STEEL DESIGN & CONSTRUCTION BULLETIN
DCB

HERA

17-19 Gladding Place
Manukau City

P O Box 76 134
Manukau City, New Zealand
Phone: +64-9-262 2885
Fax: +64-9-262 2856
Email: admin@hera.org.nz

No. 51 **AUGUST 1999**

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

For those of you wondering why your August *DCB* has reached you in September - no, it wasn't held up in the mail. It's running late, for which I apologise. It's also slightly shorter than the previous issue, which at 27 pages ended up longer than intended.

This issue has two general themes, namely:

- material/guidance/amendments relating to relevant standards
- design guidance utilising publications available from HERA, including two design examples.

One item not now being covered in this issue, despite the indication given in *DCB* No. 50, is the next part of the cost-effective structural steelwork series on connections. That is being held over to the next issue. The MEP article and two design examples presented herein offer a cost-effective connection option for moment-resisting steel framed seismic-resisting systems that has not been properly explored to date.

Fire Engineering Design: Three Issues Relating to the June 1999 Draft Approved Documents for Fire Safety

Introduction and Background

All readers with an interest in fire engineering design will know that the *Draft Approved Documents for Fire Safety* [1] were released in late July for review, with the closure dates for comments being 17 September, 1999. This article is not intended as a "call to comment", as it will not be released until after the closure date. Rather, it is intended to address three issues relating to the new draft which are directly relevant to the fire safety design of structural steelwork.

These issues relate to:

(i) Modifying the structural fire severity (S) rating to take account of the thermal properties of the materials of construction of the firecell (enclosure) under consideration.
(ii) Application of the "collapsed wall condition" as described in *DCB* Issue No. 20, in accordance with the new Appendix C to the Fire Safety Annex, [1].
(iii) Fire resistance ratings required for building elements in steel framed car parks.

The purposes of this article are to:

- provide more background to important aspects relating to use of the new drafts
- to provide background to potential areas of proposed change from the current provisions [2] to the new provisions [1]
- to link the advice with that already given in past *DCB* issues

In This Issue

- to allow engineers to vary the proposed new provisions by specific FED, in order to obtain the most cost-effective structural steel solution without compromising the *Building Code* [3] or the owner's own property protection requirements.

Each issue is now dealt with in turn, covering the following:

- background to the provisions of [1]
- application in accordance with [1]
- application in a specific FED.

Modifying the S Rating to Account for the Thermal Inertia of the Bounding Elements

Background

The revised structural fire severity (S) ratings for unsprinklered firecells (ie. the t_e values) in the draft *Approved Documents* are given in Table 1 of [1]. The proposed new Table 1 values of t_e are 1.34 times the current values [2], rounded to the nearest 10 minute interval.

According to the explanation in [1], the increase has come about because of the need to provide an increased safety factor, to overcome deficiencies in the design formula, and the need to take account of greater structural fire severity when fires occur in firecells constructed of less thermally conductive materials than those used to develop the current values in Table 1 of [2].

Of these two reasons, only the latter one has any relevance in regard to steel structures. There is no evidence that the current S ratings formula is inherently unconservative; both large-scale UK fire tests [4] and application to other fire tests, eg. as reported in [5], show that the current equation gives realistic time equivalent values (which are equivalent to the S rating for unsprinklered firecells) for application to structural steel elements.

The expression for t_e is given [6] as:

$$t_e = e_f k_b w_f \qquad (51.1)$$

where:

e_f = Fire Load Energy Density (MJ/m^2 floor area)

k_b = thermal inertia factor for the bounding elements

w_f = factor to account for the available ventilation; see eg. equation E.3 of [6] or equation 6.4 of [7]

In Table 1 of [2], k_b = 0.067. This corresponds to an enclosure/firecell with the ceiling, walls and floor built of moderately conductive materials, such as normal weight concrete. The material of construction that most influences the structural fire severity is that on the surfaces of the bounding elements of the enclosure/firecell. Furthermore, the influence of the ceiling, walls and floor vary according to enclosure size and shape. For practical design purposes, the thermal inertia of the ceiling contributes 50% of the thermal inertia of the enclosure, the thermal inertia of the wall contributes 40% of the thermal inertia of the enclosure and the floors contribute 10%.

In a multi-storey (steel framed) building with fire rated timber floors and dry wall construction, the floors will be fire separations and the ceilings will comprise fire-resistant plaster board. Walls that are not fire separations will incorporate standard plaster board, while walls that are fire separations will incorporate fire-resistant plaster board. Thus a firecell (storey) in such a building could well have a fire-resistant plaster board ceiling, ordinary plaster board walls and a timber floor. This compares with a firecell (storey) in a typical multi-storey steel framed building with composite floors incorporating normal weight concrete, which will have normal weight concrete ceiling and floor and, typically, ordinary plaster board walls.

Values of thermal inertia for common materials of construction are given in Appendix B of [6]. These values can be used, in conjunction with the above breakdown of the relative importance of ceiling, walls and floor to the overall thermal properties of the firecell, to determine the differences in thermal inertia between the two enclosures illustrated above. The more insulated enclosure has a thermal inertia, $b_{enclosure}$ = 675 J/m^{2}s$^{0.5}$C while the typical enclosure has a thermal inertia of 1740 J/m^{2}s$^{0.5}$C. (The greater the value of thermal inertia, the more heat that can be absorbed into the surface from the fire).

As one would expect, there is a relationship between the thermal inertia of the enclosure and the structural fire severity. For the same fire load and ventilation conditions, the structural fire severity (t_e) will increase with decreasing thermal inertia. This effect is embodied into equation 51.1, has been demonstrated experimentally [4] and has been observed in the analyses being undertaken as part of HERA's long-term fire research programme (eg. as reported in *DCB* Issue No. 48).

The large enclosure test series reported in [4] were undertaken to test the adequacy of the time-equivalent equation 51.1, at least with regard to structural steel systems. A summary of these tests is given in *DCB* Issue No. 6, December 1994. The relevant conclusions from these tests, as reported on page 6 of *DCB* Issue No. 6, were that:

(1) The equation (ie. equation 51.1) is a valid and sufficiently accurate design tool

(2) The values of k_b should be modified to give:

k_b	=	0.09	for enclosures with a low value of thermal inertia (ie. the example given above with plaster board lined ceiling and walls)
k_b	=	0.045	for enclosures with a high value of thermal inertia (eg. enclosures with a sheet steel roof)

As previously mentioned, the values of t_e given in the current C3/AS1 Table 1 [2] are derived using equation 51.1 with k_b = 0.067. In the revision [1], it has been decided to increase them to the value applicable to a low thermal inertia enclosure, thus giving a multiplier of (0.09/0.067) = 1.34. For enclosures lined with material with higher thermal inertia, the values of t_e can and should be reduced in accordance with specific fire engineering design (FED).

There should be a note to this effect in the Notes for Table 1 of C3/AS1[1]. It is anticipated that this note will be added prior to its final publication.

The HERA Structural Engineer was on the BIA working group which formulated a methodology for adjusting the value of t_e to take account of firecells built using materials of construction with different values of thermal inertia. How to achieve this is given below.

Adjustments to t_e values in proposed C3/AS1 Table 1 to account for thermal inertia differences in the firecell linings

The tests undertaken by Kirby et.al. [4] have given benchmark relationships between the thermal inertia of the enclosure, ie. $b_{enclosure}$, and the thermal inertia factor, k_b, as used in the equation for t_e (ie. equation 51.1 herein). These relationships are b = 712 J/m^{2}s$^{0.5}$C and k_b = 0.09 for an insulated enclosure (low thermal inertia); b = 2500 J/m^{2}s$^{0.5}$C and k_b = 0.045 for a conductive enclosure (high thermal inertia). A straight line relationship between these points is given by equation 51.2;

$$k_b = 0.108 - 2.528 \times 10^{-5}\, b_{enclosure} \qquad (51.2)$$

This is the appropriate equation to use when making adjustments directly to the time equivalent equation itself - ie. to equation E1 from [6], also given as equation 6.3 in [7].

When this equation is normalised to a baseline value of k_b = 0.09, the resulting equation is the adjustment factor needed to adjust the proposed t_e values in the draft C3/AS1 Table 1 [1] for different values of k_b. This adjustment factor, k_a, is given by equation 51.3.

$$k_a = 1.2 - 2.8 \times 10^{-4}\, b_{enclosure} \qquad (51.3)$$

where:

k_a = multiplier on the t_e values from [1] for different values of $b_{enclosure}$

$b_{enclosure}$ = thermal inertia for the enclosure (firecell)

Limits on k_a are $1.0 \geq k_a \geq 0.50$.

Using the contribution given on page 2 herein to $b_{enclosure}$ from each of the walls, ceiling and floor and the values of thermal inertia for typical enclosure lining materials given in Annex B of [6], it is straight-forward to derive values of $b_{enclosure}$ to use in equations 51.2 or 51.3 for a range of common enclosure materials used in steel framed building construction. These values are given in Table 51.1.

Nature of Enclosure Construction	$b_{enclosure}$ (J/m^{2}s$^{0.5}$C)
Fire-resisting plaster board lined ceiling, plaster board lined walls, timber floor	712
Normal weight concrete ceiling and floor, plaster board lined walls	1700
Lightweight concrete (density ≈ 1500 kg/m^3) ceiling and floor, plaster board lined walls	1100
Sheet steel roof, with or without building paper and any wall construction	2500

Table 51.1
Recommended Enclosure Thermal Inertia Values for Fire Engineering Design

Note: Ignore the presence of suspended ceilings unless they are designed to remain in place for a specified minimum time under fully developed fire conditions.

Application of the Collapsed Wall Condition in Accordance with the Revised Enclosing Rectangles Provisions for Fire Separation

Background to relevant changes in the *Acceptable Solutions* [1]

The principal method in the current *Acceptable Solutions* [2] for determining the fire separation requirements between a building and a relevant boundary is the "enclosing rectangles" method given as Method 4 in Appendix C of Annex/AS1.

This is based on the "mirror image" method, which assumes that the receiving body is the same distance away from the relevant boundary as the emitting firecell. The requirement is to limit the radiation received by the receiving body to specified values, using an assumed set of conditions in the emitting firecell.

The revised "enclosing rectangles" method is now presented as Method 2 in the draft [1]. It is a complete revision of procedure and philosophy, now based on limiting the radiation received from the emitter at a point 1.0 m across the relevant boundary to specified levels associated with combustible surface finishes. The limits on received radiation and the conditions within the emitting firecell have been revised to reflect the current state of fire engineering research. Maximum limits have also been put on the width of emitter that needs to be considered for the different fire hazard categories (FHC).

Background to the *collapsed wall condition* design method

When a fire rated concrete external wall panel is supported off a steel portal frame, 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 that this might be necessary relates to prevention of fire spread across the boundary.

For FHC 3 and FHC 4 buildings, the fire is likely to cause significant distortion of the steel frame. If the columns are not protected, then there is a possibility that the column will be pulled inwards. A fire in a FHC 3 or FHC 4 building is also likely to be severe enough to cause the concrete wall panels to fall outwards and this must be prevented by tying the panels along their tops into the steel frame, using a suitable detail such as that given in section 4.5.1 of HERA Report R4-91 [5]. This means that, if the columns are unprotected and pulled inwards by the fire, then the external wall panels will also be pulled inwards. If the wall completely collapses inwards, then the fire front will be pushed back from the boundary by a distance equal to the height of the wall and there will then be 100% openings at the new position of the fire front.

