stress of the doubler plates

T_r = the thickness of the flange reinforcing plate

For a column stiffened with an external flange reinforcing plate and with web stiffeners as shown in Figure 9b, Plugge and Walpole (16) found the tension flange load P_{mt} to cause the yield line pattern shown in Figure 10 to be:

$$P_{mt} = F_{yc} T_c^2 \left[(n+m) \left[\frac{1}{w} + \frac{2}{v} + \frac{1}{s} \right] + \frac{1}{m} (s+2v+w) + r B_f \left[\frac{1}{2w} + \frac{1}{v} + \frac{1}{2s} \right] \right]$$

where

$$r = \frac{F_{Yr} T_r^2}{F_{Yc} T_c^2} \quad \text{but } r \leq 1$$

$$w = \sqrt{m(n+m+rB_f/2)} \quad \text{but } w \leq d_f - v$$

For a column stiffened with internal flange reinforcing plate and with web stiffeners as shown in Figure 9(a), Plugge and Walpole found the tension flange load P_{mv} to cause the yield line pattern shown in Figure 11 to be:

$$P_{mv} = F_{yc} T_c^2 \left[\frac{2(m+n)}{g} + 2 \frac{n}{v} + \frac{c+2g}{m} + \frac{c+2v}{2n} + 2r(m+n) \left[\frac{1}{v} + \frac{1}{g} \right] + \frac{r(v+g)}{m} \right]$$

$$\text{where } g = \sqrt{2m(m+n)(r+1)/(r+2)}$$

The reinforcing plates must be sufficiently long for k to satisfy:

$$k > \sqrt{m} (\sqrt{2r+2} + \sqrt{r}) \sqrt{(m+n)/(r+2)}$$

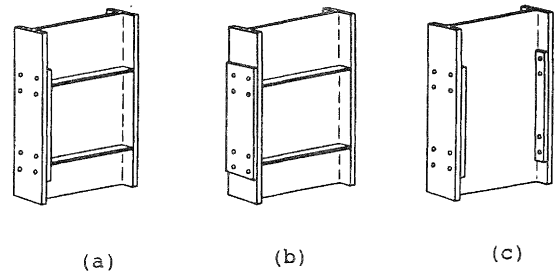


Figure 9

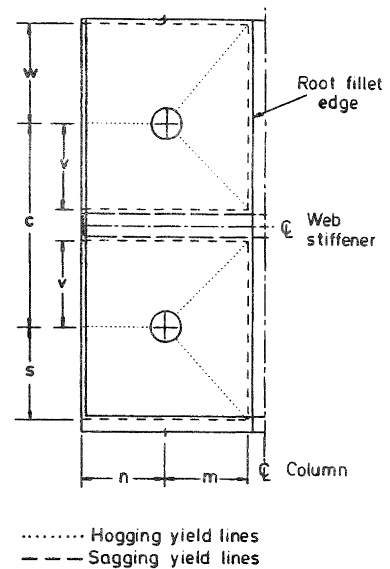


Figure 10

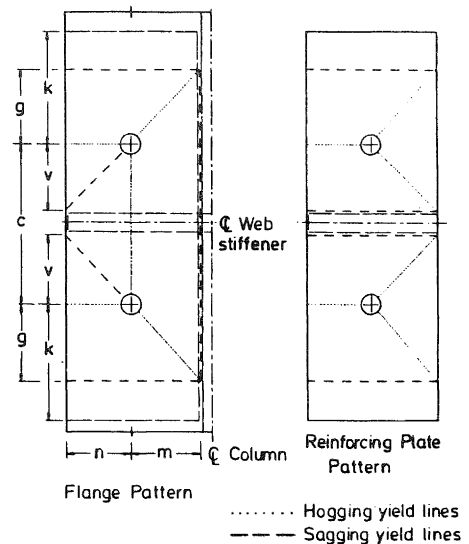


Figure 11

For a column without web stiffeners but with a flange reinforcing plate as shown in Figures 9(c) and 12, Whittaker and Walpole (12) calculated the tension flange force P_{mu} to cause the yield line pattern shown in Figure 13 to be:

$$P_{mu} = \left[\frac{F_{yc} T_c^2}{2} + \frac{F_{yr} T_r^2}{2} \right] \left[\frac{4(m+n)^2}{gm} - \frac{2D(m+n)}{gm} + \frac{c+4g-2D}{m} \right] + \frac{c F_{yr} T_r^2}{2m}$$

where

$$g = \sqrt{(m+n)^2 - 0.5D(m+n)}$$

Two beam-column specimens which satisfied the above equation were tested by Plugge and Walpole(16) and found to provide adequate ductility.

