STEEL DESIGN & CONSTRUCTION BULLETIN

DCB

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All articles in this publication are written by G Charles Clifton, HERA Structural Engineer, unless otherwise noted.

Several articles in this issue have been subjected to detailed review and revision. The effort and input of the reviewers is greatly appreciated.

Introduction

This issue has three general themes, namely:

- new guidance available on a range of issues
- update on HERA semi-rigid connection research programme
- optimising the cost of multi-storey steel buildings in New Zealand.

The latter theme is included in the cost-effective structural steelwork feature for this issue. It summarises material produced by Clark Hyland, SSAS Manager, drawn from the many preliminary steel framed building solutions designed and costed by the SSAS within the last two years.

Also covered, first up in this issue, are three aspects revisited from the previous Issue No. 51.

Finally, note that you will have received a copy of the August Issue (Issue No. 51) along with this issue. Please replace the previous copy of Issue No. 51 with this new copy. The reason for this is that some equations in the original copy sent to you are missing, illustrating the occasional downside of modern technology!

The problems occur on pages 17 and 19 therein and involve some of the equations not being printed, despite their showing on the screen. For example, what should show:

> This check is using equation 50.7, *DCB* Issue No. 50.
>
> $$t_{fc} \approx 0.5 C_3 C_4 \left(0.9\, t_i + t_{fb}\right)\left(\frac{f_{yi}}{f_{y,cf}}\right) \quad = 22.7 \text{ mm}$$

has printed as:

> This check is using equation 50.7, *DCB* issue No. 50.
>
> = 22.7 mm

The problem has occurred in 5 places on page 17 and 3 places on page 19. The new copy of Issue No. 51 has (obviously) included these missing equations and also included the centreline locator arrows for hole punching which were omitted from the previous copy.

We regret this oversight and realise that replacing the copy of Issue No. 51 is the simplest way to rectify the situation for *DCB* readers.

In This Issue

Three Items From DCB Issue No. 51 Revisited

Extending Use of the Collapsed Wall Condition for Support of External Concrete Wall Panels in View of Australian Experience and Recommendations

Background

When a fire rated concrete external wall panel is supported off a steel frame in single storey portal frame type buildings, then the designer has to determine if the steel column needs a fire rating, in order to ensure that the wall remains dependably upright during a severe fire. The reason for keeping the wall dependably upright during a severe fire relates to preventing fire spread across a boundary or to neighbouring buildings by excessive radiation.

The philosophy and design requirements for controlling the spread across the boundary are undergoing a major revision in the new *Draft Acceptable Solutions for Fire Safety* (reference [1] from *DCB* Issue No. 51), with a change of approach from the traditional "mirror image" method to one based on limiting radiation at critical points across the boundary. Details of this change are given on pages 3 and 4 of *DCB* Issue No. 51.

On pages 4 and 5 of that issue, the previously developed *collapsed wall condition* design criteria have been reviewed in light of the changing requirements for controlling fire spread across the boundary and recommendations for applying this procedure under the proposed new fire separation provisions have been given.

In terms of whether the supporting steel columns are likely to need passive fire protection or not, the outcome of applying the collapsed wall condition recommendations from *DCB* Issue No. 51 would have been as follows:

- For unsprinklered fire hazard category (FHC) 1 and 2 building columns - no passive protection is needed.
- For unsprinklered FHC 3 and FHC 4 building columns - passive protection will typically be needed.

In September this year, the HERA Structural Engineer visited two prominent Australian Fire Engineers, Drs Ian Thomas and Ian Bennetts, at Victoria University of Technology. They presented him with a BHP publication *Supporting Construction: A Guide to Fire Safety* [1] which presents the requirements of the Building Code of Australia (BCA) in terms of fire resistance requirements for supporting elements. These requirements with regard to portal frame columns supporting fire rated external concrete walls are very simple; the columns providing lateral support to the wall are not required to have a fire resistance rating.

The rationale for that comes from Australian actual fire experience, confirmed by comprehensive analytical studies undertaken up to 1990 and reported in [2], that columns with a minimum practical degree of rotation restraint at the base will remain effectively vertical in any fire condition, even when the rafters undergo significant sag. The HERA Structural Engineer has observed the effects of two burn-out fires in FHC 4 portal frame buildings supporting concrete wall panels, with unprotected columns, in which similar behaviour was observed.

In the analyses undertaken by O'Meagher et.al. [2], the rotational stiffness of the base restraint was modelled as 1500 kNm per radian at ambient temperatures. This value represented "the minimum restraint that is likely to be achieved by a typical connection used in practice" [2]. When frames with pinned bases with this level of restraint were analysed under different fire scenarios, the conclusions from O'Meagher et.al. were that the columns would dependably remain upright, as the rafters sagged, for all scenarios studied. For a column developing an inward rotation of 1° (= 17.5×10^{-3} radians), this stiffness corresponds to a moment of 26 kNm. Any of the BPP connections for I-sections from [3] involving 4 bolts, which are the details recommended for I-sections with a depth > 290 mm, will achieve this, provided that the anchor bolts are adequately embedded into the concrete foundation pad.

The evidence therefore points unambiguously to there being no need to provide passive fire protection to steel portal frame columns supporting fire rated external concrete wall panels for FHC 3 or FHC 4. However, given that the studies reported in [2] cover only part of the range of building sizes and fire scenarios possible, the HERA Structural Engineer's advice is more cautious than this and is as follows:

Updated recommendations for applying the collapsed wall condition to steel portal frame columns supporting external concrete wall panels

(1) For unsprinklered FHC 1 and FHC 2 buildings

No special detailing or considerations are required, ie. the advice is unchanged from that given in *DCB* Issue No. 51.

(2) For unsprinklered FHC 3 buildings

The columns can be left unprotected, provided that the baseplate incorporates at least the degree of moment restraint provided by the 4 bolt BPP connection, as given by R4-100 [3] for I-sections with a depth > 290 mm.

(3) For unsprinklered FHC 4 buildings

Apply the *collapsed wall criteria* given on pages 4 and 5 of *DCB* Issue No. 51. If the result of step 4 is that the permitted unprotected area <100%, then the columns do not need passive protection, however the connection at the column base needs to be designed to resist the moment generated by the pull-in force from the rafters. The method for this is given in Appendix C4 of HERA Report R4-91 [4] and the baseplate will be extended beyond the beam flanges, as shown in Item 2a from [5]. Excellent design advice for such a connection is given in Section 6 of [6]; as the level of moment is relatively low, the bolts will typically be grade 4.6 material, with two bolts each end of the baseplate positioned outside the flanges, and the baseplate thickness will be determined by moment demand from the more severe of the tension side (bolts in tension) and the compression side (bearing against concrete).

This recommendation for unsprinklered FHC 4 buildings accounts for cases where the cantilever moment on the column base generated by pull-in of the rafters during the fire may be more severe than that for building configurations studied in [2] or observed in actual fires and hence the tendency of the columns to collapse inwards might be higher.

(4) For sprinklered buildings, FHC 1 to FHC 4

Follow the guidance on page 5 of *DCB* Issue No. 51.

HSFG Nut Sizes for Threaded Rods Above M36

Background

Page 14 of *DCB* Issue No. 51 presents an article on specifying HSFG bolts, nuts and washers for sizes above M36, which is the largest diameter covered by AS/NZS 1252 [7]. For bolt lengths of up to 200 mm, two larger HSFG bolt sets (ie. bolt, nut and hardened washer) are available, as described therein.

For bolt lengths greater than 200 mm and/or diameters other than M42 and M48, a HSFG system can be assembled using threaded AISI 4140 rod, with suitably sized nuts and hardened washers of appropriate material. The *DCB* Issue No. 51 presents advice from Chris James of EDL Fasteners Ltd on each of those components. EDL Fasteners Ltd supply the Bremick range of HSFG bolts and the advice came from one of the Bremick offices.

Subsequent to the article being published, the Technical Manager from Bremick's Brisbane office has expressed concern over the adequacy of nuts sized in accordance with that advice. The advice regarding nut sizing was that they should be sized to AS/NZS 1112 [8] Table 2 (the relevant table from that standard) but with the nut height made equal to the rod shank diameter. His concern is that this increased thickness in itself may not be sufficient to prevent nut failure prior to threaded rod failure when the fastener is subjected to its ultimate tension load. (It is a requirement of HSFG fasteners that failure occurs in the threaded region of the bolt or rod when these fasteners are subject to tensile testing to destruction).

Although the advice given in *DCB* Issue No. 51 is based on industry practice as reported to EDL Fasteners, the above concern is a valid one and, in the absence of experimental testing to confirm the adequacy of nuts sized to those recommendations, the concern needs to be addressed.

The strength of the nut is controlled by the through thickness (nut height) and by the plan thickness (width across flats, width across corners). The original advice related to increasing the nut height from that specified in AS/NZS 1112 [8] to be equal to the shank diameter. If this advice was applied to the sizes of nuts for shank diameters up to M36, it would bring the nut height within the range specified by AS/NZS 1252 [7], however it would leave the width across the flats and the width across the corners less than that specified by AS/NZS 1252.

The revised recommendation given below is based on comparing the ratio of these two plan dimensions specified by [7] and [8] for shank diameters from M20 to M36, then applying that increase to the width across flat and width across corners specified by AS/NZS 1112 Table 2 for nuts for shank diameters above M36. That advice is given below.

Recommended change to nut sizing from that given in *DCB* Issue No. 51

Nuts are to be specified as for property class 8 nuts to AS/NZS 1112 [8] Table 2, with the following modifications:

- The nut height to be made equal to the shank diameter
- The width across flats and width across corners to be increased by a factor of 1.1

- The minimum washer face or chamfer diameter to be increased by a factor of 1.1

Assessing The Structural Capacity of Corrosion-Damaged Steel Beams: Brief Design Example

Background and description of example

Pages 12 and 13 of *DCB* Issue No. 51 present guidance on assessing the structural capacity of a corrosion-damaged steel beam, using a UK paper [9] in conjunction with the guidance given on pages 5-7 of *DCB* Issue No. 46.

Since writing that article, the HERA Structural Engineer has applied it to a design query involving bridge beams with no applied corrosion protection supporting a concrete deck. The bridge has been in service 15 years and the client wanted to know what its strength would be currently and in 50 years time.

For this design example, the question is simplified to what would be the current moment capacity compared with that originally available and the expected moment capacity in 50 year's time compared with that originally available. The beams are 310UB40 members, which have a compact cross section and have full lateral restraint. The bridge is located near to Warkworth, north of Auckland.

Also for this design example, it is assumed that the pattern of corrosion observed is similar to that shown in Fig. 51.3 of *DCB* Issue No. 51 - ie. that the deck has shielded the top flange from direct exposure to the weather and the bottom flange and lower part of the web are the most corroded. It is also assumed that the bridge is not located next to vegetation or other sources which would cause debris to collect on the top surface of the bottom flange and trap moisture there. If that can happen, conservatively the corrosion loss on the top surface of the bottom flange can be doubled, from that given in step 2 below, to allow for this.

Application of example

Step 1: Estimate the first year corrosion rate.

Use Hyland & Enzensberger's paper [10] and making allowance for the beam being unwashed, because of the shielding effect of the deck, determine the first-year corrosion rate, taking the worst case of unwashed or washed conditions using page 6 from *DCB* No. 46.

T_a = average daily temperature = 14.9°C (from [10]).
W_{9am} = 9am time of wetness = 0.8 (from [10]).
Distance to seacoast = 5km (appropriate for this site; as determined using a topographical map).
R_a = annual rainfall = 1834 mm/yr (from [10]).