The philosophy behind the *collapsed wall condition* is to determine, from the "enclosing rectangles" method of Annex/AS1, whether 100% openings are allowed at the position of the collapsed wall fire front. If so, then wall collapse can be permitted. If not, then the columns must be protected to dependably keep the wall upright during the fire.

Details of the method were first presented in *DCB* Issue No. 20. It aroused considerable interest from fire engineers, who questioned aspects of it. These questions were addressed in followup articles in *DCB* Issue Nos. 21 and 22. The final method has been published in *DCB* Issue No. 22 and again in section 4.5.2 of [5].

The revisions to the "enclosing rectangles" method given in [1] make not only that procedure more logical and dependable, but also the application of the *collapsed wall condition* method. This is because of the change of philosophy from mirror image method to limiting radiation method - ie. the limit on radiation to be met at specific points across the relevant boundary remains the same under the new exposure conditions when the *collapsed wall condition* is checked.

Furthermore the revised "enclosing rectangles" method now recognises that there is a limit to the width of emitter needed to be considered in fire spread across the boundary calculations, which is consistent with the approach taken in the *collapsed wall condition* method.

It was foreshadowed, on page 11 of *DCB* No. 50, that the *collapsed wall condition* method would become redundant when the new "enclosing rectangles" method was published. That isn't the case; there is still some benefit in applying it and its application is much more straight-forward under the new provisions [1]. Details of how to apply it are as follows:

Application of the *collapsed wall condition* method under the proposed new "enclosing rectangles" provision [1] for fire separation

(1) For FHC 1 and FHC 2 buildings

No special fire-resistant design and detailing of the steel members or of their connections to the wall components is required.

(2) For FHC 3 and FHC 4 buildings

Apply the provisions in section 4 of R4-91 [5] relating to column base detail, design and detailing of eaves members and connection of fire-rated wall components to the eaves members.

In determining whether the supporting column needs passive fire protection, in unsprinklered buildings with external walls approximately parallel to the relevant boundary, apply the *collapsed wall condition* as follows:

Step 1: Determine the distance between the "collapsed wall fire front" and the relevant boundary.

This distance equals the distance from the relevant boundary to the external wall, plus the height of the wall.

Step 2: Determine the height of the enclosing rectangle.

This is the original height of the wall; use the average height where this varies.

Step 3: Determine the width of the enclosing rectangle, b_{fire}.

For FHC 3, b_{fire} = 15 m.

For FHC 4, b_{fire} = 15 m + 5 m for every 500 MJ/m^2 floor area of fire load above 1500 MJ/m^2, up to the maximum width requiring consideration, from [1], for FHC 4 of 30m.

These values are taken from [5] and updated to tie in with the maximum enclosing rectangle widths needing consideration as specified in [1]. The values are applicable to the fire after it has been burning for a sufficient period of time to cause collapse of the external wall, hence are typically less than the maximum widths required from [1].

Step 4: Determine the percentage of unprotected area allowed in the collapsed wall condition.

This involves use of the new Tables C2 from Appendix C of [1]. Go into these tables for FHC 3 and 4, using the distance to the relevant boundary from step 1, the rectangle height from step 2 and the rectangle width from step 3.

Step 5: Determine whether the columns need a fire resistance rating.

If the permitted unprotected area from step 4 = 100%, no fire resistance rating and hence no passive fire protection is needed to the columns.

If the permitted unprotected area from step 4 <100%, the columns need a fire resistance rating. Apply the UK method, as given in section 4.5.2.2A of [5], to achieve this.

For sprinklered buildings, the HERA Structural Engineer recommends that the columns do not need a fire resistance rating and hence passive protection, provided that the connection detail at the column base incorporates a four-bolt hold-down detail. This can be either as recommended by section 4.5.3 of [5] or as recommended by R4-100 [8] for I-sections with a depth > 290 mm (see page 257 of [8] for this detail). The reason for this recommendation rests with the results of advanced analyses and actual fire case histories, as reported in [5], which show that columns with the base rotational stiffness afforded by this detail are expected to remain vertical or near-vertical with or without applied passive fire protection. Thus the probability of the sprinklers failing to suppress the fire and the columns collapsing inwards and pulling the wall inwards and this resulting in fire spread across the relevant boundary are sufficiently small to not require consideration in design.

As a final note, the HERA Structural Engineer would strongly recommend that designers use the draft enclosing rectangles provisions from [1] instead of the current provisions from [2]. This is because of their much sounder basis. The *collapsed wall condition* method can then be readily applied in conjunction with these new provisions. However, designers will find that, in many instances, the result from step 5 above will be less than 100% openings allowed under the *collapsed wall condition* and hence the columns of these unsprinklered single storey buildings will require passive fire protection.

FRRs of Structural Elements in Steel Framed Car Parking Buildings

Actual requirements

In terms of meeting the requirements for maintaining structural stability of the building, the structural steel elements should have a 15 min. fire resistance rating (FRR). This advice comes from an article on *Fire resistance rating requirements for steel beam and column members within car park floors*, presented on pages 4 and 5 of *DCB* Issue No. 42.

In practice, provision of this level of FRR will mean that structural elements in open car park buildings will be undamaged in the event of a car fire. In closed car parks with small floor area, some permanent distortion of elements directly exposed to the car park fire might occur but collapse or loss of integrity won't occur.

The recommendation from *DCB* Issue No. 42 was for a 15 min. FRR for the structural elements and 30 min. FRR for fire separating elements, eg. floor slabs and wall elements.

Revised requirements from the draft *Acceptable Solutions*

The current Acceptable Solutions [2] present the requirements in terms of a reduced *S* rating. As advised in *DCB* Issue No. 42, this produces results that are at variance with observation and fire engineering design.

The revised Acceptable Solutions [1], in Para. 2.12.5, have simplified the requirements to an F30 rating, reduced to F15 if sprinklered.

The *F* rating is a life safety rating and, by definition, an element with an F rating need not survive a full design fire; it only has to survive for the minimum specified period of fully developed fire exposure. However, the above requirements will be sufficient for the structure to survive a design car park fire, typically with no damage. As such, they should be designated *S* ratings.

Recommended changes to the draft *Acceptable Solutions*

(1) Reclassify the ratings as *S* ratings.

(2) The 15 minute rating should apply to open car parks as well as to sprinklered car parks. This would require a definition for *open* and a suitable definition could be:

An *open* car park is one with at least the following extent of openings in the external walls on any floor:

(i) ≥ 50% of the wall area open to the environment in each of any two opposing perimeter walls; or

(ii) ≥ 50% of the total perimeter wall area open and these openings distributed uniformly along at least 50% of the total length of perimeter wall.

(3) Fire separations should maintain a 30 minute FRR so as to provide protection against a more severe fire in the adjacent firecell. This is particularly important for a floor fire separation between a car park firecell below and another firecell above, because the floor is fire rated from the underside.

Useful source of further information

BHP Australia has published an interesting booklet entitled *Economical Carparks - A Guide to Fire Safety* [9]. The publication contains the following sections:

- introduction
- BCA deemed-to-satisfy provisions
- fire engineering approach
- experimental basis
- research outcome, references.

The principal purpose of the publication has been to show what options for the use of unprotected steel are already permitted in Australia, under their deemed to comply, prescriptive *Building Code of Australia* (BCA) and to show how better solutions can be achieved by applying rational fire engineering design.

Because our performance-based NZBC [3] requires us to follow the rational FED route directly, the chapter dealing with the BCA deemed to satisfy provisions is not relevant to New Zealand. However, content in the other sections makes interesting reading, such as:

- the introduction contains a good pictorial description of some recent unprotected steel framed car parks built around Australia and New Zealand.
- the chapter on fire engineering provides a useful summary in tune with the above recommendations for New Zealand.
- the chapter on the experimental basis is particularly interesting, as it summarises the comprehensive car park burn-out test programme recently undertaken by BHP.
- a comprehensive list of references is included.

To obtain a copy, see details on the attached order form.

Design and Durability of Steel Bridges: Three Useful Design Publications are Available

General

Structural steel bridge beams are an increasingly attractive solution when compared with reinforced concrete bridge beams, provided that the bridge design solution using steel beams is efficient and cost-effective, and that an appropriate solution for achieving durability is specified.

This article briefly introduces three publications [10-12], all available from HERA, that will assist in obtaining cost-effective answers in regard to both design and durability. Before commencing with these introductions, designers need to also remember that preliminary costing of any bridge designs should use the rational costing method, as is given in HERA Report R4-96 [13] *Structural Steelwork Estimating Guide*.

When using [13] for costing of bridges, the section on erection costs will need some modification for application to bridges instead of buildings. Instead of building height, the ease of access on site will influence the erection cost. Advice from a fabricator/erector should be sought. In terms of comparing steel bridge beam costs with reinforced concrete bridge beam costs, it is important to note that generally the erection price of concrete bridge beams will increase more rapidly than that of steel bridge beams as the site access becomes more difficult.

Now, back to the three publications available from HERA, starting with that for design.

Composite Steel Road Bridges: Concepts and Design Charts

Scope and applicability to New Zealand

This publication [10], which has been published by BHP Integrated Steel Products, Australia, has been prepared to assist bridge designers with the preliminary design of economical composite I-section steel bridges using section 6 of the 1996 Australian Bridge Design Code [14].

The design charts contained in [10] will enable rapid structural design of bridge superstructures to be carried out with adequate accuracy for preliminary cost estimating. Brief guidelines are given on a number of aspects which influence the economics of steel bridges. Information on available design aids, references for further advice and BHP technical support, all of which are available through BHP New Zealand Steel, are included in [10].

The charts can be used for the design of composite steel I-section girder and reinforced concrete deck road bridges with spans up to 60 metres. Typically these bridges would be used for motorways, highways or main roads where the bridge solution with the lowest cost is usually adopted.

The charts include standard BHP Universal Beams and Welded Beams for spans up to 30 metres and custom made three-plate girders for spans in the range of 20 to 60 metres.

The design solutions contained in [10] are directly applicable to bridge superstructure design for New Zealand. With regard to strength, the relevant limit state equation to be resisted is:

$$S^* \leq \phi R_u \qquad (51.4)$$

where:
S^* = limit state design actions
ϕR_u = limit state design capacity

The loading and load factor requirements of [14], for calculation of S^*, are slightly more conservative than the Transit Bridge Manual [15] requirements for New Zealand. The composite design provisions of [14] are very similar to those of NZS 3404 [16] and, where differences occur, the NZS 3404 provisions are slightly less conservative. Hence, from a strength requirement, design solutions presented in [10] will satisfy New Zealand conditions, except for specific seismic considerations. However, seismic design requirements impact on the supporting system, rather than on the bridge superstructure, and the only seismic design aspects that will require consideration and are not covered in [10] relate to detailing of the superstructure to supporting structure supports and the bearings.

With regard to non-seismic limits on stiffness, the provisions of [10] will be adequate for New Zealand applications.

Contents in more detail

There are 11 sections in [10] of direct relevance to New Zealand application. These are:

Section 1: General
- covers scope and notation.

Section 2: Preliminary Design
- this is an excellent section, covering choice of material and layout, steel grades and sections
- important requirements of the Australian design code [14].
- Table 2.1 gives an excellent, comprehensive listing of the advantages of steel beams for bridges over reinforced concrete beams.

Section 3: Design Charts for BHP Standard I-Section Bridges
- design charts are given for bridge spans of 10 to 35 metres and for girder spacings of 1.7 metres to 3.5 metres
- information is given on use of the charts for preliminary design and on optimising the design for maximum economy
- worked examples are given.