5.7 Unstiffened Webs

Graham et al(18) have shown for non-seismic connections that a web stiffener is not required to prevent web yielding under a compressive flange load provided that the column web thickness, t_c , satisfies:

$$t_c \geq \frac{B_b T_b}{T_b + 5k}$$

where

k = the thickness of the column flange plus the web root fillet

Witteveen et al(20) has modified this equation to allow for the presence of end plates and weld splay, as shown in Figure 14.

$$t_c \geq \frac{B_b T_b}{T_b + 5k + 2T_{ep} + 2w_f}$$

For seismic connections this should be modified to

$$t_c \geq \frac{\phi_{os} B_b T_b}{T_b + 5k + 2T_{ep} + 2w_f} \cdot \frac{F_{yb}}{F_{yc}}$$

AS 1250:1981(21) Cl.5.13.2.1 provides that a web stiffener is not required to prevent web buckling provided that

$$0.6 B_b T_b F_{yb} > F_{ac} t_c B$$

i.e. with allowance for overstrength

$$t_c \geq 0.6 \frac{B_b T_b F_{yb} \phi_{os}}{B F_{ac}}$$

where

F_{ac} = is found from Table 6.1.1 of AS 1250 using a slenderness of $d_1 \sqrt{3}/t_c$

- d_1 = distance between the inside edges of the flanges of the member subjected to the compression load
- B = is the length obtained at the neutral axis by a 45° spread from the edge of stiff bearing. (i.e. the edge of the weld splay, as shown in Figure 15).

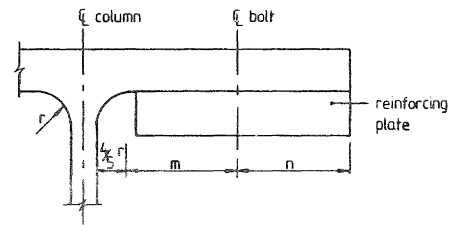


Figure 12.

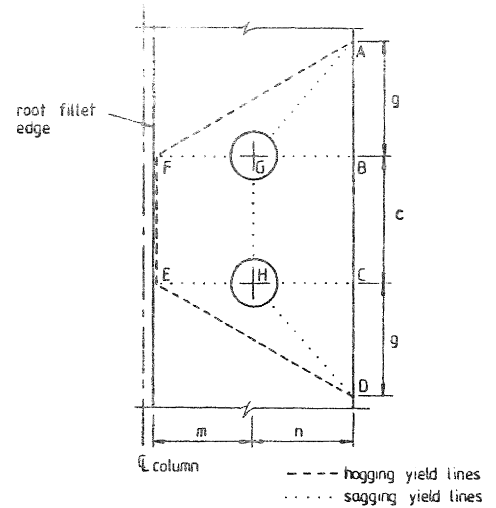


Figure 13.

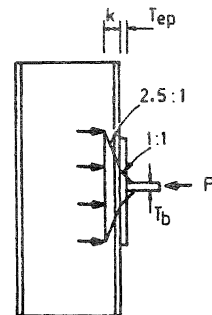


Figure 14.

If a doubler plate is used reinforce the web it should be assumed to act independently in this clause unless plug welded to the web.