Using equation (2) from [10].

Y_{5km} for washed conditions = 26 μm/year

Y_{5km} for unwashed conditions = 25 μm/year
W_{9am} = 1.0 (see page 6 of *DCB* No. 46)
R_a = 0

Adopt $Y_{site} = Y_{5km}$ = 26 μm/year

Step 2: Estimate design long-term corrosion rate

This is undertaken using Table 46.1 from *DCB* No. 46. As the site sits on the border between moderate and marine atmosphere classifications, a long-term multiplier of 0.40 is appropriate.

$Y_{site,\ long\text{-}term}$ = 26 x 0.4 = 10.4 μm/year

Step 3: Estimate percentage of new moment capacity available after 15 years

This estimation is made using Figure 2 of [9]. The reduction in bottom flange thickness is used for the assessment, as this flange is exposed from both sides and the site survey has shown it to be the most corroded.

Original flange thickness, $t_{f,new}$ = 10.2mm

Loss of material on flange (2 sides) = 2 x 10.4 x 15 x 10^{-3} = 0.31 mm

% loss of thickness in 15 years = 3%
% moment capacity currently remaining ≈ 95% (from Fig. 2 of [9])

Step 4: Estimate percentage of new moment capacity available after 65 years (ie. 50 years from current time)

Loss of material on flange = 2x10.4x65x10^{-3}
= 1.35mm

% loss of thickness in 50 years time = 13%
% moment capacity remaining in 50 years = 85%

Updates for Industry

Change to Galvanising Standards

Background

In what is a major revision of the standards covering galvanising, NZS/AS 1650:*1989 Hot-Dipped Galvanised Coatings on Ferrous Articles* has been replaced by five standards, providing separate and more comprehensive provisions for the different applications of galvanising. Details of these changes are given below. These details are taken from an article written by W L Mandeno, Opus International Consultants Ltd, and published in *the SESOC Journal*, Vol. 12, No.2.

In a departure from the normal format for the *DCB*, only the new (1999) standards mentioned below are referenced and included in the order form. The two existing standards mentioned in the article are also available through HERA (for HERA members) if desired.

Details of new standards

AS/NZS 4680:1999 *'Hot-dip galvanized (zinc) coatings on fabricated ferrous articles'* [11] This standard covers structural and reinforcing steel, fabricated wire and tubes, castings and nails that are galvanized using the conventional batch process, with or without centrifuging, and closely matches the international ISO 1461 Standard.

Coating thickness is determined by the thickness of the article and is now specified in microns instead of g/m^2 (eg. sections > 6 mm are required to have a minimum average coating thickness of 85 microns (μm), and a local minimum of 70 microns which is equivalent to an average of 600 g/m^2 of zinc).

AS/NZS 4791:1999 *'Hot-dip galvanized (zinc) coatings on ferrous open sections, applied by an in-line process'* [12] covers manufactured products such as cold-formed purlins. These are designated by the letters ILG (in-line galvanized) and by a two or three digit number representing the specified minimum average zinc coating mass in g/m^2 on **each** side (eg. ILG150 has a minimum average thickness of 21 μm. This section would previously have been designated as Z300, which was based on the total weight of coating in g/m^2 on both sides of the sheet).

AS/NZS 4792:1999 *'Hot-dip galvanized (zinc) coatings on ferrous hollow sections, applied by a continuous or specialised process'* [13] covers three different manufactured products as follows;

a) Hollow sections coated on both surfaces. RHS with a coating thickness of 300 g/m^2 (42 μm) would be designated HDG300.

b) Hollow sections produced by welding pre-galvanized strip. A section made from Z275 sheet would be designated ZB135/135. These sections may have a different coating thickness between external and internal surfaces.

c) Hollow sections coated on the external surface only. A tube with an external coating of 100 g/m^2 would be designated as ILG100.

Notes:

1. Continuously galvanized coatings are different to those formed in the traditional hot-dip galvanizing process. Batch dipped items have a longer time in contact with molten zinc, typically from 2 to 8 minutes. This results in a coating which is less ductile, but is thicker and more abrasion resistant (due to the formation of a thicker iron/zinc alloy layer that is harder than the base steel). Care is therefore required in specifying product for external use, especially in marine or other aggressive environments where the in-line/continuously galvanized coating will require additional protection to give eg. a 50 year durability.

2. Galvanized wire and welded wire fabric is covered by the existing AS/NZS 4534:1998 *'Zinc and zinc/aluminium-alloy coatings on steel wire'*, and galvanized sheet is covered by the existing AS 1397-1993. Abstracts of both these now follow:

AS 1397-1993 *'Steel sheet and strip - Hot-dipped zinc-coated or aluminium/zinc-coated'* Specifies requirements for coating formable and structural grades of hot-dipped zinc-coated and aluminium/zinc-coated sheet and strip, up to and including 5.0mm thickness. Also includes fabrication characteristics and guidelines on the selection of specific grades.

AS/NZS 4534:1998 *Zinc and zinc/aluminium alloy coatings on steel wire.* Specifies requirements for mass, quality and testing in zinc coatings and zinc/aluminium-alloy coatings on steel wire of circular or non-circular cross-section. The coatings are applied using a continuous process which may comprise immersion in molten metal or electrodeposition. The Standard applies to coatings on wire at its final size and specifies six coating mass classes. It specifies methods for the determination of coating mass and gives advice on the transport and storage of coated wires, and on coating selection for corrosion protection.

Company	Contact	Phone/Fax	Bath Size L x W x D
Avon Industries Ruatangata Road P.O. Box 27 Whangarei 0115	Richard Fisher	Ph: 0-9-435 1033 Fax: 0-9-435 2565 Mob: 025 939 060 Email: avonind@xtra.co.nz	4.88 x 0.9 x 1.35
CSP Galvanizing 40-44 Gavin Street P.O. Box 11 165 Ellerslie Auckland 1131 CSP Galvanizing 27 Washbournes Road P O Box 11 183 Sockburn Christchurch 8030	Paul Thomsen Andrew Burn	Ph: 0-9-579 0063 Fax: 0-9-579 8907 Mob: 025 306 188 Email: paulth.@fcsp.co.nz Ph: 0-3-348 8522 Fax: 0-3-348 5888 Email: AndrewBu@fcsp.co.nz	9.0 x 1.37 x 2.2 7.5 x 1.5 x 2.1
East Tamaki Galvanizing 2/33 Springs Road East Tamaki P.O. Box 58 666 Greenmount Auckland 1730	Bob Hamilton	Ph: 0-9-274 0524 Fax: 0-9-274 0525 Mob: 025 906 145 Email: etgalv@xtra.co.nz	2.4 x 0.9 x 1.4
Franklin Machinery 37 Subway Road P.O. Box 445 Pukekohe 1800	Ian Richards	Ph: 0-9-238 9289 Fax: 0-9-238 5686 Mob: 025 486 243 Email: terric@gallagher.co.nz	4.5 x 0.9 x 1.3
Galvanizing Services 23 Edinburgh Street P.O. Box 13 181 Onehunga Auckland 1132	Andrew Lonsdale-Cooper	Ph: 0-9-636 6003 Fax: 0-9-636 5967 Email: gsl@xtra.co.nz	3.3 x 1.0 x 1.5
Galvanising Hawkes Bay 41 Thames Street P.O. Box 1114 Napier	Dave Bickerstaff	Ph: 0-6-835 4499 Fax: 0-6-835 9499 Email: d.bickerstaff@xtra.co.nz	6.5 x 0.8 x 1.5
Kibby's Metal Pressings Dawson Street P.O. Box 75 New Plymouth	Marsh Kibby	Ph: 0-6-758 2210 Fax: 0-6-757 5029	2.5 x 0.7 x 1.1
New Zealand Galvanisers 129 Hutt Park Road P.O. Box 38 956 Petone Wellington 6015	Neil Hoey	Ph: 0-4-568 4139 Fax: 0-4-568 4808 Mob: 025 420 072	8.5 x 1.1 x 1.3
Perry Metal Protection (BOP) Ltd 253 Oropi Road P.O. Box 9226 Tauranga 3030	Ken Tynan	Ph: 0-7-541 2222 Fax: 0-7-541 0813 Mob: 025 938 507 Email: kentynan@voyager.co.nz	5.0 x 0.9 x 2.4
Perry Metal Protection 14 Manchester Place P.O. Box 10 406 Te Rapa Hamilton	Steve Williams	Ph: 0-7-850 0120 Fax: 0-7-850 7782 Mob: 025 720 650 Email: stevepm@perry.co.nz	7.0 x 1.2 x 2.4
Southgalv 7 Mount View Place P.O. Box 16-439 Hornby Christchurch	Eric Black **Chairman** **G.A.N.Z.**	Ph: 0-3-349 0290 Fax: 0-3-349 0123 Mob: 021 610 899 Email: ericsgal@xtra.co.nz	5.0 x 1.2 x 1.8
Webforge 23 Kelvin Grove Road P.O. Box 1506 Palmerston North 5315	Chris James	Ph: 0-6-356 1246 Fax: 0-6-356 7782 Mob: 021 436 338 Email: webforgecj@manawatu.gen.nz	7.2 x 1.3 x 1.6

Table 52.1
Galvanizing Baths Belonging to GANZ Members in New Zealand
(as of October 1999)

Galvanizing Bath Sizes Available in New Zealand

Following on from the previous article on changes to the galvanizing standards, it is timely to include details of the galvanising bath sizes available in New Zealand and their locations.

That information is presented in Table 52.1. It covers galvanising baths run by members of the Galvanising Association of New Zealand (GANZ) and which will accept external work.

For more information on GANZ, contact the current chairman, Eric Black of Southgalv (for details see second entry from the bottom of the table).

Inspection of Shear Studs Installed by a Machine Which Records Weld Cycle Parameters

Background

DCB Issue No. 44, on pages 3-6, contains detailed guidance on the on-site inspection of end arc-welded headed shear studs during construction. This guidance is written around the requirements of the relevant standard, AS 1554.2:1993 [14] and is referenced from HERA's structural steelwork specification, HERA Report R4-99 [15].

The provisions of [14], and hence of the *DCB* Issue No. 44 guidance, are written around the capabilities of conventional arc stud welding machines. These machines operate a pre-set welding cycle, for which the equipment parameters such as current, length of time the stud is held in each position of the welding cycle, etc. are set by the operator in the prequalification setup prior to commencing production welding. Once these are set, they are kept constant during the production run, within the operating tolerances of the machine. No record of the parameters is kept for each stud welded, so the only way of determining whether a stud has been successfully welded is to examine the finished appearance of the weld (this is termed the *flash*). The requirements of AS 1554.2 [14] therefore attach considerable importance to this visual examination, as detailed in the Part 2 and Part 3 notes given on pages 4-6 of *DCB* Issue No. 44.

However, there is a new generation of stud welding machine now used in New Zealand, which controls the welding operation in a different, more direct manner and which produces a permanent record of the key weld cycle parameter for each stud welded. Brief details of this machine's capabilities have been presented in *DCB* Issue No. 22, on page 7.

There are two important differences between this type of machine and conventional stud welding machines, with regard to this article. These are:

(1) For a stud to be effectively welded in place, each stage of the stud welding cycle must be successfully completed before the next stage is commenced. Successful completion of any given stage is dependent on a number of parameters, which will vary slightly throughout the production run. With a traditional machine, the welding cycle is preset during the prequalification stage. This means that variation in parameters such as site power supply, voltage and current, length of cabling between machine and welding gun, will directly affect the finished weld. The new generation machines, in contrast, continuously monitor the weld cycle and make automatic adjustments to each step of the weld cycle for variations in critical input parameters, such that each stage of the stud welding cycle is successfully completed before the next stage is commenced.