Section 4: Design Charts for Multiple Plate Girder Bridges
- similar scope of coverage to section 3, for multiple plate girder bridges with spans of 20 to 60 metres.

Section 5: Design Charts for Twin Plate Girder Bridges
- similar scope of coverage to section 4, for bridges comprising two girders only with spans of 20 to 60 metres.
- guidance is given on when to use which option (ie. multiple plate girders or twin plate girders) for maximum economy.

Section 6: Bridge Systems and Details for Economy
- this section gives guidance on optimising the layout and detailing for economy
- it covers jointed and jointless bridges
- it covers issues such as camber, bearing design, deck slab construction.

Section 7: Specifications for Steelwork Fabrication

- this section gives some general information on appropriate specifications for Australian use
- however, in New Zealand, the appropriate specification is the *HERA Specification for the Fabrication, Erection and Surface Treatment of Structural Steelwork* [17].

Section 8: Surface Protection

- this section simply refers to the BHP *Coating Guide for New Steel Bridges* [11], which is the second of the three publications being introduced in this *DCB* article.

Section 9: Safety - Tips for Design and Construction

- this section provides a one page table with safety tips and points of particular emphasis to consider with regard to each stage of the steel bridge fabrication and erection.

Section 10: Estimating Bridge Construction Costs

- this section highlights the importance of deriving accurate cost estimates and makes reference to the "rational method" of costing developed by BHP
- for New Zealand application, the appropriate rational costing method is that contained in R4-96 [13]; see further details under *scope and applicability to New Zealand* on page 7 above.

Section 11: References

- presents a comprehensive reference listing relating to steel bridge design, construction.

Coatings Guide for New Steel Bridges

The choice of surface coating for the protection of steelwork depends on the macro atmospheric environmental conditions at the bridge site and the micro environment at particular locations of the steelwork in the bridge.

Assessment of both these factors for sites throughout New Zealand is straightforward. Details are given in *DCB* Issue No. 46, pages 5-7. These in turn refer to an excellent paper [18] by Hyland and Enzensberger, which presents the relationship between the first year corrosion rate and readily available site location and meteorological data.

Having assessed both these factors, excellent guidance on the selection of coatings is given in the *Coatings Guide for New Steel Bridges* [11].

This document provides guidelines for the very long term protection of new steel bridge superstructures including I-girders, troughs, box girders, trough-girders, trusses and all parts of the substructure subject to atmospheric exposure. The assumed design life of the bridges is 100 years.

The Guide [11] includes:

- identification and classification of environments prevailing in Australia and New Zealand, as well as the micro-environments around bridges and their likely effect on coating performance (read that in conjunction with *DCB* Issue No. 46)
- details of surface preparation and coating systems, including application guidelines for shop and site conditions
- data on performance of coatings including their life expectancy
- bridge examples with estimated maintenance schedules over the assumed 100 years design life
- methodology for the calculation of maintenance costs using Net Present Value Analysis
- estimates of initial and life-cycle costs, based on the above maintenance schedules, to assist in the selection of appropriate coatings in various environments
- specification types, summary of advantages and disadvantages
- an overview of the process of application contractor selection, project management, contract administration and options for contractual arrangements
- guidelines for quality assurance, inspection requirements and health, safety and environment protection regulations.

The Guide is generally aligned to AS/NZS 2312 [19] with one important difference: it takes into account the findings of recent research carried out by the steel industry on bridge coatings, which illustrates the excellent performance of single coat Inorganic Zinc Silicate (IZS) coatings in most environments. This coating has a long track record of excellent performance and reduces both initial and maintenance costs significantly. A detailed article on how IZS paints operate, their benefits and the different IZS systems available is contained in *DCB* Issue No. 41, pp. 1-5.

Durability of Steel Bridges: Coatings System Performance

The third publication introduced in this article is SCI Technical Publication No. 241 [12], *Durability of Steel Bridges: A Survey of the Performance of Protective Coatings*.

This publication is not intended for the same direct design application, within the New Zealand environment, as are the two previous publications introduced herein. Rather, it provides interesting details on the actual performance of different corrosion protection systems in the United Kingdom environment. Given that the UK environment is typically more corrosive, for a given rainfall, temperature, and distance from the sea coast, than is the New Zealand environment, the findings from [12] are of interest and some relevance to New Zealand.

However, one significant omission from [12] is the recommended use of single coat IZS systems. This is a major omission, as these systems can offer equal or superior performance at lower cost than the multi-coat recommendations of [12].

For those readers who are heavily involved in corrosion protection of steel bridges and/or want some guidance on the expected actual performance of installed multi-coat systems, this SCI Publication [12] contains useful material and is available on loan from HERA.

NZS 3404: Two Areas of Amendment Required

Introduction

This article deals with two separate areas in which the current provisions of NZS 3404:1997 [16] require amendment. Both these items will be included in the Amendment No. 1, which is currently in preparation and proceeding slowly.

The first deals with design of the bearing surfaces of a ply which are supporting a pin in bearing, for which the relevant clause is Clause 9.5.4.

The second deals with member restraint requirements for category 1, 2 and 3 members, as covered by Clause 12.6.2.

In each case, the background is given, followed by the recommended changes.

Bearing Surfaces of a Ply Supporting a Pin

Background

As has been previously reported, HERA is engaged in two long-term research projects related to the seismic design of steel structures. The first is the development of new semi-rigid joints for moment-resisting steel frames (MRSFs) and the second is the influence of weld quality, weld strength and weld type (butt weld versus fillet welds) on the seismic performance of welded joints. Examples of reports on the former project are given in *DCB* Issue No. 48, while findings from the latter project are incorporated into recommendations on designs of connections subject to potential inelastic demand given on pages 21, 22 of *DCB* Issue No. 50.

For both research projects, the research being undertaken during 1999 is involving the testing of joint components under both pseudo-static and seismic-dynamic rates of loading. These tests are being undertaken at the University of Auckland, Department of Civil Engineering Dynamic Testing Facility.

For the semi-rigid joints, one of the components being tested is the beam flange to flange plate connection for the standard Flange Bolted Joint - ie. the component shown enlarged in Fig. 48.16, *DCB* Issue No. 48. To test the desired range of bolt sizes, bolt layouts, beam flange and flange plate thicknesses involves generating loads in the test specimen of over 1000 kN. The dynamic loading actuator has a maximum capability, under seismic-dynamic rates of loading, of 280 kN, hence part of the pre-test setup has involved the design and construction of a multi-functional test rig. This has to be capable of suitably magnifying the actuator force, simulating the appropriate part of the overall connection, being able to develop appropriate lateral movement and the influence of joint rotation in the test specimen and allowing for ease of specimen mounting, testing and removal.

A suitable test rig was developed, built and put into service during the first half of 1999. The test rig set-up is shown in Fig. 51.1. Design of the rig was to NZS 3404 [16]. The test rig was designed with the lowest practicable factor of safety, based on 1.0x the ultimate jack load of 320 kN, with all components designed using their design capacity as derived from NZS 3404. This meant that, when the maximum allowed dynamic jack load in testing was applied, the minimum factor of safety was 1.27 for a steel component and butt weld and 1.43 for a bolt or pin.

There were several reasons for adopting this relatively low factor of safety and these are given in section 5.3.1 of [20]. The reason most relevant to this article was that the HERA Structural Engineer, as Chairman of the Steel Structures Standard Committee, was keen to test the NZS 3404 design provisions on a complex item of mechanical equipment, when these provisions were tested to near their design limit. The reason for this was that the test rig involved the application of a wide number of NZS 3404 clauses, including those relating to pin design. The pin about which the beam unit rotates is shown in Fig. 51.2 and requires the design of this pin to allow for rotation.

Section 9.5 of NZS 3404 [16] contains the requirements for pin design. The requirements come direct from the Australian *Steel Structures Standard*. Application of true pins is rare in

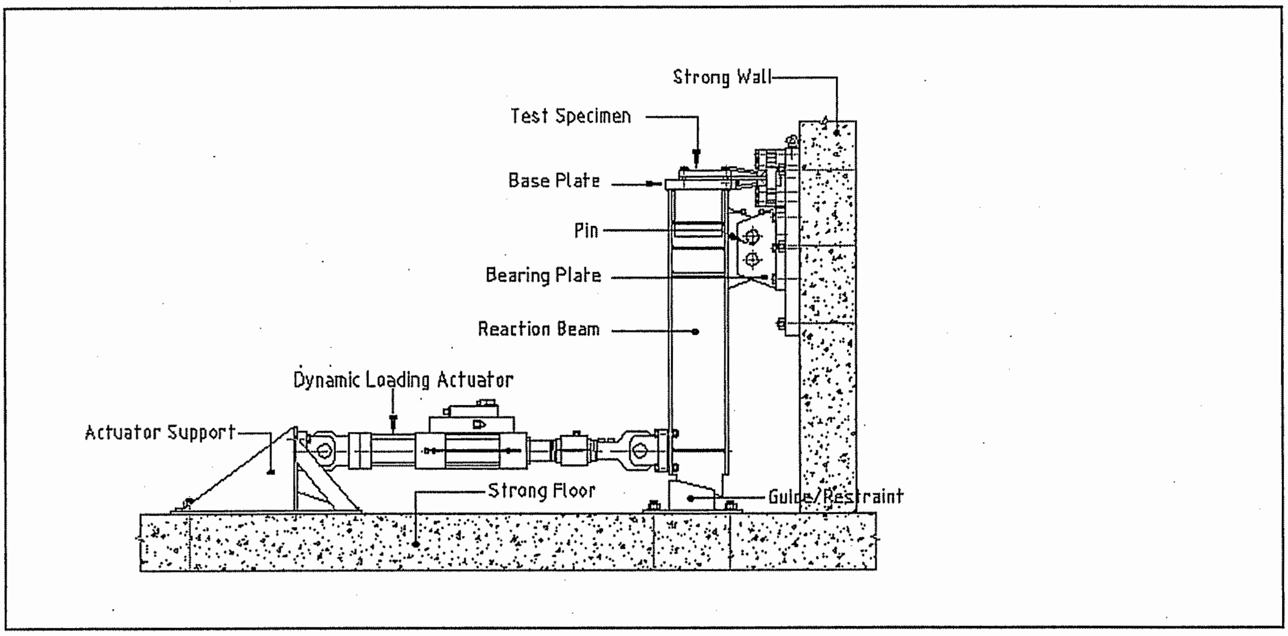

Fig. 51.1
Test Rig Set-up for the Component Tests (from [20])

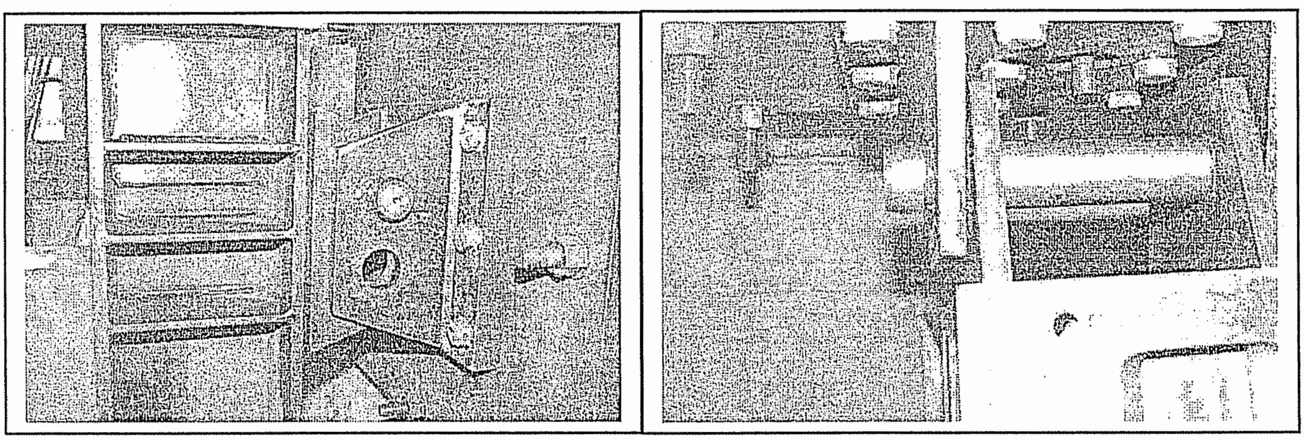

Fig. 51.2
Bearing Unit with Wall Base Plate, Beam and Pin from the Side (Left) and from the Top (Right) (from [20])

structures and the HERA Structural Engineer had never used these provisions in practice, nor is he aware of any applications where the performance of the pin and its supports would be subject to rigorous testing, as in this case.