5.8 Web Stiffeners

To prevent web yielding it has been suggested by Graham et al(18) that the area of stiffener A_{st} provided need only be sufficient to carry the extra load which cannot be carried by the unstiffened web.

$$A_{st} \geq \phi_{os} B T_b \frac{F_{yb}}{F_{ys}} - t_c (T_b + 5k + 2T_{ep} + 2w_f) \frac{F_{yc}}{F_{ys}}$$

To prevent web buckling a cruciform cross-section area A_{cr} comprising the web stiffeners and an effective width of web of $20 t_c$ on each side as shown in Figure 16 should be checked for stability under the total compressive flange load, as recommended by AS 1250.

$$F_{ac} A_{cr} \geq 0.6 \phi_{os} B T_b F_{yb}$$

where

A_{cr} = the area of the cruciform cross-section

F_{yb} = the specified yield stress of the beam

F_{ys} = the specified yield stress for the horizontal stiffener

F_{ac} = found from Table 6.1.1 AS 1250 :1981 using an effective length l and a radius of gyration r as follows:

$$l = 0.7 d_1$$

d_1 = the distance between the inside edges of the flanges

$$r = \sqrt{I/A_{cr}}$$

I = the second moment of area of the cruciform cross-section about the centre-line of the web.

5.9 Shear in Panel Zone

Axial load and shear forces applied to the panel zone within the beam-column joint regions, see Figure 17, should comply with the following equation from Cl. 12.4.9 of NZS 3404:1977 (6). The equation may be derived from the von Mises criterion of yielding.

$$\left[\frac{P}{A_c F_{yc}} \right]^2 + \left[\frac{V}{0.55 A_s F_{yw}} \right]^2 < 1.0$$

where

$$V = 2M_b / d_f - V_c$$

V_c = the shear force in the column

It will often be necessary to reinforce the web in order to comply with the above equation. Where doubler plates, which may be in contact with the web or welded to the flange tips, are used, the effective area of the web A_w used the equation above should be the sum of the areas of the web and doubler plate(s).

Where a diagonal stiffener is used to reinforce the web in addition to horizontal stiffeners then V shall be reduced by the shear strength of the diagonal stiffener

$$V = 2M_b / d_f - V_c - F_{ys} A_{st} \cos \beta$$

where

A_{st} = the area of the diagonal stiffener

F_{ys} = the specified yield stress for the diagonal stiffener

β = the angle made by the diagonal stiffener with a horizontal line

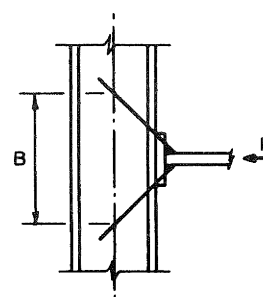


Figure 15.

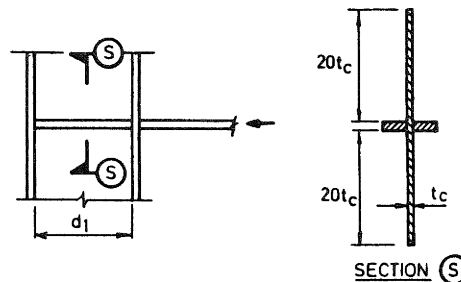


Figure 16.

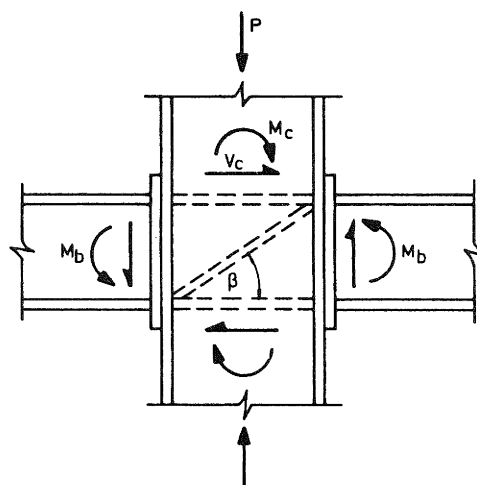


Figure 17.

6. STIFFNESS OF JOINTS

Further research is required before any useful guidance can be given regarding even the initial elastic stiffness of joints. The stiffness under a major earthquake can only be estimated at present. Most testing measures only the deflections at the ends of members and it is difficult to derive the joint stiffness from these results. It is thought that the flexibility of "rigid" joints will increase frame deformations by about 20%.

In some circumstances semi-rigid joints may be used and these are design methods based on this type of joint.

The limited guidance given by NZS 4203:1976 has now been removed with reference to the material code in NZS 4203:1984; but no guidance is given in NZS 3404:1974, the current standard.