(2) The critical part of the weld cycle, in regard to the effect of variation in input parameters, is when the end of the stud and receiving surface of parent metal are melted to produce the molten weld metal that will form the weld. Key parameters are current strength and time. The suitability of these parameters is initially set during the prequalification stage. The new generation machine provides for a written output of these parameters for each stud welded, which offers a permanent record as to the successful completion of the weld cycle.

The availability of this written record can be used to facilitate quality control testing of arc welded headed shear studs. No guidance on this is given in AS 1554.2 or in *DCB* Issue No. 44, because the Standard was written before the new generation machines have become available and the Bulletin article refers only to the Standard.

The purpose of the next part of this article is to present guidance on how to modify the on-site quality control requirements given in AS 1554.2 and *DCB* Issue No. 44, for studs welded by new generation stud welding machines, to take full advantage of the monitoring and recording capability of these machines.

Testing of shear studs installed by a machine which monitors and records weld cycle current and time

Preamble

The guidance is written in the same format and using the same headings as that presented for traditional stud welding machines on pages 3-6 of *DCB* Issue No. 44. Readers should be familiar with those requirements when following the advice below. See page 3 of that article for definitions.

Part 1: Prequalification requirements

This follows the procedure given on page 4 of *DCB* Issue No. 44, except that, once the current setting appropriate for the planned production run covered by the prequalification is determined (this will have been done off-site with the settings pre-programmed into the machine), two test studs should be placed at current settings +10% over and -10% under the optimum setting. These two studs should be bent to 30° from the vertical without failure and are not considered effective in design.

This prequalification procedure establishes the suitability of a 10% variation from the target current to still produce sound welds.

Part 2: Production testing by the stud welder

The stud welder will remove the arc shields from the base of the studs and will keep a written record of the current settings obtained for each stud welded.

Any studs for which the current setting is within 10% of the target setting may be assumed satisfactory prior to part 3 testing.

Any stud for which the current setting deviates from the target setting by more than 10% shall be tested by bending to 15° off vertical. If any of the flash is missing on such a stud, the direction of bending shall be away from the missing portion of flash. A stud which successfully passes this bend test may be left in place and assumed fully effective in design, provided that it has not been bent beyond 20° off vertical.

The stud welder should identify any stud so tested on the written record provided by the welding machine for review by the construction reviewer.

Part 3: Quality control testing by the construction reviewer or by his/her nominated representative

1. Start by noting the target current setting from the prequalification procedure and check that the two studs welded to ± 10% variation on this setting have been bent to 30° off vertical without failure.

2. Then sight the printed output from the stud welding machine and ensure that this output covers all studs being inspected.

3. Check that any studs, for which the current setting has deviated from the target setting by >10%, have been bent to 15° and are either satisfactory or have been replaced with a satisfactorily welded stud.

4. In addition, select 1 in 200 studs at random and bend these to 15° off vertical. In selecting these additional studs, observe the flash and select studs with any missing flash for testing. Studs which pass this test may be assumed fully effective in design.

   If all additional studs testing pass the bend test, then the inspection is complete and step 5 is not needed.

5. If any of these additional studs tested fail the bend test, then inspect the flash of all studs and bend any stud exhibiting less than 360° flash to 15° off vertical, with the direction of bending opposite to the missing portion of the flash. Any stud which passes this bend test, provided that it has not been bent more than 20°, may be assumed fully effective in design. Any stud failing this test must be replaced with a satisfactorily welded stud.

   Who is to be responsible for any additional inspection required under step 5 (if this is needed after implementing step 4) should be specified in the contract documents prior to stud welding commencing. This will either be the stud welder or the independent inspector. The cost of this additional inspection and any retesting of replacement studs should be borne by the stud welder, as such cost will only be incurred if the desired level of quality control implicit in steps 1-4 is not met.

Part 4: Replacement of unacceptable studs

Follow the requirements of *DCB* Issue No. 44 on page 6 therein.

Final point to make

The aim behind this guidance is to take advantage of the written record of current setting achieved for each stud, as provided by the new generation stud welding machines, so as to avoid the need to visually inspect each flash and to bend test each stud not showing a full 360° flash. Making use of the written output offers a quicker way of inspecting for on-site quality of welded studs, without compromising this inspection. The inclusion of step 5 is intended to ensure that the backup reliability obtained through inspecting the flash and bend testing is maintained, but is only called into use if the written record offered by the new generation stud welding machine proves unreliable.

Minor Revisions to Recommended Deflection Limits for Composite Floor Systems

Preamble

Designers need to have an understanding of the behaviour of composite beams under short-term and long-term loading, so that an adequate allowance for serviceability deflection can be made in the design and allowance can be made for construction effects such as concrete ponding.

These issues are covered by a comprehensive article on pages 8-14 of *DCB* Issue No. 33 on deflection of composite floor systems. That article addresses the nature of composite floor system deflection, the important influences and the interaction of these influences. It ends with Table 33.2: *Recommended Deflection Limits for Composite Slab on Decking and for Composite Beams.*

Since that article and table were published, in June 1997, those provisions have been applied to a wide variety of composite floor systems. These applications have shown the recommendations to be generally satisfactory in terms of constructability and in-service performance. They have also identified two areas where minor revisions should be made. These revisions are given below.

Proposed revisions to *DCB* Table 33.2

The first two limits to be revised are for *constructability*, to limit soffit deflection from ponding under the wet concrete. They are:

- The current deflection limit for unpropped internal beams of L/250 should have an upper limit of 20 mm added, thus giving L/250 < 20 mm.

- The current deflection limit for unpropped spandrel beams of L/360 < 25 mm should read L/360 < 20 mm.

The second two limits to be revised are for *functionality: surface slopes*. They relate to carpark occupancies and are as follows:

- For unpropped beams, in carpark buildings, the current limit of L/250 < 25 mm should read L/250 < 35 mm.

- For propped beams in carpark buildings, the current limit of L/250 < 25 mm should read L/250 < 35 mm.

In all typical cases, L = span between supports.

Bulletin readers should make the above modifications to their copy of *DCB* Table 33.2.

Precambering of Hot-Rolled Beams

One option for limiting the deflection induced by the concrete wet load in unpropped steel beams is to precamber them upwards to counter the calculated wet concrete deflection.

As mentioned on page 9 of *DCB* No. 33, cold-bending of hot-rolled sections to deliver the precamber is only practical for the smaller UB sections. There is a significant cost and time associated with the precambering; from the *Structural Steelwork Estimating Guide* (HERA Report R4-96 [24]) this is given as 2.9 hours and \$135 per beam for ≤ 310UB and 4.1 hours and \$189 per beam for the 360UB and 410UB designations. Cold-bending of > 410UB is not practical, nor is hot-bending of the numbers of beams typically required for a floor system.

Therefore, the advice to designers when considering hot-rolled beams for support of concrete slabs on steel decking is:

- For beam sizes up to 410UB, precambering is an option but be aware of the cost involved
- For beam sizes above 410UB, precambering is not a practical option.

Alternatives to not precambering an unpropped hot-rolled beam are to use a deeper size to reduce concrete wet load induced deflection to less than the limits given opposite or in Table 33.2, or to use a precambered, welded beam. The latter can be optimised for weight and there is little or no additional cost for precambering.

Guidelines for Light-Weight Steel Framed House Construction

General

There is increasing interest in the use of light-weight steel frames (LSF) for houses, because LSF houses offer superior dimensional accuracy and long-term stability compared with timber framed houses, as well as greater lateral strength and stiffness, all at a competitive price.

The general concepts are similar to timber framed construction, as shown by Fig. 52.1, however the design, specification and fabrication requirements are very different. The principal differences are that:

- The steel stud is a carefully engineered item, involving significant design and manufacturing input in order to generate the required level of performance
- Each proprietary system of LSF house construction is a specifically engineered package of members and connections
- The connections may incorporate connectors and connection components which are specifically designed and tested for use in a particular system.

This combination of similarity and difference between LSF and timber framed house construction can lead to some confusion and even omission of important aspects in the design, building consent, inspection, specification or construction stages. This is apparent from various queries on LSF construction fielded by the HERA Structural Engineer from time to time.

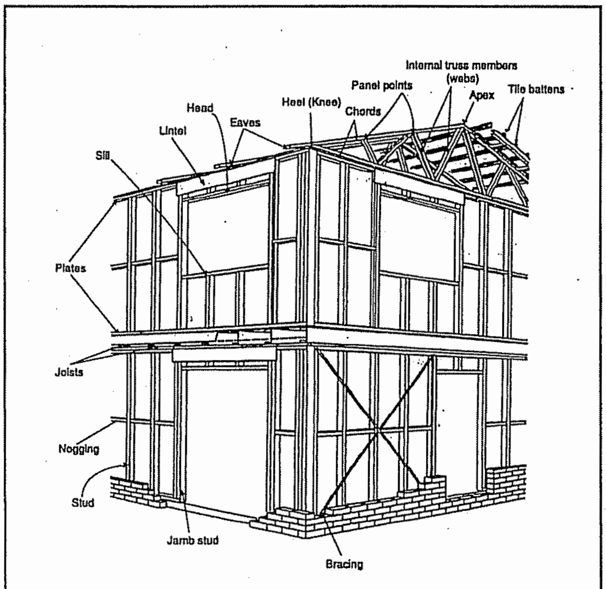


Fig. 52.1
Typical Schematic of LSF House Framing System (from [16])

Many of these queries are answered in a publication from NASH New Zealand (The New Zealand Chapter of the National Association of Steel-Framed Housing), entitled *Guidelines for Light-Weight Steel Framed House Construction: First Revision*. This publication [16] has been written to provide an overview of the important aspects of design, fabrication and construction of LSF houses.

Scope of NASH Guidelines

This document [16] has four sections, dealing with design, specification/documentation, fabrication and construction. It lists the key aspects to consider in each of these sections, typically through raising a series of points or asking questions which require consideration.

There are three appendices included. The first, Appendix A, contains a checklist of items for consideration by designers, territorial authorities or building certifiers, design reviewers and construction reviewers.

Appendix B provides design shear and tension capacities for screws into various commonly used thicknesses of light gauge steel. These design capabilities have been established by experimental testing to the required standard, ie. AS/NZS 4600 [17].

Appendix C contains two durability statements. The first covers durability of above-ground LSF construction and the second covers durability of floor joists. Each is also available separately from NASH New Zealand.

It is intended that these guidelines will be used by all personnel involved in the design, construction and inspection of a LSF house. This use will involve:

- Identifying all relevant points requiring consideration by those supplying data or performing a service
- Identifying relevant areas of inspection by building consent reviewers and construction reviewers.

Design Guidance for use with AS/NZS 4600

AS/NZS 4600:1996 [17], *the Cold-Formed Steel Structures* Standard, is a relatively complex standard to use. This is because the standard covers design of high-strength, thin gauge, cold-rolled structural sections and the connections between them. These sections and the structural systems formed from them exhibit modes of failure and deformation that are not commonly encountered in heavier gauge structural steel design.

Designers using AS/NZS 4600 should therefore take advantage of the supporting design guidance available for use with the Standard [17]. Details of this supporting design guidance are given on pages 16-18 of *DCB* Issue No. 47.

Lateral Restraint and Load Bearing Capacity in The Support Regions of Continuous Beams

General

There has recently been reported a partial collapse of a suspended floor in a steel framed car parking building during construction.