During the design of the rig, one clause in particular caused some misgivings. This was Clause 9.5.4 relating to the ply in bearing. This refers designers back to Clause 9.3.4.2.1, relating to a ply supporting bolts in bearing. For a bolt in bearing, the material in the ply at the bearing surface is highly constrained, by the surrounding material, the bolt head and nut and, for property class 8.8 bolts when fully tensioned, by the high clamping force of the bolt. In contrast, the bearing surfaces of the bearing plates supporting the pin are constrained only in one plane by the surrounding material, as shown in Fig. 51.2. On the HERA Structural Engineer's advice, the rig designer [20] based the design of the ply in bearing on NZS 3404 Equation 9.3.2.4(1), but modified by incorporating the k_p = 0.5 factor from Clause 9.5.2 for a pin with rotation.

Because of concern over potential bearing plate yielding under near-maximum actuator loads, the bearing plates were painted with a brittle white paint prior to testing, so that any yielding would be observed.

This turned out to be wise, as the Grade 250 plates showed minor visible signs of yielding on the first application of actuator load above 210 kN.

After several tests up to a maximum actuator force of 240 kN, the pin was knocked out and all components measured. The pin had not suffered any distortion, however the holes housing the pin had elongated by 0.5mm each side, on an initial diameter of 68mm. The elongation was unacceptable for dynamic loading and the bearing plates required replacement.

The replacement bearing plates were designed using the same criteria as for the pin, namely that of Clause 9.5.2, but with $f_{y,ply}$ used instead of $f_{y,pin}$. The new bearing plates have been tested under subsequent experimental tests to the full actuator test load of 280 kN (corresponding to 1280 kN force on the pin) with no plate yielding/hole elongation observed.

The replacement design required an increase in bearing plate yield strength (from Grade 250 to Grade 690, Bisalloy 80 steel) and a slight increase in plate thickness.

This provides a clear illustration that the current provisions of Clause 9.5.4 are not sufficient to prevent bearing plate yielding around a pin, prior to development of the design load, and need amendment. The design modification used in the rig repair has worked very satisfactorily and that amendment now follows:

Proposed amendment to NZS 3404 Clause 9.5.4: Ply in Bearing

A ply subject to a design bearing force $\left(V_b^*\right)$ due to a pin in shear shall satisfy:

$$V_b^* \leq \phi V_{b,ply}$$

where

ϕ = strength reduction factor (see table 3.3)
$V_{b,ply}$ = nominal bearing capacity of the ply.

The nominal bearing capacity of the ply ($V_{b,ply}$) shall be calculated as the minimum of Equation 9.3.2.4(2) and the following:

$$V_b = 1.4 f_{y,ply}\, d_f t_p k_p$$

where

$f_{y,ply}$ = yield stress of the ply
d_f = pin diameter
t_p = thickness of the ply
k_p = 1.0 for pins without rotation, or
= 0.5 for pins with rotation

Member Restraint Requirements for Restraint of Category 1, 2 and 3 Members

Background

As stated in Commentary Clause C12.6, the purpose of the member restraint provisions is to ensure comparable behaviour of a member of a given seismic category with that designed to the member restraint requirements presented in the 1989 edition of the Standard. The reason for this is that the adequacy of 1989 provisions have been confirmed by experimental testing ([21]; ref. (12.8) from NZS 3404).

While the aim in going from the 1989 edition restraint requirements to the 1992 edition requirements was not to change the philosophy or the basic design provisions, the 1992 provisions were quite badly worded and awkward to apply. They were substantially revised in the 1997 edition to make them more easily understood and implemented.

However, the 1989, 1992 and 1997 provisions all contain a potential problem that may make their application unconservative, at least in part. This is because the provisions are written on the basis that the design moment at any yielding region will be equal to the design section moment capacity, ϕM_s. All design examples written by the HERA Structural Engineer illustrating the use of these provisions have involved applications where this is the case.

There will inevitably be applications where the member is oversized, such that $M^*_{max} < \phi M_s$. In this instance, when the category 1, 2 or 3 seismic-resisting system containing the member is subjected to the design level severe earthquake (ie. that specified in NZS 4203 [22]), as the system is overloaded, the design moment developed in the yielding region will at first equal and then exceed ϕM_s. This is likely in category 3 systems and is certain in category 2 or 1 systems.

The design restraint provisions of NZS 3404 for segments containing yielding regions (Clause 12.6.2.2) and for adjacent segments in the same member, which don't contain yielding regions (Clause 12.6.2.5) take account of the fact that the moment magnitude along the member under severe seismic action will increase from that associated with development of $M^*_{max} = \phi M_s$ at the yielding regions. The ability of the NZS 3404 design restraint provisions to adequately restrain the member under the actual moments developed has been established experimentally [21]. However, these provisions may not be adequate for an initially underdesigned member, as the actual moment along that member increases from M^*_{design}, to ϕM_s, then to M_{actual} as the yielding region(s) undergoes (undergo) inelastic rotation.

Potential sources of unconservatism may arise in two areas, namely:

(i) Equation 12.6.2 may underpredict the length of yielding region, when the moment gradient is low.

(ii) Clause 12.6.2.5(a) may give an unconservative answer, because of the low value of M^*_x.

This second source is only a problem with restraint of beam members, because in beam-column members (ie. subject to bending and significant axial actions) all segments along the member must have full lateral restraint, from Clause 12.6.2.5(b) of [16].

The solution to this potential unconservatism is very straightforward, namely to increase the design moment magnitude and distribution along the member, for determination of restraint, to that corresponding to development of ϕM_s in all yielding regions. This is to be introduced through a proposed new clause, as detailed below.

This solution will also be applicable for category 2 or 3 beams containing yielding regions due to moment redistribution.

Proposed new clause for NZS 3404 Clause 12.6.1.3 Design bending moment for application for restraint to category 1, 2 and 3 members

When calculating the design moment magnitude and distribution along the member for application of Clause 12.6.2;

(i) At the point of maximum moment in each yielding region, set the design bending moment for calculation of restraint equal to the design section moment capacity, ie.

$$M^*_{r,max} = \phi M_s$$

(ii) At each point along the member,

$$M^*_r = M^*\left(\frac{\phi M_s}{M^*_{max}}\right) \qquad \text{Eq. 12.6.1}$$

where:

M^*_r = design bending moment for calculation of restraint to Clause 12.6.2.

M^* = design bending moment from analysis at the point under consideration

M^*_{max} = design bending moment from analysis at the adjacent yielding region(s)

ϕM_s = design section moment capacity

(iii) Use the value of M^*_r from Equation 12.6.1 in the application of Clauses 12.6.2.1 to 12.6.2.5.

Other proposed changes to Clause 12.6

(1) Change the value of $(\alpha_m \alpha_s) \geq \phi_{oms}$ for category 1 members in Table 12.6.2 of NZS 3404 to $(\alpha_m \alpha_s) \geq 1.0$.

Reason for change: the more stringent restraint requirements for category 1 yielding regions are now given in Table 12.6.3, so this requirement is not necessary - ie. Table 12.6.2 now has equal requirements for all three categories. The table could be deleted, however editorially the above is the easier change to make, given that this is only an amendment to NZS 3404.

(2) Change the variable M^*_x in Clause 12.6.2.5(a) to M^*_r, for application with the new Clause 12.6.1.3.

Acknowledgement

The HERA Structural Engineer would like to thank Ted Blaikie, Consulting Engineer with Opus International Consultants, for bringing this matter to his attention.

Assessing the Structural Capacity of Corrosion-Damaged Steel Beams

Introduction and Scope

In previous issues of the *DCB* (eg. Issue Nos. 46, 47, 49) we have provided design guidance on rates of corrosion and on the durability of structural components and systems.

One important piece of advice that has not yet been published is how to assess the structural capacity available from steel beams that are partially corroded. The reason for this lies in the previous lack of a published design model, consistent with the assessment of structural capacity in accordance with a limit state design standard.

Such a design model has been published [23] by Sarveswaran and Smith, in *The Structural Engineer*, July 1999. Entitled *Structural Assessment of Corrosion-Damaged Steel Beams Using Minimum Capacity Curves*, the paper provides a design model and set of design tables that can be used for this purpose. The design model is developed in accordance with UK limit state design practice, which is sufficiently similar to New Zealand practice to make the results applicable here.

This paper [23] first presents the corrosion decay model used. That model involves a variable loss of thickness across the steel beam, based on the pattern of material loss observed from field studies. Fig. 51.3 shows the pattern of corrosion used in the corrosion decay model. The field studies used to derive this pattern have been conducted in the UK and USA (and are referenced in [23]) so should be sufficiently representative, for design purposes, of New Zealand conditions.

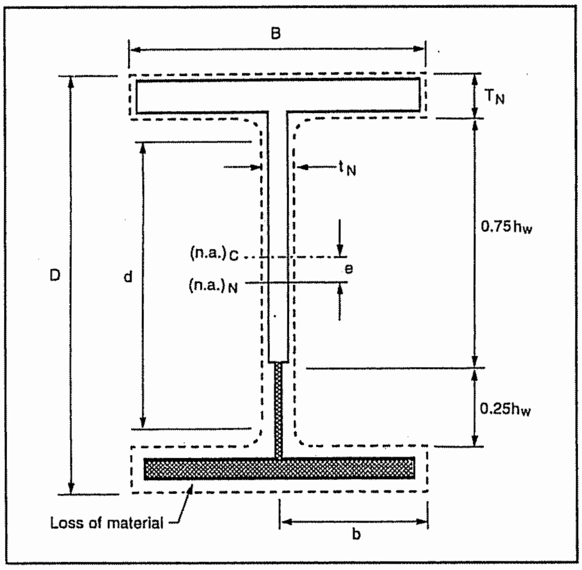

Fig. 51.3
Corrosion Decay Model used in [23]

The authors have then used this decay model to formulate a proposed pattern of cross section material loss as a function of the percentage loss of bottom flange thickness.

They have then analysed corrosion-damaged steel beams for the following modes of failure:

- yielding or buckling of flanges under normal bending stresses
- shear failure of web
- lateral (lateral-torsional) buckling
- bearing or buckling failure of the web near the support or under concentrated loads.

Full details of this analysis procedure are given in [23]. It takes account of change of section status - eg. from compact to non-compact - in the calculation of remaining design capacity.

The outcome, for design application, is a set of minimum capacity curves relating the % loss of bottom flange thickness to the % remaining capacity.

Examples of application of these curves are given.