The choice of joint components influences the stiffness of the joint. Pretensioning bolts will increase connection stiffness by decreasing flange and plate distortion through clamping action and by reducing bolt elongation and connection slip. Web stiffeners, between flanges in line with other flange loads, reduce flange and web deformation. Increasing the thickness of flanges, webs, doubler plates and end-plates, and increasing the depth of columns and beams will increase the stiffness of joints.

7. FURTHER RESEARCH

The recommendations given here are based on a limited amount of experimental testing. Further testing of other types of joints is required, particularly where these are significantly different from those tested so far.

With the acceptance of design to differing levels of ductility, different categories of joint detailing may be used. There may be some need for testing joints at moderate ductilities.

Tests should be made with snug-tight high-strength bolts to see whether it is possible to accept these in some situations with cyclic loading.

At present it is difficult to predict when fracture will occur for a particular connection arrangement. More research is required so the science of fracture mechanics can be applied to the low-cycle fatigue problem.

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9. NOTATION
- A = the horizontal distance (gauge) between bolt holes
- A_c = the gross cross-sectional area of a column
- A_{cr} = the area of the stiffener plus the effective area of web
- A_g = the gross area of a plate or section
- A_n = the net area of a plate or section
- A_s = the effective area of a section for shear
- a = the distance from the centre of the bolts to the top edge of the end plate
- B = the length at the neutral axis assumed to carry bearing
- B_p = the gross width of an end plate, without reduction for bolt holes
- B_b = the width of the flange of a beam
- b = the distance from the edge of the weld fillet to the centre of the bolt
- c = the vertical distance (pitch) between bolt holes
- D = the diameter of a bolt hole
- d_f = the distance between the centres of the beam flanges
- d_l = the distance between the inside edges of the flanges
- F_{ac} = the allowable stress in axial compression
- F_u = the ultimate stress
- F_y = the specified yield stress
- F_{yb} = the specified yield stress of the beam
- F_{yc} = the specified yield stress of the column
- F_{yd} = the specified yield stress of the doubler plate
- F_{yp} = the specified yield stress of the end plate
- F_{yr} = the specified yield stress of the flange reinforcing plate
- F_{ys} = the specified yield stress of the stiffener
- F_{yw} = the specified yield stress of the web
- f_{eq} = the equivalent uniaxial stress
- f_1, f_2, f_3 = the principal stresses
- g, h = the distances from the centre of the bolt to the edge of yielding
- I = the second moment of area
- k = the thickness of the column flange plus the web root fillet
- l = the effective length
- M_b = the yield moment of the beam, allowing for overstrength phenomenon
- m = the distance from the edge of the fillet to the centre of the bolt
- n = the distance from the edge of the flange to the centre of the bolt
- P = the axial force in the column
- P_{df} = the dependable strength of a fastener in tension
- P_f = the tensile force in a fastener
- $P_{mp}, P_{mq}, P_{mt}, P_{mu}, P_{mv}$ = the forces in the flange of a beam to cause certain yield line patterns
- P_{tb} = the compression flange force in the beam
- P_y = the squash load of the column, i.e. the gross cross-sectional area multiplied by the specified yield stress of the column
- p = $0.6 d_f$
- Q = the additional force in a bolt due to prying
- r = $F_{yd} T_d^2 / F_{yc} T_c^2$ or $F_{yr} T_r^2 / F_{yc} T_c^2$
- S = the structural type factor from NZS 4203
- s = T_r^2 / T_c^2
- T_b = the thickness of the beam flange
- T_c = the thickness of the column flange

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T_{ep}	= the thickness of the end plate	V_f	= the shear force on a fastener
T_r	= the thickness of the flange reinforcing plate	v	= the distance from the edge of the fillet to the centre of the bolt
t_b	= the thickness of the web of the beam	w	= the distance from the outside yield line to the centre of the bolt
t_c	= the thickness of the web of the column	w_f	= the splay of the weld fillet on both sides of the flange
V	= the shear force carried by the panel zone	w_w	= the splay of the weld fillet on both sides of the web
V_c	= the shear force in the column	ϕ_{os}	= the overstrength factor
V_{df}	= the dependable strength of a fastener in shear	μ	= the ductility capability factor from SDPP (4)