The gravity load-carrying system for the suspended floor comprised precast concrete slabs with topping supported on a grid of continuous, long span secondary beams, which were in turn supported by three primary beams. These primary beams were located at each end and at the centre of the building, with the centre primary beam supporting 50% of the total floor area vertical load. The secondary beams were located above the primary beams, thereby achieving their continuity over the primary beam supports, in the manner shown in *Connection Detail 12* of HERA Report R4-92 [18]. (Except that, in this particular case, there were no load-bearing stiffeners in the primary (main) beam under the secondary beams).

Each primary beam was a two-span beam supported on three columns and continuous over the central supporting column. The primary beams were unstiffened over all the columns, thus the connection detail between the beams and columns at each end of the primary beams was the same as that shown in *Connection Detail 7* of HERA Report R4-92 [18].

The collapse occurred after the topping for the suspended floor had been poured and levelled. It involved the central primary beam failing where it was supported over the central column, causing the bottom flange and supporting column to kick out sideways.

It is not the purpose of this article to investigate the collapse of that particular floor, but rather to review the requirements for lateral restraint and load bearing capacity in the support regions of continuous beams. This review now follows, with reference to the detail shown in Fig. 52.2. That detail has been altered somewhat from the detail that initiated the partial collapse mentioned above, in order to remove this article from being seen as directly providing answers as to why that collapse may have occurred.

The lateral restraint and load bearing capacity issues to consider in this situation are as follows:

- Beam web capacity under combined shear and bending
- Bearing capacity of the beam web
- Lateral restraint of the beam in bending
- Lateral restraint of the top of the column for compression

Each of these is now covered in turn.

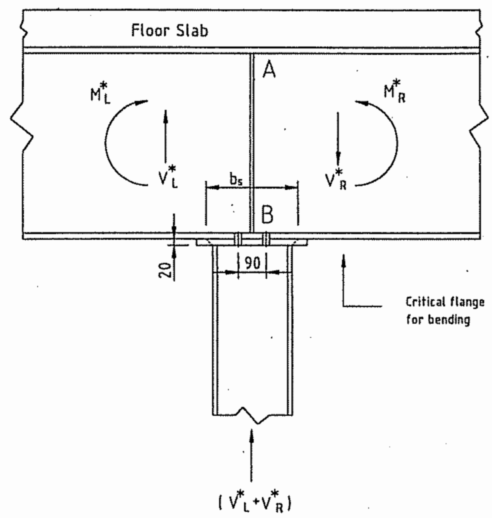


Fig. 52.2
Continuous Primary Beam Running Over Supporting Column

Beam Web Capacity Under Combined Shear and Bending

This relates to the capacity of the beam web at the face of the column, where it is subject to design moment M^* and design shear V^*. The check is to NZS 3404 [19] Clause 5.12.2. For a hot-rolled I-section, this check will typically be easily satisfied.

Bearing Capacity of the Beam Web

At the supports, the load carried by the primary beam must be transferred into the column. This requires a check on the bearing capacity of the beam web.

The dispersion of force within the beam web for this check is given in NZS 3404 Clause 5.13.1 and Fig. 5.13.1. Calculation of the bearing capacity of the web requires a check on both bearing yield (Clause 5.13.3) and bearing buckling (Clause 5.13.4). The calculation of stiff bearing length, b_s, is as given in Fig. 5.13.1.2 of [19] right hand diagram and also illustrated in Fig. 52.2. above.

Calculation of the unstiffened web bearing capacity for hot-rolled sections manufactured by BHP Australia is made very straightforward through the tabulation of bearing yield and bearing buckling capacity as a function of the relevant bearing length, in the *AISC Design Capacity Tables* [20].

For primary beams over supporting columns, load bearing stiffeners will be required when the unstiffened web fails the above bearing checks. Calculation of the yield capacity and buckling capacity of the stiffened web is given by NZS 3404 Clause 5.14.

When load bearing stiffeners are used, a pair of them is required; typically these are placed about the centreline of the column, as shown in Fig. 52.2.

When the load-bearing stiffeners are placed within the stiff bearing length, b_s, as shown in Fig. 52.2, then the web bearing capacity checks and the check for combined shear and bending are undertaken separately. More background information on the design of webs for a range of combined actions is given in section 9.7 of HERA Report R4-80 [21].

Lateral Restraint of the Beam in Bending

The beam is subject to negative moment over the supporting column, making the bottom flange critical. Furthermore, under elastic analysis, the maximum moment along the beam span is the negative moment over this support region. This means that, at the least, there will be a need for the lateral restraint supplied to the segments on each side of this support to be near or at full lateral restraint status. If moment redistribution is used to make the negative and positive moments along the beam spans more equal, then this involves moment redistribution away from the support region, with the segments on each side of the support then required to have full lateral restraint, in accordance with NZS 3404 Clause 12.6.2.

In the situation shown in Fig. 52.2, the flooring system will provide inherent overall lateral stability in any practical structural system with a solid floor slab. This means that the top flange, which is the non-critical flange, will be laterally restrained, however the bottom flange will not be directly laterally restrained.

From *Connection Detail* 7 of HERA Report R4-92 [18], the bottom flange in the detail shown in Fig. 52.2 inherently has full section restraint (F). **However, this is only achieved when the strength requirements associated with F section restraint are met.** It is very important that this strength check is met in this connection detail, to avoid a lateral buckling failure due to the bottom flange kicking sideways under applied loading.

The requirement is to prevent lateral movement of the bottom flange - ie. at point B (bottom of stiffener). To achieve this, a lateral restraining force as given by equation 2 from R4-92 [18] applied at that point must be resisted.

This can be resisted in one of two ways; **the first method** is through effective twist restraint of the cross section. In this, which is described in section 2.2.2 of HERA Report R4-92 [18], a moment is developed at the beam top flange level (ie. at point A in Fig. 52.2) and this moment must be resisted through the floor system. This will require determination of the flexural restraint available between the beam and floor system and the flexural stiffness of the floor system to resist this moment. Where a concrete slab is present, the flexural restraint between the beam and floor slab can be provided using shear studs to transfer the tension component. Also a concrete slab will typically provide adequate flexural stiffness and strength to resist the incoming restraint moment without detailed consideration.

However, as described in section 2.2.2 of [18], an unstiffened web is unlikely to be able to develop the required moment capacity, hence at least a stiffener on one side will be needed for this option; the load-bearing stiffeners shown in Fig. 52.2 can be used for this, provided that they can develop the necessary moment capacity, which will typically easily be met.

As an alternative to moment restraint via this detail, and especially when the web is unstiffened, a fly brace restraint can be used to transfer the restraining force at B up to the floor slab and anchor it in there, provided that there is sufficient local capacity in the floor system.

If there is a secondary beam framing into the primary beam at no further than a beam depth remote from either column face, then this can be used to resist the moment. This involves a *Connection Detail 10* type condition from [18].

The second method in which the lateral restraining force at B can be resisted is via the beam and supporting column. In this case, the column system is considered to run from a pinned connection at the top flange to the column base and is subject to a point lateral load at B. This generates minor axis bending in the column, which reaches a maximum at point B. The magnitude of this moment is given by the simple equation (Pab/L) and will be almost equal in magnitude to the moment generated at A by the force B in the first method detailed above. Once again, this moment will usually exceed the design moment capacity of the unstiffened beam web at B, necessitating the use of one or two stiffeners. The connection between the beam and column will need to resist this out-of-plane moment and the column will be subject to minor axis bending. This second method must be used if there is no moment restraint available at the top of the beam into the floor slab, which rules out using method 1.

If there is no direct moment or lateral connection to the beam top flange located within a distance equal to the beam depth away from the face of the column on either side of the connection, then method 2 can still be used when the beam is not expected to form yielding regions adjacent to the supporting column. The restriction to situations where the beam is designed to remain elastic is because the loss of stiffness associated with the beam becoming inelastic, in those regions, could well lead to lateral instability, even with the stiffener in place.

The restraint available during construction may be different from that available once the floor slab is hardened and this must be considered wherever appropriate.

Lateral Restraint of the Top of Column for Compression Action

Where the floor system provides inherent overall lateral stability to the structural system, the column will be designed as a braced member and the top of column will need to be restrained against out-of-plane movement accordingly.

The restraining force is 2.5% of the design axial compression force in the column and is applied at point B in the same manner as for lateral restraint to the beam bottom flange for negative moment. The two restraining forces are not cumulative; design to resist the greater of the two and this will cater for the lesser case.

The two methods available for resisting this restraining force back to anchorage points are as for the bending restraint force described above.

Improved Design and Detailing for Column Panel Zone Doubler Plate Reinforcement

Background

For hot-rolled I-section columns, or for welded I-section columns in which the column web to column flange welds can develop the design tension capacity of the column web, doubler plates can be used to increase the capacity of the column web to resist out-of-balance shear force across the connection region, eg. as developed by earthquake action on the connection. The design and detailing of doubler plates for this is covered in *DCB* Issue No. 47, with that advice incorporated into the supplementary issues relating to WM and MEP connection design given in *DCB* Issue No. 50. These provisions are then applied to design example no. 51.2 in *DCB* Issue No. 51.

The need for doubler plates is ascertained in accordance with NZS 3404 [19] Clauses 12.9.5.2 and 12.9.5.3.2. For the MEP connections, these clauses are cross-referenced from Clause M4.

When using these provisions, it will generally be found that doubler plates are not needed for connections in which only one beam frames rigidly into the column, as shown in Fig. 51.4, Design Example 51.1, *DCB* Issue No. 51 for example.

When two beams frame rigidly into the column, as shown in Fig. 51.5, Design Example 51.2, *DCB* Issue No. 51, then one or possibly two doubler plates will be required. Fig. 51.5 shows an example where 1x6 mm thick doubler plate was required.

The fitting of this doubler plate into the connection and the weld details around its sides that are shown in Fig. 51.5 follow the recommended details given in *DCB* Issue No. 47. These details are based on extending the doubler plate 50 mm beyond the connection region, welding it to the column web and welding the tension/compression stiffeners onto the doubler plate.

Not only does this detail result in two additional runs of weldment across the column web per doubler plate, compared with the number of welds that would be required in the absence of the doubler plate, but it results in a complex load path to get axial forces from the tension/compression transverse stiffeners into the column web. These forces must transfer from the transverse stiffener to the doubler plate and then into the column via the top, bottom and sides of the doubler plate.

The SSAS Manager and the HERA Structural Engineer have developed an improved detail for fitting the doubler plate into the column joint and welding the top and bottom of it into the connection. This is presented overleaf.

Construction Concept Behind Improved Detail

This concept is based around the fact that tension/compression stiffeners will be required in any connection containing column web doubler plates.