Use of the UK Paper in Conjunction with Previous DCB Guidance

This first involves assessing the loss of flange thickness. For unprotected steel surfaces exposed to the external atmosphere, the guidance given on pages 5-7 of *DCB* Issue No. 46 will be useful if the length of time for which the unprotected steel surface has been exposed is known.

For steel surfaces which have undergone corrosion following breakdown of a surface protection system, an indication of material loss can usually be gained from determining the thickness of the rust layer. Steel corrosion losses are represented by some 10% to 20% of the rust thickness. However, rust layers over 10mm thick will tend to spall off from curved surfaces and corners, so make the thickness estimate from a surface where this is unlikely to have occurred, eg. the inside top surface of the bottom flange.

Then go into the appropriate curve from [23] for the design capacity being sought. In some instances, only one capacity (eg. section moment capacity) will need to be checked; in other instances, such as continuous beams over supports, several capacities (eg. section moment capacity, shear capacity, web buckling capacity) may need to be checked.

Use the curves to determine the percentage of as-new design capacity remaining.

Compare with the design action required for residual capacity determination, which will often be less than that required for new construction (eg. for road bridges see the Transit Bridge Manual [15]). Hence determine if the structural capacity of the corrosion-damaged member is still adequate.

The minimum capacity curves can be used for non-composite and for composite beams; in the latter case the curve used for moment capacity is that for a compact beam with full lateral restraint and the original capacity is that from section 13.4 of NZS 3404 [16].

High Strength Structural Bolts, Nuts and Washers in Sizes Above M36

The information presented in the following article was provided by Chris James of EDL Fasteners Ltd. Charles Clifton acknowledges Chris's input, which has been much appreciated.

Introduction

The high strength structural (HSFG) bolt, for which design and installation provisions are given in NZS 3404 [16], is manufactured and supplied in accordance with AS/NZS 1252 [24]. That standard specifies material and dimensional requirements for the bolt, nut and hardened washer; all three components are integral to the expected performance of the HSFG bolt. It specifies testing requirements in terms of dependable strength to be achieved and required modes of failure of the bolt, nut and washer.

Because of the importance of each component to the successful performance of the overall fastener, they are usually supplied as a bolt set, with this set containing the bolt, hardened washer and nut.

AS/NZS 1252 specifies HSFG bolt set sizes up to M36 (36mm diameter with an ISO metric coarse thread). These bolts are typically available in lengths of up to 200mm (bolt length is measured from the underside of the bolt head to the end of the shank). The maximum nominal bolt length specified by AS/NZS 1252 [24] is 240mm.

For applications requiring longer bolt lengths, the "bolt" can be formed from AISI 4140 threaded rod. This material has f_{uf} = 900MPa and elongation properties very similar to that for HSFG bolt steel from [24]. A typical application for this rod is for hold-down bolts at the base of seismic-resisting system columns, as detailed in section 5.5(2), pages 25-26 of *DCB* Issue No. 50. For bolt diameters of up to M36, the nut and hardened washer will be supplied to AS/NZS 1252.

However, what about HSFG bolt sets above M36? Given that these are outside the scope of AS/NZS 1252, is such a bolt set dependably available?

The answer is yes - in two sizes they are available as a bolt set, while the industry in Australia has developed criteria for sizing the nuts that will give them equivalent performance to a HSFG nut to [24]. Each of these cases is now detailed.

Standard HSFG Bolt Set Sizes Above M36

There are two sizes of bolt set available - M42 or M48. These correspond to a bolt shank thickness of 42 and 48mm respectively.

The bolt sets can be ordered from a supplier of structural steel fasteners in the normal way, however they will incur a significantly higher unit cost because they are a non-standard item.

HSFG Nuts and Washers for Bolt Sizes Above M36

The nuts are specified as for property class 8 nuts to AS/NZS 1112 [25] *ISO Metric Hexagon Nuts...*, **however with the nut height** (the terminology for nut thickness used in [24, 25]) **made equal to the shank diameter.**

Nuts made to these dimensions have been found to meet the performance requirement from [24] of being able to fracture the bolt through the tensile stress area, when subject to tensile testing to failure, rather than to fail by nut fracture or thread stripping.

This allows HSFG nuts to be formed for threaded rod of up to M64, using the above modification for nut height in conjunction with the other nut dimensions from Table 2 of [25].

For applications such as hold-down bolts, where the bolt or threaded rod will be snug tight or only lightly tensioned, the washer which is placed under the component being turned should comply with the material properties and dimensions given in AS 1237 [26] for the appropriate bolt or rod shank diameter.

For applications requiring the bolt or threaded rod to be fully tensioned (suitably powerful tensioning equipment will be needed for this!) the washer material must, in addition, be hardened and tempered to meet the hardness requirements of Clause 4.3 of AS/NZS 1252 [24].

Moment End Plate Connections: Extension of Application and Design Example

Introduction and Scope of Article

HERA Report R4-100, *Structural Steelwork Connections Guide* [8], contains pre-engineered, load-rated connections covering a wide range of structural steelwork applications. *DCB* Issue No. 50 presents a companion article covering supplementary connection design issues, such as stiffeners and doubler plate reinforcement in moment-resisting beam to column connections.

One of the connection types covered is the moment end plate connection, designated MEP in [8] and covered on pages 123-144 therein. This connection offers a more cost-effective solution for beam to column connections of moment-resisting steel framed seismic-resisting systems than the solution conventionally used in New Zealand, which involves a shop welded WM connection, Christmas-tree construction and either one or two bolted welded beam splices (BWBS) per beam span. (The number of BWBS required depends on the column spacing).

The MEP connection has not been commonly used in New Zealand to date, principally because it has previously involved considerable designer effort and the need for iteration in design if the hierarchy of connection component and connector strengths required don't work first time around. The advent of the MEP design tables in [8] and the simplified design procedures for stiffeners and column flange tension strength assessment published in *DCB* Issue No. 50 have greatly sped up and simplified MEP connection design for steel moment-resisting frames, at least for the category 3 and 4 connections covered in R4-100.

Having reviewed the literature on MEP connection performance under inelastic seismic (cyclic) loading, the HERA Structural Engineer can advise that it is very straight-forward to extend the published design guidance given in [8] and *DCB* No. 50 for category 3 MEP connections to category 2 and 1 connections. The requirements for this extension of scope are presented in the next part of this article.

The last part presents two design examples, illustrating use of the R4-100 design tables and drawings, the *DCB* Issue No. 50 requirements and the additional guidance given above.

Extending the MEP Provisions of R4-100 to Connections to Category 2 or 1 Members

Changes to design procedure

The only changes required are to increase the overstrength factor in the two instances where this is needed. These are:

(i) Moment capacity through the bolt group to be greater than the category 2 or 1 overstrength beam section moment capacity. In terms of the design formulae, given in sections E1, E2 and E4 on pages 126 and 127 of [8], $N^*_{fmt} \le \phi N_{tb}$ is required, where:

$$N^*_{fmt} = \frac{\phi_{oms} Z_{ex} f_{yf}}{d - t_f}$$

$$\phi N_{tb} = n_{tb}\phi_b N_{tf} - N^*_p$$

ϕ_{oms} = overstrength factor from Table 12.2.8(1) of NZS 3404 [16] for the category of beam

(ii) The beam flange to endplate weld, if it is a double-sided fillet weld, shall develop the overstrength tension capacity of the flange. In terms of the design formulae, given in sections E1, E2 and E4 on pages 126 and 127 of [8], $N^*_{ft} \le \phi N_{wf}$ is required, where:

$N^*_{ft} = \phi_{oms} b_f t_f f_{yf}$ (from section E2 of [8])

$\phi N_{wf} = 2\phi_w 0.6 f_{uw} \frac{t_{wb}}{\sqrt{2}} b_f$ (from section E6 of [8])*

all variables are as defined in [8] except for ϕ_{oms}.

* Note: b_f is missing from this equation in [8] as first published, however the tables are correct.

By comparing the connection component and connector capacities of MEP connections designed to the above provisions with those tested by Whittaker & Walpole [27], it is shown that the MEP connections designed to the above provisions will sustain the rotation demands expected of them.

Inelastic action will occur principally in the beam, as intended, with minor inelastic action possible in the endplate as well as in the panel zone. Both these places can accommodate this action; one test from [27] involved a deliberately understrength endplate to determine the connection behaviour, ductility and failure mode when most of the significant inelastic action occurs in the endplate. This connection sustained 7 cycles of inelastic rotation to $\mu = 4$ prior to failure occurring in the weld between the beam flange and endplate. Most of the inelastic demand, as planned, occurred in the endplate. The endplate in this test was from Grade 250 steel, 20 mm thick, onto a Grade 250 310UB46 beam.

A second specimen tested in [30] with a 25 mm thick endplate, Grade 250 steel, sustained 8 cycles to $\mu = 6$ prior to failure, with minimal distortion in the endplate.

Adjusting for the different yield strength of the beams between [27] and [8], for a MEP connection to a Grade 300 310UB46 to deliver comparable performance to the second specimen from [27] described above, an endplate thickness

≥ 30 mm is required. This requirement is achieved from [8], with a 32 mm thick endplate being specified for a Grade 300 310UB46 category 3 MEP connection. Other connection details (eg. bolt numbers, diameters, weld sizes) are comparable between [27] and [8], taking into account the difference in beam grade.

Thus, the performance of the MEP specimens from [27] show that the connection specified in [8], for the tested beam size, is likely to deliver category 1 performance. This outcome will also apply for the many other beam sizes covered in [8], subject to the bolts and welds being checked.

Extending the use of the R4-100 tables for MEP category 3 member connections to category 2 or category 1 member connections

Select the connection components and connectors associated with the given beam member size from the appropriate Cat 3 MEP table from R4-100, then check the flange weld and the bolt group capacity ((n_{tb} + n_{cb}) and diameter) for the increased overstrength actions from (i) and (ii) on page 15 above.

If selecting the lightest member within a designation (eg. a 610UB101, which designation also contains a 610UB113 and 610UB125), then a rapid way to determine the suitability of the bolts and the flange weld is to multiply the design moment capacity given in the table for that member, ie. ϕM_{con}, by the ratio of (ϕ_{oms}/1.1), where ϕ_{oms} = overstrength factor for the category under consideration and 1.1 = overstrength factor for the category 3 member used in R4-100, then to adopt the bolts and flange weld details given for a heavier member within the designation for which $\phi M_{con,heavier} \geq \phi M_{con,lighter}$ (ϕ_{oms}/1.1). For example, take a MEP connection to a category 2 610UB101 Grade 300 member. ϕ_{oms} = 1.15 for this category and grade, hence check the bolts and weld size for a heavier 610UB member with $\phi M_{con} \geq$ 783 x (1.15/1.1) = 819 kNm. A 610UB113 has ϕM_{con} = 829 kNm, so adopt the bolts and flange welds associated with the heavier designation, while all other components and connectors are those specified for the 610UB101. In this particular example, there is no change required to the bolts and flange welds in using the 610UB101 category 3 details from [8] in a category 2 connection. (This example has relevance to section 1 in design example no. 51.2, on page 18 herein).

MEP Connection Design Examples

Scope of design examples

Two MEP design examples are presented. Both are for beam to column connections in primary moment-resisting steel frame (PMRSF) seismic-resisting systems and are selected in order to illustrate use of the MEP design tables from [8], the supplementary design requirements for *DCB* Issue No. 50 and the recommendations given above.

The first example is for a category 3 beam in a category 3 (nominally ductile) system, the second example for a category 2 beam in a category 2 (limited ductile) system. The beam and column sizes chosen correspond to those that would be used in a high-rise PMRSF; this choice has been made to illustrate that MEP connections are a practical option for high-rise PMRSF construction involving large members and connection forces.

Design example no. 51.1

A 610UB101 Grade 300 beam is connected to one side of a 610UB179 Grade S275 column. The column is a British Steel made section. The beam and the PMRSF system are both category 3.

The connection is located at a lower level of the frame, hence is not adjacent to a column free end.

Material properties (f_y, f_u) for the Grade 300 beam and Grade S275 column are given on pages 20, 21 of R4-100 [8]. Flat bars and plate used for connection components are taken as Grade 250.

Design actions from the beam are M^* = 783 kNm and V^* = 350 kN.

The design procedure is set out in a series of numbered sections, for ease of reference and to show where iteration or reference to previously designed components is required.

The design example is intended to be read in conjunction with [8], so details that are given in [8] are only repeated herein where necessary.

1. Choose Endplate, Bolts, Weld Details Between Endplate and Beam

These are obtained from the MEP Cat 3 table, on page 139 of [8]. From that table,
$\phi M_{con} > M^*$ and $\phi V_{con} > V^*$ OK

Dimensions for bolt layout etc. are given on page 299 of [8].

Bolt size: M36
Endplate thickness, t_i = 40 mm

2. *Check Column Flange Requirements*

2.1 *Column flange width required*

$b_{fc} \geq S_g + 3.0\ d_f$ (equation 50.6.1, *DCB* Issue No. 50)

$b_{fc,supplied}$ = 307 mm (see page 2 of [28])

$S_g + 3.0d_f$ = 140 + 3.0 x 36 = 248 mm

Condition is met.

2.2 *Preliminary design check on column flange transverse tension capacity when stiffened with a tension stiffener*

This check is using equation 50.7, *DCB* Issue No. 50.

$$t_{fc} \approx 0.5 C_3 C_4 (0.9\, t_i + t_{fb}) \left(\frac{f_{yi}}{f_{y,cf}} \right) = 22.7 \text{ mm}$$

C_3 = 0.95 (column not at free end)
C_4 = 1.0
t_{fb} = 14.8 mm (from either [28] or [29]
f_{yi} = 250 MPa (see page 21 of [8])
$f_{y,cf}$ = 265 MPa (see page 21 of [8] for column flange thickness from [28])

t_{fc} = 23.6 mm, from [28], for a 610UB179

Hence the stiffened column flange tension capacity, when stiffened with a tension stiffener, is likely to be adequate (ie. additional stiffening, such as that given by section 4.2(6) on page 16 of *DCB* Issue No. 50, is not likely to be needed). This is checked in section 3.2 below.

3. *Design of Tension / Compression Stiffeners*

3.1 *Sizing of tension/compression stiffeners*

This follows section 3.2 of *DCB* Issue No. 50, using the same step numbers as given therein. Only those steps requiring calculation input are included herein.

(1) Calculate minimum width of each stiffener.

$b_{s,min} \geq (0.9 b_{fb} - t_{wc})/2$ = 96 mm

b_{fb} = 228 mm
t_{wc} = 14.1 mm

(2) Calculate area of each pair of stiffeners required using equation 50.2, DCB Issue No. 50.

$$A_{s,pair} \geq (b_{fb} t_{fb} - t_{wc} t_{fb}) \left(\frac{f_{yb}}{f_{ys}} \right) = 3799 \text{ mm}^2$$

f_{yb} = 300 MPa
f_{ys} = 250 MPa

(3) Calculate minimum stiffener thickness necessary to prevent local buckling in compression using equation 50.3, DCB Issue No. 50.

$$t_{s,min} \geq \left(\frac{b_s}{C_1} \right) \left(\sqrt{\frac{f_{ys}}{250}} \right) = 6.4 \text{ mm}$$

C_1 = 15 as the incoming beam is a category 3 member.

(4) Select appropriate stiffener thickness, width.

If use b_s = 120 mm and t_s = 16 mm

$A_{s,pair}$ = 120x16x2 = 3840 mm² OK

(5) Calculate fillet weld size required for the double-sided fillet weld between stiffener and column flange at the end adjacent to the incoming beam, from equation 50.4, DCB Issue No. 50.

$$v^*_{w,s,cf} = \frac{0.9 b_s t_s f_{ys}}{2 b_s} = 1.80 \text{ kN/mm}$$

f_{ys} = 0.250 kN/mm²

(6) Select suitable fillet weld leg length

ϕv_w = 1.96 kN/mm for t_w = 12 mm, E48XX category SP.

Adopt $t_{w,s,cf}$ = 12 mm fillet weld size

(8) Calculate fillet weld size for double-sided fillet weld between stiffener and column web, from equation 50.5, DCB Issue No. 50.

$$v^*_{w,s,(cw\ or\ dp)} = \frac{0.9 b_s t_s f_{ys}}{C_2 d_{1c}} = 0.40 \text{ kN/mm}$$

C_2 = 2.0 (1 beam framing into column)
d_{1c} = 540 mm (from [28])

(9) Select suitable fillet weld leg length.

ϕv_w = 0.82 kN/mm for t_w = 5 mm, E48XX Category SP
Adopt $t_{w,s,(cw\ or\ dp)}$ = 5 mm fillet weld size

3.2 *Check column flange transverse tension capacity*

This check is to NZS 3404 [15] Clause M2.2.1. The relevant equation is Equation M2.4, which is not written out in full herein.

Calculation of the dimensional variables used in that equation is fully covered herein.

$d'_{fb} = d_f + 3 = 36 + 3 = 39$ mm

$n = (b_{fc} - S_g)/2 = 84$ mm

$m = 0.5b_{fc} - n - 0.5t_{wc} - r_c = 46$mm
$r_c = 16.5$ mm (from [28])

$v = (S_p - t_s - 2t_{w,s,cf})/2 = 55$ mm
S_p = bolt pitch = 150 mm (from [8])

$$w = \left[m\left(m + n - 0.5d'_{fb}\right)\right]^{0.5} = 71 \text{ mm}$$

$\phi N_{ms} = 0.9 \times 265 \times 23.6^2 \times 11.76 \times 10^{-3}$
$= 1562$ kN

$$N_t^* = \phi_{oms} \frac{M_{sx}}{(d - t_f)_b} = 1624 \text{ kN}$$

$\phi_{oms} = 1.1$ for category 3
$M_{sx} = 782/0.9 = 868$ kNm
$d_b = 0.603$ m

$$N_t^* / \phi N_{ms} = 1624 / 1562 = 1.04$$

Column flange tension action is 4% over tension capacity - accept.

3.3 *Check column panel zone*

This check is to NZS 3404 Clauses 12.9.5.2(b) and 12.9.5.3.2.

(1) Calculation of panel zone design shear force.

$$V_p^* = \frac{M_L}{(d_b - t_{fb})_L} + \frac{M_R}{(d_b - t_{fb})_R} - V_{COL}$$

$= 1308$ kN
M_L = 0 (no left hand beam; see Fig. 51.4)
$M_R = C_2 M_{sx} = 1.0 \times M_{sx} = 868$ kNm
V_{COL} = 168 kN (assumed for this design example and realistic for this size and configuration of joint)

(2) Calculation of panel zone design shear capacity, without doubler plate.

This check is to NZS 3404 Equation 12.9.5.3(5).

$\phi V_c = 0.9 \times 0.6 \times 275 \times 620 \times 14.1 \times 1.0 \times 1.03 \times 10^{-3} = 1340$ kN
$f_{yp}^* = f_{ycw} = 275$ MPa
t_p = 0 for this check
d_c = 620 mm
η = 1.0 for this example and the typical value for a PMRSF

(3) Check panel zone adequacy without doubler plate.

$\phi V_c > V_p^*$ OK

Design example no. 51.1 is complete; details of the connection are shown in Fig. 51.4.

Design example no. 51.2

Two 610UB101 Grade 300 beams are connected to a 914UB201 Grade S275 column, one each side as shown in Fig. 51.5. The column is a British Steel made section. The beams and the PMRSF system are category 2.

The connection is located at a lower level of the frame, hence is not adjacent to a column free end.

Sources of material properties are referenced from the preamble to design example no. 51.1.

In this design example, the beam framing into the column from the left hand side (see Fig. 51.5) is assumed to have its negative moment end (as defined in Clause 1.3 of NZS 3404) at the column and the beam framing into the column from the right hand side has its positive moment end at the column.

Design actions (ignoring sign of moment) are:

$$M_L^* = 783kN, V_L^* = 350kN \downarrow$$
$$M_R^* = 730kN, V_R^* = 105kN \uparrow$$

The format and layout of the design example are similar to that for design example no. 51.1.

1. *Choose Endplate, Bolts, Weld Details Between Endplate and Beam*

1.1 *Endplate and weld between beam web and endplate*

These are obtained directly from the MEP Category 3 table, page 139 of [8].

1.2 *Bolts and weld between beam flange and endplate*

Follow the procedure given on page 16 above; the example therein is for a category 2 610UB101.

1.3 *Conclusion*

The cleat, bolt, weld details given on page 139 of [8] are suitable for this category 2 connection.

2. Check Column Flange Requirements

2.1 Column flange width required

$b_{fc} \geq S_g + 3.0d_f = 248$ mm
$b_{fc,supplied} = 303$ mm (see page 2 of [28])

Condition is met.

2.2 Preliminary design check on column flange transverse tension capacity when stiffened with a tension stiffener

This check uses equation 50.7, *DCB* Issue No. 50, multiplied by
$(\phi_{oms,cat2}/\phi_{oms,cat3}) = 1.15/1.10 = 1.05$

$$t_{fc,reqd} \approx 1.05 \times 0.5C_3C_4\,(0.9t_i + t_{fb})\left(\frac{f_{yi}}{f_{y,cf}}\right)$$

$$= 23.7 \text{ mm}$$

all variables above are as for section 2.2 of design example 51.1 on page 17.

$t_{fc,supplied}$ = 20.2 mm, from [28], for a 914UB201.

Hence the stiffened column flange tension capacity, when stiffened with a tension stiffener, is not likely to be adequate and the additional split backing plate stiffening given in section 4.2(6) on page 16 of *DCB* Issue No. 50 is likely to be needed. This is checked in section 3.2 below.

3. Design of Tension/Compression Stiffeners

3.1 Sizing of tension/compression stiffeners

This follows section 3.2 of *DCB* Issue No. 50 and many details are the same as for design example 51.1 or very similar.

(1) Calculate minimum width of each stiffener.

$b_{s,min} \geq (0.9b_{fb} - t_{wb})/2 = 95$ mm

(2) Calculate area of each pair of stiffeners required.

$A_{s,pair} \geq 3780$ mm^2

(3) Calculate minimum stiffener thickness necessary to prevent local buckling in compression.

$$t_{s,min} \geq \left(\frac{b_s}{C_1}\right)\left(\sqrt{\frac{f_{ys}}{250}}\right) = 11.9 \text{ mm}$$

C_1 = 8 as the incoming beam is a category 2 member

(4) Select appropriate stiffener thickness, width.

Use b_s = 120 mm and t_s = 16 mm

(5) Calculate fillet weld size required for the double-sided fillet weld between stiffener and column flange at the ends adjacent to the incoming beams.

As for section 3.1(5), design example 51.1, except that this now applies to both ends of the stiffener (see Fig. 51.5).

(6) Select suitable fillet weld leg length.

Adopt $t_{w,s,cf}$ = 12 mm, as for section 3.1(6), design example 51.1.