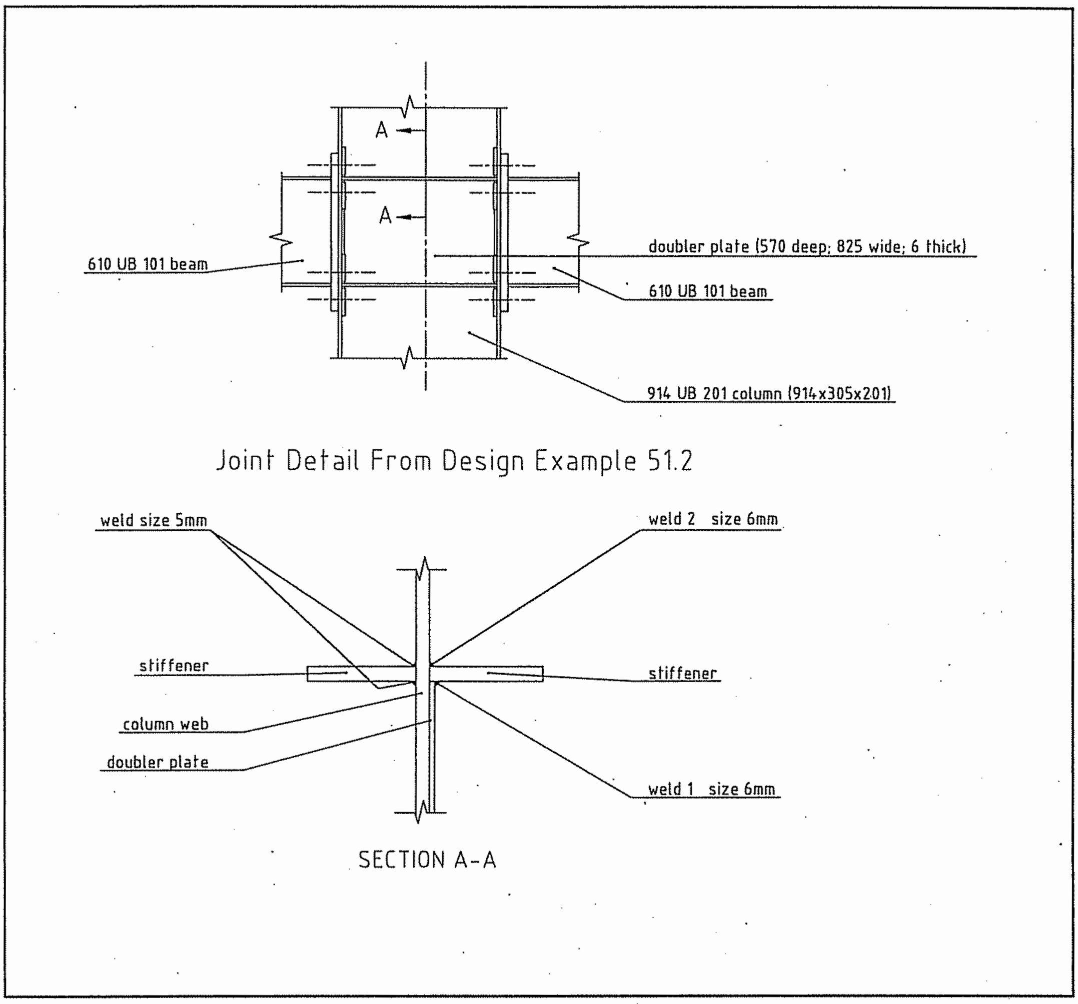


Fig. 52.3
Improved Design and Detailing of Column Panel Zone Doubler Plate Reinforcement

The concept involves first cutting and fitting the tension/compression stiffeners into the column joint and welding them into the inside faces of the column flanges. The doubler plate(s) is/are then cut and fitted between the inside faces of these stiffeners, as shown in Fig. 52.3.

Finally a fillet weld is run between the doubler plate and the inside face of the tension/compression stiffener (this is weld 1 shown in Section A-A, Fig. 52.3) and a fillet weld of at least the same size is run between the outside face of the tension/compression stiffener and the column web (this is weld 2 shown in that detail).

The advantages of this detail are that:

(i) No additional runs of weld across the column web are required when a doubler plate is used.

(ii) All tension/compression stiffeners are the same size and in the same location, irrespective of whether or not a doubler plate is used.

(iii) The load path between tension/ compression stiffener/doubler plate/column web is much more direct.

(iv) The fillet weld size between the tension/compression stiffener and column web (ie. weld 1 in Section A-A of Fig. 52.3) is not limited by the thickness of doubler plate used, as was the case for the previous detail.

(v) The fillet weld size required at the top and bottom of the doubler plate is not limited in leg length to the doubler plate thickness, as was the case for the previous detail.

The only disadvantage is that the sizing of welds 1 and 2 is marginally more complex than with the detail previously proposed in *DCB* Issue No. 47. This weld sizing is covered below.

Design Requirements for Weld 1

Weld 1 is required to transfer the difference between the panel zone design shear force and the design shear capacity of the web alone between the doubler plate and the tension/compression stiffener.

This requirement is given in section 2.5, page 10, *DCB* Issue No. 47 for doubler plates in WM connections and in section 2.5, page 12, *DCB* Issue No. 47 for doubler plates in MEP connections.

Design Requirements for Weld 2

This weld is sized for the greater of:

(1) The weld size required for weld 1, or

(2) $1.5v^*_{w,s(cw\ or\ dp)}$ where $v^*_{w,s(cw\ or\ dp)}$ is given by equation 50.5, *DCB* Issue No. 50.

The reason for (1) is straightforward.

The reason for (2) is that this weld must also transfer axial force from the incoming beam flanges through into the web. Equation 50.5, *DCB* Issue No. 50 was based on developing the design section capacity of the transverse stiffener through two fillet welds along the web. With the proposed detail shown in Fig. 52.3, only one weld is now available for this, however experimental tests have shown that it is conservative to base the design requirement on developing the design section capacity of the stiffener over only half the column depth, which is what equation 50.5 does for two beams framing into the column at the connection. Instead the one weld is sized to dependably transfer 0.75 x the design section capacity of the stiffener. Because the length of weld available for this is now halved, the increase in design shear force for sizing the weld is 0.75/0.5 = 1.5. As illustrated in the application below, the first case will usually still govern the size of weld 2.

Design Requirements for Welds to the Sides of the Doubler Plate

These are the welds between the sides of the doubler plate and the column root radius or column web to flange weld, as shown in Fig. 47.5 where they are designated DP,S.

For doubler plates to WM connections, there is no change to the requirements, which are given in section 2.4, page 10, *DCB* Issue No. 47.

For doubler plates to MEP connections, the requirement is for these side welds to develop the design shear yield capacity of the doubler plate, as stated in section 2.4, page 12, *DCB* Issue No. 47. The new provision for fitting the doubler plate between the tension/compression stiffeners will reduce the doubler plate depth, hence reducing the design shear area, A_w. However, it will also reduce the length of weld available to resist this force. The result will be little if any change in weld size required, as illustrated in the operation example which follows.

Application of These New Provisions

Use of these new provisions is now illustrated by application to the doubler plate previously sized for Design Example 51.2, on pages 20 and 21 of *DCB* Issue No. 51.

Sizing of doubler plate

The width and thickness of doubler plate remain unchanged from that specified in sections 3.4(4) and 3.4(7), pages 20 and 21, *DCB* Issue No. 51.

The depth of doubler plate reduces from the original depth of 705 mm to a new depth of (602 - 2x16) = 570 mm. This is the clear depth between the top and bottom tension/compression stiffeners.

The check on doubler plate thickness in relation to weld leg length along the tension/compression stiffeners given in section 3.4(6), *DCB* Issue No. 51 is no longer required, for the reason given under item (iv) of Construction Concept, page 14 above.

Welds down sides of doubler plate

V^*_w = $0.9 \times 0.6 f_{yp} A_{wp}$ = 517 kN

$A_{wp} = t_p d_p$ = 6x570 = 3420 mm^2
f_{yp} = 280 MPa (from Table C2.2.1 of [19])

ϕV_w for t_w = 6 mm (largest possible)
= 0.978 x 570 = 557 kN OK
E48XX, Category SP weld

Must build edge of weld out as per NZS 3404 Fig. 9.7.3.3 (c).

Note that this weld detail and size is unchanged from that previously used, as stated in section 3.4(8) of *DCB* Issue No. 51.

Sizing of weld 1

This is the weld between the doubler plate and the inside face of the tension/compression stiffener, as shown in Section A-A, Fig. 52.3.

Weld 1 is sized in accordance with section 2.5, page 12, *DCB* Issue No. 47.

$V_w^* = \left(V_p^* - \phi V_{c,web}\right) = 704\ kN$

$\phi V_{c,web} = 2106\ kN$

ϕV_w for $t_w = 6$ mm

$= 0.978 \times 825 = 807$ kN OK

Note that, unlike the previously recommended detail, given in section 3.4(9), *DCB* Issue No. 51, the size of (leg length) this weld is not limited to the doubler plate thickness, although in this particular case they are the same.

Sizing of weld 2

This weld is sized for the greater of $t_w = 6$ mm (weld 1) or $1.5 v^*_{w,s(cw\ or\ dp)}$, where $v^*_{w,s(cw\ or\ dp)}$ is given by equation 50.5, *DCB* Issue No. 50.

Referring to section 3.1(8), *DCB* Issue No. 51;

$v^*_{weld\ 2} = 1.5 v^*_{w,s(cw\ or\ dp)} = 1.5 \times 0.52$
$= 0.78$ kN/mm

$\phi v_w = 0.82$ kN/mm for $t_w = 5$ mm, E48XX Category SP

$t_{w,weld\ 2}$ = greater of 6 mm or 5 mm = 6 mm

Weld size for fillet welds between tension/compression stiffeners and column web on the side of the web without doubler plate

As given by section 3.1(8), *DCB* Issue No. 51;

$t_w = 5$ mm

Comparison Between Doubler Plate Sized and Fixed to the New Provisions Compared With That Previously Designed

- The doubler plate thickness and width are unchanged, depth reduces from 705 mm to 570 mm
- The volume of deposited weld down the sides of the doubler plate is reduced, because of the reduction in depth
- The number of weld runs and volume of deposited weld metal laid in runs across the column web is significantly reduced, from 8 runs of 5 mm fillet weld and 2 runs of 6 mm fillet weld to 4 runs of 5 mm fillet weld and 4 runs of 6 mm fillet weld
- The fillet weld to the top and bottom of the doubler plate does not need building out to obtain the required leg length.

All these factors will reduce the installed cost of doubler plate and hence the overall cost of the joint.

HERA's Semi-Rigid Joint Research; November 1999 Update on Research Programme and Design Provisions

General Introduction and Scope of Article

HERA is engaged in a long-term research project aimed at developing new forms of semi-rigid joints for moment-resisting steel framed seismic-resisting systems (MRSFs). These joints are intended to remain rigid up to the design level ultimate limit state earthquake moment, eg. as derived from NZS 4203 [22], then to allow rotation to occur between the beam and the column, when this design moment is exceeded. The joint is then designed and detailed to withstand the expected inelastic rotation associated with the design level ultimate limit state earthquake with negligible damage, such that minimum or no repair is necessary following that magnitude of earthquake. Finally, the joint is expected to be able to withstand greater levels of inelastic rotation, associated with more severe events, without catastrophic failure, instead undergoing at worst a gradual loss of capacity with increasing cyclic rotation demand beyond the design severe seismic level.

Of the joint types that have been researched to date, two details have emerged as preferred options for the beam to column connections of MRSFs. These are the *Standard Flange Bolted Joint* (SFBJ) and the *Sliding Hinge Joint* (SHJ).

The aim of this article is to briefly present an update on the research work recently completed or currently underway on both of these joint details.

With regard to the SFBJ, this will cover;
- results from the component testing to date
- revised joint design requirements.

With regard to the SHJ, this will briefly cover;
- concept behind the joint
- results from component testing
- planned large-scale test.

The Standard Flange Bolted Joint (SFBJ)

First results from 1999 component testing

The SFBJ is intended for low ductility demand applications, with $\mu_{design} = 2$ for MRSF systems using this joint. It will deliver high moment capacity, possesses low design inelastic rotation capacity (ie. up to the point beyond which repair is required) but can sustain significant additional inelastic rotation demand without complete loss of connection integrity and is still readily repairable.

During July/August 1997/98, two full-scale tests on a SFBJ joint were undertaken, from which mathematical models of the moment-rotation capacities of these joints were developed. These mathematical models were then used in time-history analyses of a range of representative MRSFs to determine the system response and inelastic rotation demand on the joints under a range of representative severe earthquake conditions.

Details of these full-scale tests and the resulting preliminary SFBJ design procedure were published in the *SESOC Journal*, Vol. 11, No. 2 [23]. This presented developments through to mid-1998. Subsequent developments in terms of time-history modelling and design procedure development were presented in *DCB* Issue Nos. 47 (December 1998) and 48 (February 1999).

Fig. 48.16, from *DCB* No. 48, shows the general layout of the SFBJ, which comprises separate cleats welded to the column flange and connected by bolts to the beam top and bottom flanges and to the web, with the web bolts arranged in horizontal rows at the top and bottom of the web cleat.

The large-scale tests undertaken in 1998 involved only one combination of bolt size, bolt layout and flange and web cleat thickness. It was considered necessary to test the effects of different cleat thicknesses and sizes, bolt sizes and layouts, prior to developing final design guidance for these joints.

A component test series for this is currently underway. These tests involve just the beam flange to flange cleat part of the joint; ie. the connection between the column and one of the beam flanges. This component is being tested in a purpose built test rig, the set-up of which is shown in Fig. 51.1 of *DCB* Issue No. 51. Fig. 52.4 opposite shows one of the SFBJ specimens mounted in the test rig prior to loading.

In Fig. 52.4, the specimen painted white and mounted on top of the steel reaction column is the cleat being tested. The steel reaction column represents the building's column in practice. The beam flange is represented by a beam flange stub welded to an endplate, which is bolted to the general steel backing plate, which is in turn bolted to the strong-wall on the left. The cleat is held laterally on top of the reaction columns by the square shear key. The dynamic loading actuator can be seen near the bottom right of the picture.

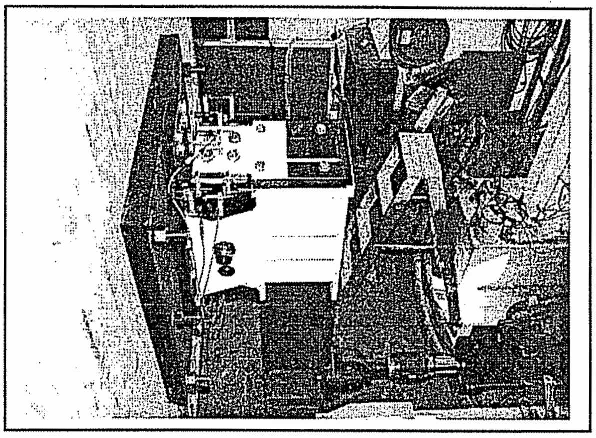


Fig. 52.4
Flange Cleat for SFBJ in Test Rig Prior to Loading

The test rig has been set up to allow for a range of cleat sizes and thicknesses to be tested for each beam flange stub and bolt diameter, by simply unbolting the four vertical restraint bolts at the end of the test, unbolting the cleat from the beam flange stub, then removing the cleat from the reaction columns and replacing it with the next cleat to be tested. A different beam flange stub detail is used for each bolt diameter tested, because of the different bolt spacings.

The aim in the SFBJ component tests has been to subject the flange cleat/beam flange stub to the inelastic axial loading/lateral displacement and curvature that will be encountered in practice.

Initial results have come from the M24 bolts and associated cleats/beam flange stub. These results have shown the following:

(i) The desired mechanism of inelastic response under imposed lateral movement between cleat and flange is for the bolts to force yielding to occur in the cleat, once displacements exceed 2-3 mm. This was the behaviour observed in the 1998 large-scale tests; as reported on page 15 of *DCB* Issue No. 48, the bolts remained upright throughout those two tests, retained significant clamping force and suffered no noticeable damage or permanent distortion even when the joint was rotated through to cleat fracture in tension, as shown in Fig. 48.17, *DCB* Issue No. 48.

(ii) However, the initial component cleats tested in 1999 were 20% wider than the large-scale cleat shown in Fig. 48.17, due to raising the recommended transverse edge distance from $\geq 2d_f$ (as experimentally tested in 1998) to $\geq 3d_f$. This meant that they were also 20% stronger relative to the bolt group shear capacity.

At the time of writing *DCB* Issue No. 48, the author had concerns that this might have increased the ratio of cleat axial capacity to bolt group shear capacity sufficient to change the mode of yielding and fracture from the cleat to the bolts, hence an upper limit on cleat capacity to bolt group shear capacity given by *DCB* Issue No. 48 equation 48.2 was recommended.

(iii) This concern was justified, as shown in the first component tests. In these tests, visible shear yielding commenced in the bolts (ie. the head end visibly moved away from the nut end) after around half the design lateral displacement was reached. From this point onwards, the bolt clamping force rapidly diminished, leading to very pinched hysteresis loops. Failure was through bolt group shear and was sudden and complete.

(iv) When the cleat width was reduced to that shown in Fig. 48.17 from the large-scale tests (the thickness being the same), the behaviour through to failure was exactly as observed in the large-scale tests and described in (i) above.

The appropriate tentative upper limit on flange plate capacity relative to bolt group shear capacity was then determined, from these tests, as being given by equation 48.2, *DCB* Issue No. 48, but with the 1.4 reduced to 1.05.

Further component tests through to the end of the first quarter, 2000, may modify this limit slightly.

The change to this limit has also necessitated changes to the sequence of component sizing and calculation of joint capacity previously published in *DCB* Issue Nos. 47 and 48, plus the *SESOC Journal* Vol. 11 No. 2 [23]. These changes are presented in the next part of this sub-article for the benefit of those currently using the design procedure. Some other shortcomings of the currently-published design provisions are also addressed.

Revised joint (SFBJ) design requirements

The revised requirements are written for application in conjunction with the design procedure in the *SESOC Journal* paper [23]. They also use, where noted, some of the specific additional recommendations for SFBJ design that are given in *DCB* Issue Nos. 47 and 48.

The author appreciates that this requires designers to use provisions from several documents for the joint design, which is cumbersome in practice. However, this is a temporary situation, intended for current and immediate future users of the SFBJ in projects and introduces important changes that have been found to be necessary for overall predicted joint performance. These provisions are to be evaluated once the component test programme is complete and a complete design procedure will then be presented in one document. It is not expected that this procedure will change to any significant extent from that given below.

In the meantime, use the provisions given below in conjunction with those from [23];

1. Changes to detailing requirements for connection components

1.1 Limit on overall beam depth is 1200 mm for the SFBJ.

1.2 Transverse flange cleat edge distance, $e_{dt,fp} \geq 2d_f$ (instead of $\geq 3d_f$), as has been given by section 5.2.1.1 (3.1) of [23].

1.3 Clearance, *f*, between the beam face and column flange is as set by page 14 of *DCB* Issue No. 47.

2. Calculate the element design action reduction factor ϕ_r^* .

This is given by equation 9 from [23], except that $M_{E\mu}^*$ is calculated for $\mu = 2$.

3. Determine the top flange bolts and top flange cleat details required.

3.1 Calculate V_{tfb}^* using equation 10 from [23].

3.2 Make a preliminary estimate of the number and diameter of flange bolts required from equation 52.1 below.

$$\sum \phi V_{bhe,tfb} \geq V_{tfb}^* \qquad (52.1)$$

where:
$\phi V_{bhe,tfb} = \phi V_{bhe}$ from equation 48.1, *DCB* Issue No. 48.

Note also the limit on plate thickness associated with the given bolt size; this limit is given in *DCB* Issue No. 47 and repeated on pages 14, 15 of *DCB* Issue No. 48. A check on this limit being exceeded is made in section 3.6 below.

3.3 Select the flange cleat width required from within the limits of equations 52.2 and 52.3:

$$b_{tfp,min} \geq 4d_f + g_f \qquad (52.2)$$

$b_{tfp,max} \leq 1.05\ b_{fc}$ (52.3)

where:
d_f = diameter of bolt
g_f = bolt gauge (see Fig. 4 of [23])

Select suitable plate width, b_{tfp}. Where possible, use an appropriate flat bar with dimensions meeting sections 3.2 and 3.5, to minimise fabrication cost.

3.4 Determine the required plate thickness from equations 52.4 and 52.5 below. These are equations 27 and 31, respectively, from [23] rearranged to solve for the plate thickness.

$$t_{tfp,tension} \geq \frac{V^*_{tfb}}{0.77\left(b_{tfp} - 2d'_f\right)f_{u,tfp}} \qquad (52.4)$$

$$t_{tfp,compression} \geq \frac{V^*_{tfb}}{0.86 b_{tfp} f_{y,tfp}} \qquad (52.5)$$

where:
all terms are as defined above or in [23].

3.5 Select the thickness of plate such that t_{tfp} ≥ (greater of $t_{tfp,tension}$ and $t_{tfp,compression}$). Where possible, use an appropriate flat bar to minimise fabrication cost.

3.6 Check that the thickness from section 3.5 is within the thickness limit for the proposed bolt diameter from section 3.2 above. If it isn't, increase the bolt diameter (eg. use a smaller number of bolts of larger diameter) so that the limit is satisfied.

3.7 Calculate the design tension capacity of the flange cleat, $\phi N_{t,tfp}$, and the design compression capacity, $\phi N_{c,tfp}$, from equations 27 and 31 of [23], respectively.

3.8 Check that the ratio of bolt group shear capacity to cleat axial capacity given by equation 52.6 below is met.

$$\phi N_{t,tfp} \text{ and } \phi N_{c,tfp} \leq 1.05 \sum \phi V_{f,tfp} \qquad (52.6)$$

where
$\sum \phi V_{f,tfp}$ = $n_{tfb}\ \phi V_{f,tfb}$

n_{tfb} = number of top flange bolts from section 3.2
$\phi V_{f,tfb}$ = design single shear capacity of bolt, from NZS 3404 Clause 9.3.2.1 or as listed in [20].

If this isn't initially satisfied, add an extra pair of top flange bolts and recheck.

4. ***Determine the bottom flange bolts and bottom flange cleat details required.***

Repeat sections 3.1 to 3.8 for the bottom flange; the only difference relates to the initial determination of V^*_{bfb}, which is the greater of V^*_{fb} calculated using equation 10 from [23] and R_{conc}, which is calculated using equation 12.1 from [23].

5. ***Determine the number and diameter of web bolts required, plus web cleat dimensions.***

5.1 Calculate the horizontal design shear force, V^*_{wh}, from equation 52.7.

$$V^*_{wh} = 0.36\phi^*_r A_{wb} f_{ywb} \qquad (52.7)$$

where:
all variables are as defined in equation 14 from [23].

Equation 52.7 is equation 14 from [23] multiplied by 0.8. The 0.8 multiplier has been added, based on parametric studies undertaken using a spreadsheet model of the design procedure, in order to obtain answers for the web cleat and web bolts that are "in proportion" to the requirements for the flange cleat and flange bolts. The physical effect of this reduction factor will be minimal, at the most leading to slightly increased yielding of the web cleat under the design level earthquake.

5.2 Calculate the vertical design shear force, V^*_{wv}, using equation 15 from [23].

5.3 Calculate the resultant design shear force on one row of web bolts, V^*_w, using equation 16 from [23].

5.4 Using the same bolt diameter and web cleat thickness as chosen for the bottom flange, determine the number of web bolts required to satisfy equation 52.8.

$$\sum \phi V_{bhe,wb} \geq V^*_w \qquad (52.8)$$

where:
$\sum \phi V_{bhe,wb} = n_{wb} \phi V_{bhe,wb}$

$\phi V_{bhe,wb}$ = as determined from equation 48.1, *DCB* Issue No. 48.