(8) Calculate fillet weld size for double-sided fillet weld between stiffener and column web from equation 50.5, DCB Issue No. 50.

$$v^*_{w,s,(\text{cw or dp})} = \frac{0.9b_s t_s f_{ys}}{C_2 d_{1c}} = 0.52 \text{ kN/mm}$$

C_2 = 1.0 (2 beams framing into column)
d_{1c} = 824 mm (from [28])

(9) Select suitable fillet weld leg length.

ϕv_w = 0.82 kN/mm for t_w = 5mm, E48XX Category SP
Adopt $t_{w,s,(\text{cw or dp})}$ = 5mm fillet weld size.

3.2 Check column flange transverse tension capacity incorporating the tension stiffener sized in section 3.1

d'_{fb} = $d_f + 3 = 39$ mm

n = $(b_{fc} - S_g)/2 = 82$mm

m = $0.5b_{fc} - n - 0.5t_{wc} - r_c = 43$ mm
r_c = 19.1 mm

v = $(S_p - t_s - 2t_{w,s,cf})/2 = 55$ mm
S_p = bolt pitch = 150 mm

w = 67 mm

ϕN_{ms} = $0.9 \times 265 \times 20.2^2 \times 11.75 \times 10^{-3}$
= 1144 kN

$$N_t^* = \frac{\phi_{oms} M_{sx}}{(d - t_f)_b} = 1698 \text{ kN}$$

ϕ_{oms} = 1.15 for category 2
M_{sx} = 782/.0.9 = 868 kNm (either beam)

$N_t^* / \phi N_{ms}$ = 1698/1144 = 1.48

Column flange tension action is 48% over tension capacity; need to increase the stiffened column flange tension capacity.

3.3 *Provide additional column flange tension capacity with split backing plates*

These are sized in accordance with section 4.2(6) *DCB* Issue No. 50.

Try t_{bp} = 20 mm

From equation 50.8 of *DCB* Issue No. 50.

$$\phi N_{ms,additional} = 0.9 f_{ybp} t_{bp}^2 0.5 \times [\ ] x10^{-3}$$

$$= 529 \text{ kN}$$

f_{ybp} = 250 MPa
[ ] = 11.75

Check $\phi N_{ms,ts} + \phi N_{ms,additional}$ = 1673 kN
$\phi N_{ms,ts}$ is the value from section 3.2 above.

$N_t^* / \phi N_{ms,total}$ = 1698/1673 = 1.02

Column flange with tension stiffeners and split backing plates is 2% over capacity - accept.

Dimensions of split backing plates - these must cover the yield line region shown in NZS 3404 Fig. M2 and should also comply with Clause M2.2.2.2. (a), (c) and with 0.5x L_{bp} from (b).

(a) t_{bp} = 20 mm as determined above

(b) $L_{bp} \geq$ larger of ((S_p + 4d_{fb})/2 = 147 mm; $w + v$ = 122 mm) = 147 mm
L_{bp} = 150 mm adopted

(c) $b_{bp} \geq$ (b_{fc} - t_{wc} - 2r_c)/2 = 125 mm
b_{bp} = 130 mm adopted

Each backing plate is a 130 x 20 flat 150 mm long. Position of these is as shown in Fig. 51.5; there are 16 in that connection.

3.4 *Column panel zone*

(1) *Preliminary design estimate of doubler plate requirements.*

From section 3.3.1.1, page 14, *DCB* Issue No. 50, as a preliminary estimate, doubler plate thickness = t_{wc} is required; range of individual plate thickness to be between 5 mm and 10 mm.

t_{wc} = 15.1 mm; Consider 2 x 8 mm thick plates; Grade 250.

(2) *Calculation of panel zone design shear force.*

$$V_p^* = \frac{M_L}{(d_b - t_{fb})_L} + \frac{M_R}{(d_b - t_{fb})_R} - V_{COL}$$

= 2810 KN

M_L = M_R = $C_2 M_{SX}$ = 1.1M_{SX} = 956 kNm
C_2 = 1.1 (see Clause 12.9.5.2(b)(ii) of [16])
V_{COL} = 440 kN (assumed for this design example and realistic for this size and configuration of joint)

(3) *Calculation of panel zone design shear capacity, with the 2x8 mm doubler plates.*

$$f_{yp}^* = \frac{t_{wc} f_{yc} + 2 t_p f_{yp}}{t_{wc} + 2 t_p} = 268 \text{ MPa}$$

$f_{yc,w}$ = 275 MPa
f_{yp} = 260 MPa (from page 21 of [8])

ϕV_c = 0.9x0.6x268x903x31.1x1.0x 1.01x10^{-3} = 4105 kN
d_c = 903 mm (from [28])
η = 1.0

$\phi V_c >> V_p^*$; look at reducing to 1 doubler plate

(4) *Calculation of panel zone design shear capacity with 1x6 mm thick doubler plate.*

f_{yp}^* = 276 MPa
f_{yp} = 280 MPa (from Table C2.2.1 of [16])

ϕV_c = 0.9x0.6x276x903x21.1x1.0x 1.02x10^{-3} = 2896 kN

$\phi V_c > V_p^*$ - Accept 1x6 mm thick doubler plate

(5) Check doubler plate slenderness to Clause 12.9.5.3.3 of [16].

$$\left(\frac{d_c - 2t_{fc}}{t_{wc} + k_1 t_p}\right)\left(\sqrt{\frac{f^*_{yp}}{250}}\right) = 55 \leq C_3 = 125 \qquad \text{OK}$$

k_1 = 0.25 (doubler plate not plug welded to web)

(6) Check doubler plate thickness in relation to welds along the tension/compression stiffeners. (This is recommendation no. 3, Commentary Clause C12.9.5.3.2 of NZS 3404 Part 2).

$t_{w,s,(cw\ or\ dp)}$ = 5 mm (see section 3.1(a) above).
t_p=6 mm OK

(7) Depth and width of doubler plate.

$d_p = d_b + 100 \approx 602 + 100 = 705$ mm
(d_p is the vertical dimension)
$b_p = d_{1c} = 825$ mm
(b_p is the horizontal dimension)

(8) Welds down sides of doubler plate (into the column root radius). These are sized in accordance with section 2.4, page 12, *DCB* Issue No. 47.

$V^*_w = 0.9 x 0.6 f_{yp} A_{wp} = 640$ kN
$A_{wp} = t_p d_p = 4230\ \text{mm}^2$

ϕV_w for t_w = 6 mm (largest possible)
= 689 kN OK
E48XX, Category SP weld

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

(9) Welds across top and bottom of doubler plate (into the column web). These are 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 (largest possible)
= 807 kN OK

E48XX, Category SP weld

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

Design example no. 51.2 is complete; details of the connection are shown in Fig. 51.5.

Suitable Equipment for Fully Tensioning the HSFG Bolts

These two MEP connections require the full tensioning of the M36 high strength structural bolts.

A suitable torque wrench for this is the *Alkitronic - E/A PG 480/A* analogue torque wrench. It weighs 11 kg and can fully tension these bolts in accordance with the part turn method specified in NZS 3404 Clause 15.2.5.2, with only the as-supplied bolt lubricant required by Clause 3.2.5 of AS/NZS 1252 [24] instead of requiring a lubricant with a lower coeffficient of friction. The machine is easy to use, silent in operation and tightens by direct and continuous torquing rather than by impact action. The machine uses a reaction arm which bears against one of the adjacent bolts in a bolt group, rather than requiring the operator to supply the reaction as is needed with a typical impact wrench.

Further details on this or other appropriate wrenches for fully tensioning the bolts are available from a specialist supplier, such as:
Hydraulic Tool Hire Ltd
Phone: 0-9-274 0121 Fax: 0-9-274 5192
Email: hytc@voyager.co.nz

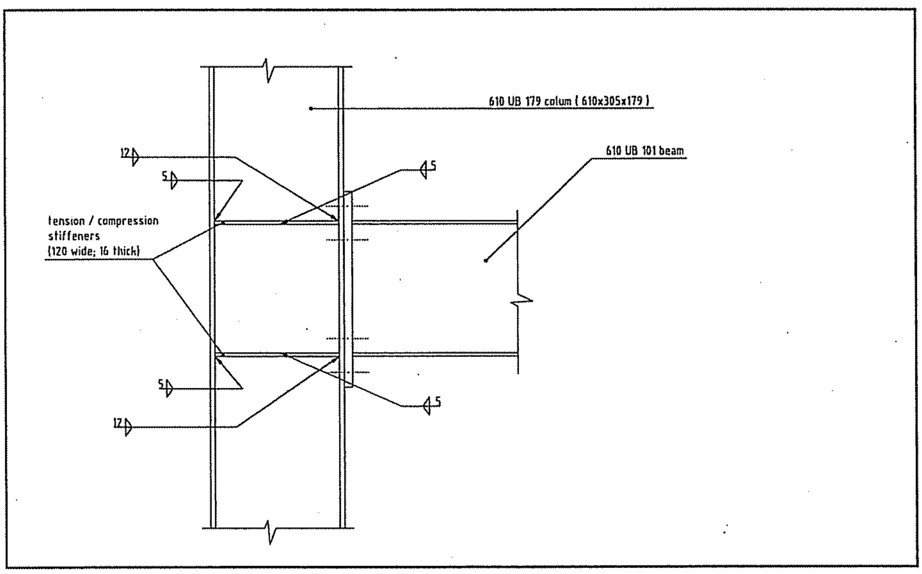

Fig. 51.4 MEP Connection for Design Example 51.1
Note: dimensions, details given in R4-100 [8] for this connection type are not repeated herein.