The above equation is equation 17 from [23] with ϕV_{bhe} substituted for ϕV_{fn}.

5.5 Check that the number of web bolts is within the limits given by equation 52.9.

$$0.5n_{bfb} \leq n_{wb} \leq 0.5n_{bfb} + 3 \qquad (52.9)$$

The lower limit on this equation, in conjunction with the bolt diameter/plate limits of section 5.4 and the web plate edge distance limit of $e_{dt,wp} \geq 2d_f$ are all present to ensure that the row of web bolts has sufficient capacity to force yielding into the web cleat under inelastic rotation, rather than to suffer bolt fracture.

The upper limit is to keep the number of web bolts "in proportion" to the number of flange bolts.

5.6 Apply the result of sections 5.4 and 5.5 to both rows of web bolts.

6. ***Calculate sizes of welds required between the column flange and the cleats.***

This is as given in sections 5.2.1.14 and 5.2.1.15 of [23], except that the check for clearance between the beam end at the flanges and the flange cleat welds given on page 14 of *DCB* Issue No. 47 should be applied.

7. ***Calculate tension/compression stiffener requirements.***

This is as given in section 5.2.1.16 of [23].

8. ***Adequacy of joint panel zone.***

The general philosophy of section 5.2.1.17 of [23] applies, however the actual design provisions are modified to the following:

The out-of-balance panel zone shear force from the joint, V^*_{pz}, is given by equation 52.10.

$$V^*_{pz} = 1.15\left[\max\left(N_{t,bfp}; N_{c,bfp}\right)\right]\left(\frac{d_b}{d_b + t_{bfp}}\right) \qquad (52.10)$$

where:
$N_{t,bfp}$ = (1/0.9) x equation 27 from [1].
$N_{c,bfp}$ = (1/0.9) x equation 31 from [1].

The factor of 1.15 allows for strain-hardening of the cleats under inelastic action.

This expression replaces the term $M/(d_b\text{-}t_{fb})$ in NZS 3404 Eq. 12.9.5.2 (1).

The design shear capacity of the panel zone, ϕV_c, is calculated to NZS 3404 Eq. 12.9.5.3.2.

The panel zone has adequate capacity when $\phi V_c \geq V^*_{pz}$.

Doubler plates, if needed, should be designed and detailed in accordance with the recommendations made on pages 13-16 of this issue. For the weld between the sides of the doubler plate and the column, follow the recommendations given for a WM connection.

Spreadsheet program available

As part of our ongoing SFBJ research development, we have produced two spreadsheets of the design procedure on Microsoft Excel for Office 97. One is for a 10 storey building, the other for a 20 storey building. A copy of each is available free-of-charge on a "use at your own risk" basis. It implements the revised SFBJ procedure as described above.

Those wanting a copy, which will be sent via email, contact Charles Clifton, email address; structural@hera.org.nz

One of the benefits of these spreadsheets is to show the typical bolt sizes and numbers that would be applicable.

Preferred bolt sizes

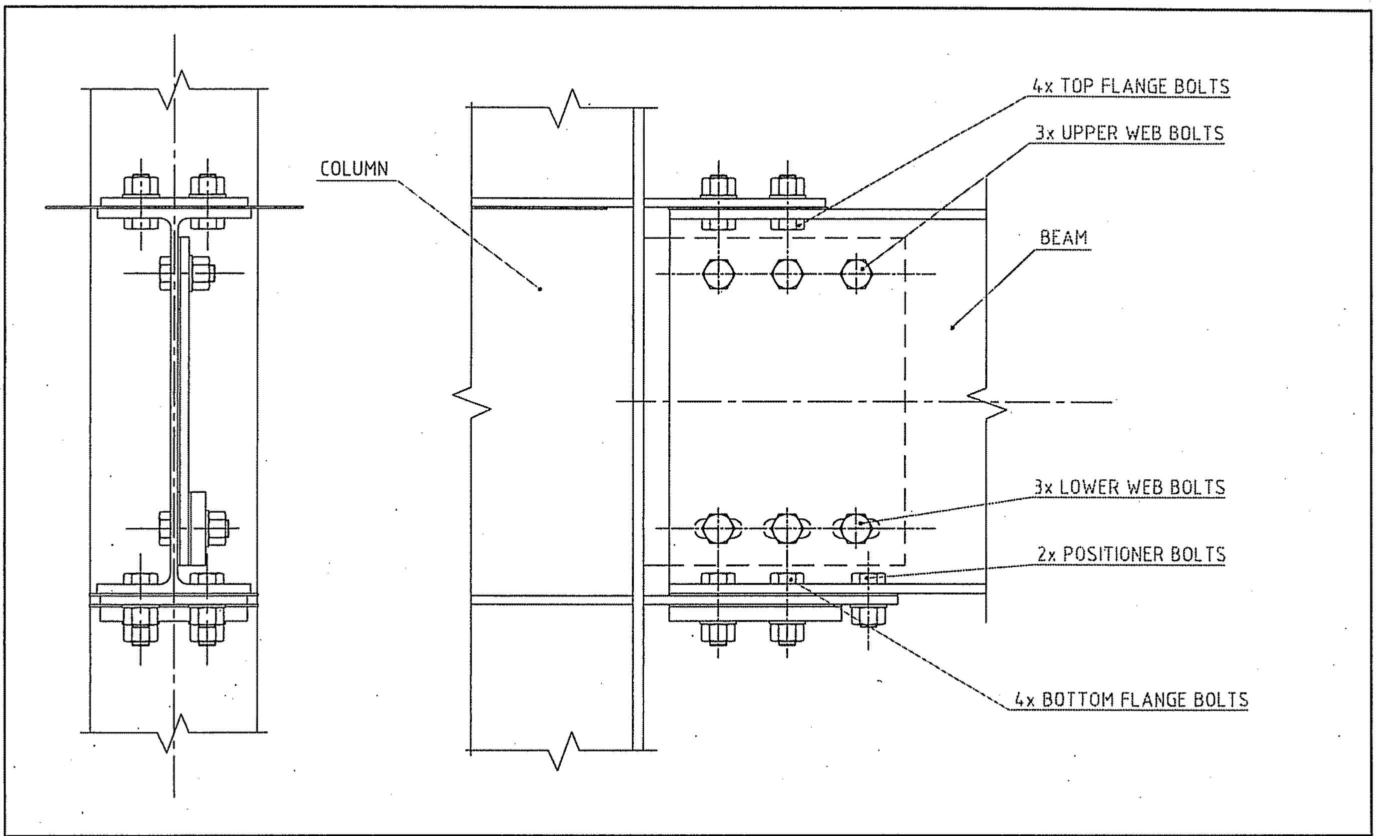
These are M20, M24 or M30 grade 8.8 bolts (ie. bolt, nut and washer sets) to AS/NZS 1252 [7]. M36 bolts should only be used when more than, say, 8 M30 bolts per flange are required. This is firstly because they are more difficult to tighten (although suitable equipment for this is available; see details on page 21 of *DCB* Issue No. 51). The second reason is that they are manufactured differently to the smaller sizes, making them slightly more variable in strength and elongation characteristics.

The Sliding Hinge Joint (SHJ)

Concept behind joint

The second and more advanced semi-rigid joint for the beam to column connections of MRSFs is the Sliding Hinge Joint (SHJ). The concept behind this joint is described below and with reference to Fig. 52.5.

In the SHJ, the beam is connected to the column via a hinge at the top and a sliding detail at the bottom, giving the SHJ its name. The top flange cleat and the top row of web cleat bolts are SFBJ



Sliding Hinge Joint as Developed for Large Scale Experimental Test

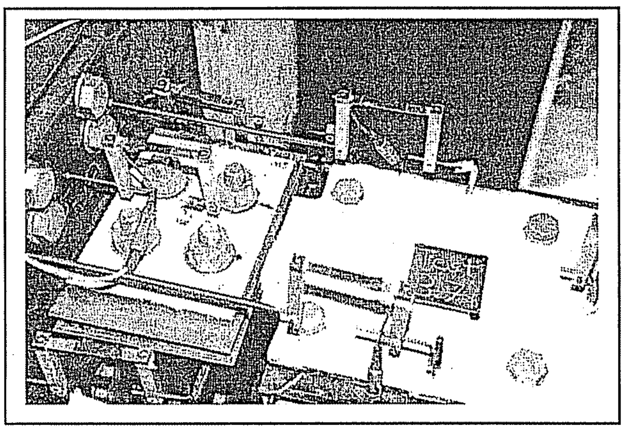


Fig. 52.6
Bottom Flange Cleat and Cap Plate Detail for SHJ Set Up for Component Test

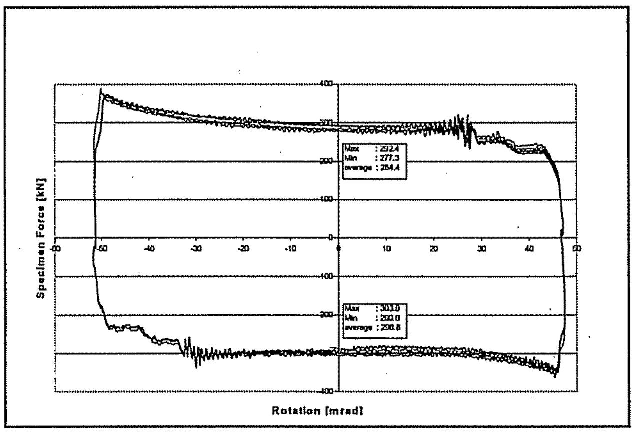


Fig. 52.7
Axial Force in Specimen Versus Rotation Curve Generated By Component Test No. 3.7 Under Design Level Rotation

details, while the bottom flange cleat and the bottom row of web cleat bolts allow sliding between the cleat and beam, with slotted holes in the cleat.

As shown in Figs. 52.5 and 52.6, the sliding detail comprises the following layers; beam flange or web, brass shim, cleat, brass shim and cap plate. The sliding layers are between the brass shims and cleat. Both the beam flange or web and the cap plate have nominal sized holes.

When the moment demand on the joint generates internal beam axial forces which exceed the sliding resistance available through the bottom flange bolts and the bottom row of web bolts, the joint will slide, thus allowing beam rotation to occur. As sliding occurs, the cap plate is anchored in position relative to the beam flange or web by the bolts, allowing the cleat to slide between these two surfaces. Once the imposed moment reduces, there comes a point where the sliding stops and the joint becomes rigid again. This is the behaviour shown in Fig. 52.7.

Results from component testing

Fig. 52.6 shows one of the component tests prior to testing commencing under dynamic rates of loading. The cap plate and flange cleat are visible, as are the two brass shims. In these tests, the painted head of the test rig represents the column flange, while the beam flange stub is attached via bolted endplate to the heavy steel plate on the left and lies underneath the cap plate and brass shims.

Component tests to date have indicated that the condition of stable sliding, without joint deterioration, is achieved even for rotation demand of over ±50 milliradians. The joint is thus potentially suited to fully ductile (μ>3) applications, probably with μ_{design} = 6 x 0.7 = 4.2.

Currently component testing is underway on a range of cleat sizes and cap plate thicknesses, bolt sizes and loading regimes in order to document bolt, cap plate and overall joint behaviour and then to determine a design model.

A simple design model has been developed and finite element analyses have been undertaken.

The design model provides a good prediction of experimentally developed bolt shear capacity and joint shear capacity and is based on a bolt deformed shape and contact points which are in good agreement with that predicted by the finite element analyses.

Planned large-scale test

A large-scale test involving a 530UB82 beam connected to a 610UB101 column and including floor slab and mesh reinforcement is currently in preparation for early February, 2000. It is planned to undertake this test prior to the 12th World Conference on Earthquake Engineering. The inclusion of the floor slab is to (hopefully) show that the joint can undergo imposed inelastic rotation into the fully ductile range without significant degradation, loss of moment capacity or serviceability limit state stiffness and with minimum damage to the floor slab.

From the component testing and large-scale testing, SHJ design and detailing procedures will be developed over the first half of 2000.

Cost-Effective Structural Steelwork: Article No. 52

Optimising The Cost of Multi-Storey Buildings in New Zealand

Background and Scope of Article

The *DCB* Issue No. 49, in April 1999, presented an article on general concepts and detailing issues relating to selecting the structural form for maximum cost-effectiveness in multi-storey steel framed buildings.

This was followed, in *DCB* Issue No. 50, June 1999, by an article on general issues regarding design and detailing of connections for multi-storey steel framed buildings.

Over the last six years, the HERA Steel Structures Analysis Service has been involved, to varying extents, in around 75 multi-storey structural steel building preliminary designs and/or costings. The SSAS, through Clark Hyland, SSAS Manager, has also produced the *Structural Steelwork Estimating Guide,* HERA Report R4-96 [24] and *Structural Steelwork Connections Guide*, HERA Report R4-100 [3].

The undertaking of these activities has produced a reliable, broad database of efficient and effective steel framed solutions, for common types of multi-storey building, which have been accurately and realistically costed. This information has been compiled into an excellent paper [25] entitled *Optimising the Cost of Steel Buildings in New Zealand*.

That paper [25], which is strongly recommended reading for any person involved in the design, fabrication and/or erection of multi-storey steel framed buildings, was written by Hyland and Kaupp for presentation to the Auckland Structural Group in early October, 1999.

The aim of this article is, firstly, to present a brief introduction to Hyland and Kaupp's paper. This will review its scope and content, followed by presentation of some key points and details of the flooring systems covered therein.

The article then goes on to present suggested design solutions, based on SSAS/HERA findings and industry feedback, for common types of multi-storey buildings.

Introduction to Hyland & Tobias's Paper

Scope and content

That paper [25], which is available through HERA (see details on the attached order form), starts by compiling cost data for steel buildings and by presenting this data, along with published cost data from [26] for general building construction.

It then goes on to cover factors in the construction industry marketplace, such as globalisation of steel supply and increased use of computers in more sophisticated analysis, that are reducing the cost of purchasing steel and predicts that these trends are likely to continue.

It then goes on to consider, in turn, each of the following cost-drivers of steel construction, these being:

- Builders P & G rates
- Steel supply
- Connections
- Shop drawings
- Transport
- Coatings
- Passive fire protection
- Erection
- Decking; floor system selection

Key points from the SSAS paper

With regard to preliminary costs of building construction the cost of steel framed buildings is consistently lower than that of buildings using traditional materials based on a widely used Quantity Surveyor's costing guide [26]. For example:

- Medium-rise parking buildings traditionally have an average structural cost (ie. cost for the structural system) of $212/m^2 floor area; an optimum steel solution has a cost range of $150-190/m^2 floor area.
- Hotels traditionally have an average structural cost of $360/m^2 floor area to $464/m^2 floor area; an optimum steel solution has a cost range of over $150/m^2 floor area for 2 storeys up to $315/m^2 floor area for 25 storeys.

There is a very large difference in price between a well thought out steel solution and one not considering economical form and connections. For example, in a 5 storey carpark, the original price of $264m^2/floor area was able to be reduced, on redesign, to ≈ $205m^2/floor area.

Connections cost between 15% and 55% of the cost of the erected structural steelwork, so optimising connection costs is vital.

Steel supply makes up between 30% and 50% of the cost of the erected structural steelwork, so careful selection of steel section types and quantities is essential.

Shop drawings cost between 3% and 20% of the cost of the erected structural steelwork. The advent of full 3-D solid modelling software packages has revolutionised the New Zealand fabrication industry in the last 5 years. Use of standard connections from R4-100 [3] will reduce shop drawings costs.

Appropriate coatings selection for corrosion protection and for passive fire protection, where necessary, is essential to achieving the most economical outcome. Both [24, 25] give good guidance in this regard.

The cost of constructing the concrete deck can range from 40% to 150% of the cost of the steelwork supporting it, so proper selection of decking system is important. Fig. 52.8 shows the different light-gauge steel based flooring systems available in New Zealand. The question of whether or not to prop the supporting deck and/or floor beams for wet concrete placement is also very important. It is covered in [25] and further below; see also the precambering issues covered in page 9 herein.

This article now presents brief details regarding suggested steel design solutions for three common types of multi-storey building.

Suggested Steel Design Solutions for Multi-Storey Steel Framed Buildings

The solutions suggested below are based on the extensive number of steel framed building preliminary design projects undertaken by the SSAS.

They provide targeted recommendations focussed on three types of building. Readers should also

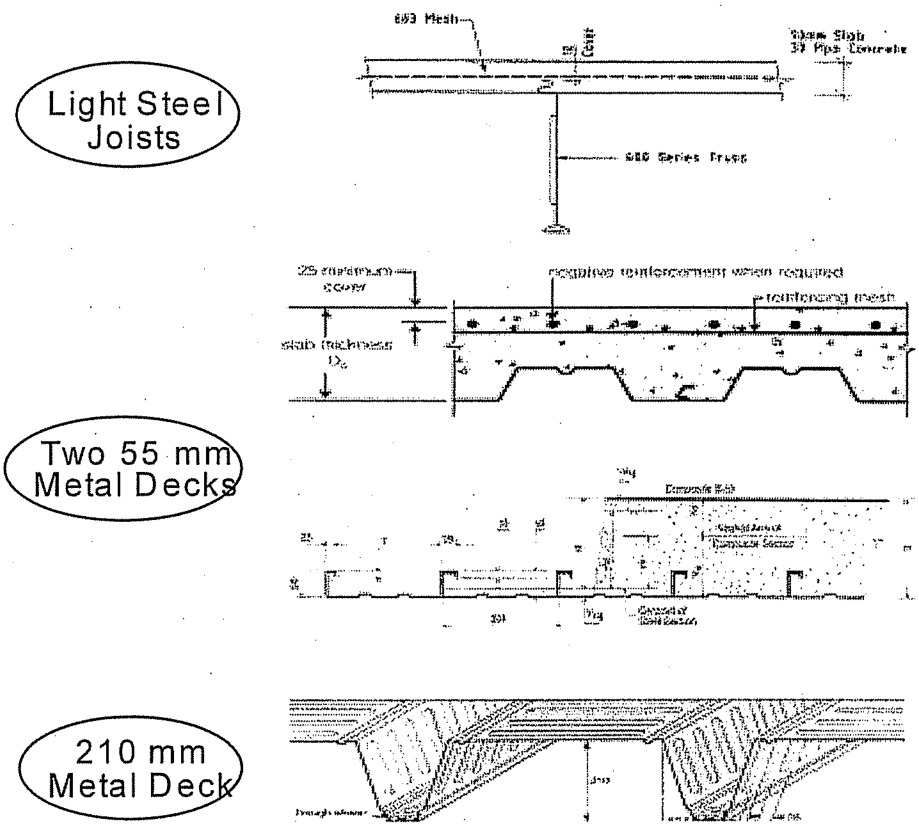


Fig. 52.8
Flooring Systems Typically Used or Proposed For Use in Multi-Storey Steel Framed Construction in New Zealand

Notes to Fig. 52.8

1. The first three systems are made in New Zealand and design guidance for them is available; see references [19, 8, 9] respectively from *DCB* Issue No. 49.

2. The fourth system (ie. the 210 mm Metal Deck) is a UK-developed system, which is currently being introduced into New Zealand. Published design guidance [27] is available from HERA. Specific design assistance and details on availability can be obtained through Corus (formerly British Steel). Contact:
   Stephen Stickland
   British Steel New Zealand Ltd
   Phone: 0-9-634 1179
   Fax: 0-9-634 2901

follow the general recommendations on selection of structural form and detailing for maximum cost-effectiveness given on pages 20-24 of *DCB* Issue No. 49.

For parking buildings

Deck
- light steel joist; unpropped; 90mm topping
- a 250mm deep joist for 5.2 m span; or
- 55 mm metal deck; 2.8 mm span; 120 mm thick; unpropped

Floor beams
- precambered, welded beams; < 16m span; unpropped
- unprecambered, hot-rolled beams; < 9m span; unpropped
- all connections WP or FE to R4-100 [3].

Gravity columns
- 3 to 4 storey lifts
- UC or CHS/RHS

Seismic-resisting system
- Category 1 ($\mu = 6$) eccentrically braced frame (EBF)

Coatings for corrosion
- 75 μm Inorganic Zinc Silicate
- 50 μm topcoat for colour if required

- refer to pages 7-14, *DCB* Issue No. 49, for comprehensive details relating to durability of parking buildings

Fire protection
- none; beams and columns to have 15 mm FRR
- refer to pages 4 and 5 of *DCB* Issue No. 42

For apartment/hotel buildings

Deck
- propped 55 mm metal deck; < 4.5 m span; 120 mm thick for trapezoidal decking, 100 mm thick for flat base decking
- propped 210 mm metal deck; < 8.0 m span; 280 mm thick

Floor beams
- unprecambered, hot-rolled sections; < 9.0 m span; propped
- all connections WP or FE to [3]

Gravity columns
- 3 to 4 storey lifts
- UC or CHS/RHS

Seismic-resisting systems
- category 1 EBF; or
- category 2 (μ = 3) perimeter moment-resisting frame (PMRF) (perimeter means supporting a greater tributary area for lateral loading than for gravity loading, although these frames will also usually be located on the building perimeter)

Coatings
- no coatings for corrosion protection or for appearance are required to steel elements within the building envelope and hidden from view.

Fire protection
- beams and columns are hidden behind plasterboard linings
- use sprinklers as first line of defence against structural damage
- use the radiation shielding from the linings in a fire engineering design to meet requirements for stability under firecell burnout. This method is described in [28]. No additional passive fire protection is used.

For office buildings

Deck
- 55 mm metal deck; 2.8 m span; 120 mm thick; unpropped
- 210 mm metal deck; < 8.0 m span; 280 mm thick; propped

Floor beams
- precambered, welded beams, unpropped with 55 mm metal deck
- precambered, hot-rolled beams up to 410UB54, unpropped with 55 mm metal deck
- unprecambered hot-rolled beams or welded beams, propped with 210mm metal deck
- all connections WP or FE to [3]

Gravity columns
- 3 to 4 storey lifts
- UC or CHS/RHS

Seismic-resisting systems
- category 1 EBF; or
- category 2 PMRF

Coatings
- no coatings for corrosion protection or for appearance required to steel elements within the building envelope and hidden from view.

Fire protection
- currently 30 min to 60 min FRR typical
- passive fire protection needed, especially for 60 min FRR
- use sprinklers as first line of defence against structural damage; this also can allow a reduction in the rating required for unsprinklered buildings in most instances
- research is leading towards general use of unprotected beams and protected columns in sprinklered buildings. An example of this research is given in *DCB* Issue No. 47.

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	for fire emergency conditions	$ 45.00	$ 50.00	$ 60.00		
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No.52
October 1999

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