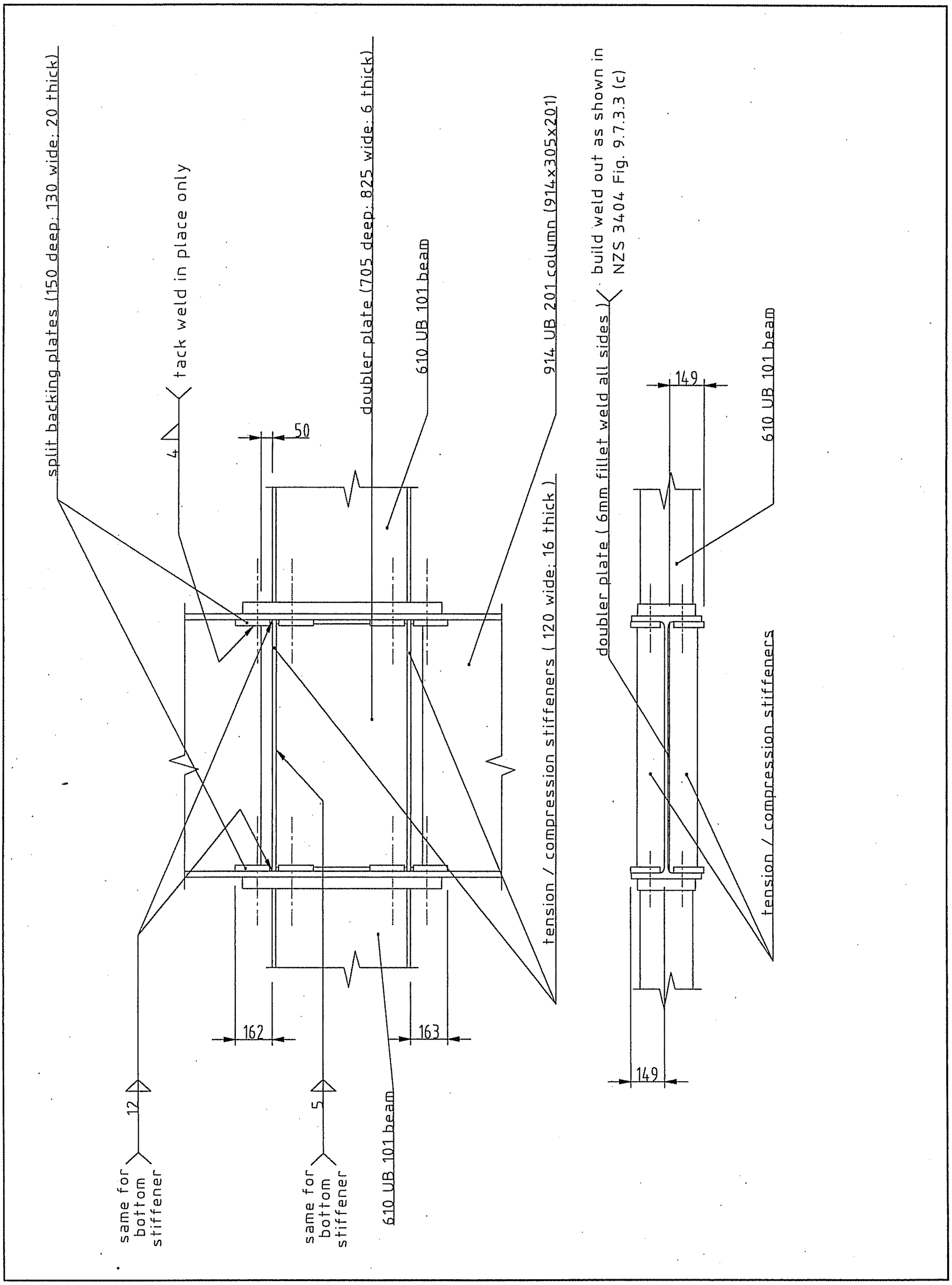

Fig. 51.5
MEP Connection for Design Example 51.2

Note: dimensions, details given in R4-100 [8] for this connection type are not repeated herein.

References

1. Draft C2/AS1 - C4/AS1 plus Annex/AS1, Draft Acceptable Solutions for Fire Safety: 1999; Building Industry Authority, Wellington.

2. C2/AS1 - C4/AS1 plus Annex/AS2, Acceptable Solutions for Fire Safety:1995; Building Industry Authority, Wellington.

3. NZBC:1992, New Zealand Building Code; Building Industry Authority, Wellington.

4. Kirby, B; et.al.; Natural Fires in Large Scale Compartments; British Steel Technical, Swindon, United Kingdom, 1994.

5. Clifton, GC and Forrest, E; Notes Prepared for a Seminar on Design of Steel Buildings for Fire Emergency Conditions; HERA, Manukau City, 1996, HERA Report R4-91.

6. DD ENV 1991-2-2:1996, Eurocode 1: Basis of Design and Actions on Structures, Part 2.2 Actions on Structures Exposed to Fire (together with United Kingdom National Application Document); BSI Standards, London, England.

7. Buchanan, AH, Editor; Fire Engineering Design Guide; Centre for Advanced Engineering, University of Canterbury, 1994.

8. Hyland, C; Structural Steelwork Connections Guide; HERA Manukau City, 1999, HERA Report R4-100.

9. Bennetts, ID et.al.: Economical Carparks - A Guide to Fire Safety; BHP Steel Marketing - Construction and Engineering; Port Kembla, Australia, 1999.

10. Rapattoni, F et.al.; Composite Steel Road Bridges - Concepts and Design Charts; BHP Integrated Steel, Wollongong, Australia, 1998.

11. Szokolik, A and Rapattoni, F; Coatings Guide for New Steel Bridges; BHP Integrated Steel, Wollongong, Australia, 1998.

12. Deacon, DH et.al.; Durability of Steel Bridges: A Survey of the Performance of Protective Coatings; The Steel Construction Institute, Ascot, England, 1998, SCI Publication 241.

13. Hyland, C; Structural Steelwork Estimating Guide; HERA, Manukau City, 1998, HERA Report R4-96.

14. SAA HB77.6, Australian Bridge Design Code, Section 6: Steel and Composite Construction; Standards Australia, Sydney, Australia.

15. Transit New Zealand Bridge Manual: Design and Evaluation; Transit New Zealand, Wellington, 1994.

16. NZS 3404:1997, Steel Structures Standard; Standards New Zealand, Wellington.

17. Clifton, GC; HERA Specification for the Fabrication, Erection and Surface Treatment of Structural Steelwork; HERA, Manukau City, 1998, HERA Report R4-99.

18. Hyland, CWK and Enzensberger, M; Prediction of Site-Specific Steel Corrosion Rates in New Zealand to Assist Coatings Selection; ASEC98, Auckland, 1998, Vol. 2, pp. 835-842; SESOC, Auckland, 1998.

19. AS/NZS 2312:1994, Guide to the Protection of Iron and Steel Against Exterior Atmospheric Corrosion; Standards New Zealand, Wellington.

20. Eisenbeis, M; Dynamic Behaviour of Welded Beam to Column Connections Under Seismic Loading - Development of Small-Scale Test Rig; Diplomarbeit prepared at HERA for the Fachhochschule Konstanz, Manukau City, 1999.

21. Guruparan, NI and Walpole, WR; The Effect of Lateral and Local Buckling in Steel Beams Under Simulated Seismic Loading; Department of Civil Engineering, University of Canterbury, Christchurch, 1984.

22. NZS 4203:1992, General Structural Design and Design Loadings for Buildings; Standards New Zealand, Wellington.

23. Sarveswaran, V and Smith, JW; Structural Assessment of Corrosion-Damaged Steel Beams Using Minimum Capacity Curves; The Structural Engineer, Vol. 77, No. 14, July 1999, pp 17-27.

24. AS/NZS 1252:1996, High-Strength Steel Bolts With Associated Nuts and Washers for Structural Engineering; Standards New Zealand, Wellington.

25. AS/NZS 1112:1996, ISO Metric Hexagon Nuts, Including Thin Nuts, Slotted Nuts and Castle Nuts; Standards New Zealand, Wellington.

26. AS 1237:1973, Flat Metal Washers for General Engineering Purposes (Metric Series); Standards Australia, Sydney, Australia.

27. Whittaker, D and Walpole, WR; Bolted End Plate Connections For Seismically Designed Steel Frames; Department of Civil Engineering, University of Canterbury, Christchurch, 1982, Report 82-11.

28. Structural Sections to BS4:Part 1 and BS 4848: Part 4; British Steel Sections, Plates and Commercial Steels, Cleveland, England, 1996.

29. Design Capacity Tables for Structural Steel, Second Edition, Volume 1: Open Sections (including Addendum No. 1); Australian Institute of Steel Construction, 1994/1997.

HERA STEEL DESIGN AND CONSTRUCTION BULLETIN No.51
ORDER FORM FOR REFERENCED ITEMS August 1999
(Side 1)

Name: ______________________
Company: ______________________
Postal Address: ______________________
Telephone: ______________ Fax: ______________
HERA Membership Number: ______________ Order No: ______________

Please specify method of payment by ticking the relevant box :

Please invoice ☐ (Members only) Cheque enclosed ☐ (Payable to "HERA") Visa ☐ Mastercard ☐

Credit Card Number ☐☐☐☐ ☐☐☐☐ ☐☐☐☐ ☐☐☐☐

Name on card: ______________ Expiry Date: ______________ Signature: ______________

The quoted prices are for items available ex stock and include GST at 12.5%.
<u>Note</u>. HERA does not lend items or supply Standards to non-members.

Reference 20 is not listed on this Order Form.

REF NO.	ITEM	UNIT PRICE ORDINARY & AFFILIATE	ASSOCIATE MEMBERS	NON-MEMBERS	QTY	TOTAL PRICE
HERA Reports:						
5	R4-91: *Notes prepared for a seminar on design of steel buildings*					
	for fire emergency conditions	$ 45.00	$ 50.00	$ 60.00		
8	R4-100:1999 *Structural steelwork connections guide*	$ 60.00	$ 70.00	$ 70.00		
13	R4-96:1998 *Structural steelwork estimating guide*	$ 60.00	$ 70.00	$ 70.00		
17	R4-99: *Specification for the fabrication, erection and surface treatment*					
	of structural steelwork (includes computer diskette)	$ 45.00	$ 50.00	$ 60.00		
Standards:						
6	DD ENV 1991-2-2, Eurocode 1 ...*Actions on structures exposed to fire*	On Loan	On Loan	See **Note**		
14	SAA HB77.6 *Australian bridge design code*					
	Section 6: Steel and composite construction	$ 141.75	$ 157.50	See **Note**		
16	NZS 3404:1997 *Steel structures standard*	$ 225.09	$ 250.10	See **Note**		
19	AS/NZS 2312:1994 *Guide to the protection of iron and steel*					
	against exterior atmospheric corrosion	$ 96.75	$ 107.50	See **Note**		
22	NZS 4203:1992 *General structural design*					
	and design loadings for buildings	$ 153.99	$ 171.11	See **Note**		
24	AS/NZS 1252:1996 *High-strength steel bolts...structural engineering*	$ 49.35	$ 54.84	See **Note**		
25	AS/NZS 1112:1996 *ISO Metric hexagon nuts...*	$ 45.40	$ 50.45	See **Note**		
26	AS1237-1973 *Flat metal washers for general engineering purposes*	$ 19.73	$ 21.93	See **Note**		

Subtotal carried forward ______

Please Turn Over. Published Papers and Other Items are Listed Overleaf.

HERA STEEL DESIGN AND CONSTRUCTION BULLETIN
ORDER FORM FOR REFERENCED ITEMS

No.51
August 1999

(Side 2)

The quoted prices are for items available ex stock and include GST at 12.5%.
Note. HERA does not lend items or supply Standards to non-members.

REF NO.	ITEM	UNIT PRICE ORDINARY & AFFILIATE	ASSOCIATE MEMBERS	NON-MEMBERS	QTY	TOTAL PRICE
Other Items Available Through HERA:						
4	*Natural fires in large scale compartments*	On Loan	On Loan	See **Note**		
7	*Fire engineering design guide*	On Loan	On Loan	See **Note**		
9	*Economical carparks - A guide to fire safety*	On Loan	On Loan	See **Note**		
10	*Composite steel road bridges - Concepts and design charts*	On Loan	On Loan	See **Note**		
11	*Coatings guide for new steel bridges*	On Loan	On Loan	See **Note**		
12	*Durability of steel bridges: Survey of performance...protective coatings*	On Loan	On Loan	See **Note**		
15	*Transit New Zealand bridge manual: Design and evaluation*	On Loan	On Loan	See **Note**		
21	*The effect of lateral and local buckling in steel beams...*	On Loan	On Loan	See **Note**		
27	*Bolted end plate connections for seismically designed steel frames*	On Loan	On Loan	See **Note**		
29	*Design capacity tables for structural steel, Vol 1, Open sections*	$ 88.65	$ 98.50	$ 118.20		

Published Papers:
In accordance with the Copyright Act 1994, copies of papers may be made only for the purpose of research or private study. The Copyright Act prohibits the sale, letting for hire or further copying of any such copy. To obtain copies of papers please sign and date the declaration below and indicate what you require by ticking the applicable reference. Single copies only will be supplied.

Copyright Declaration
Please provide me with a copy of the following. I declare that I will not use this material for any purpose other than research or private study and will not supply any copies to any other person.

Signed ______________________ Date ______________

REF NO.	ITEM	UNIT PRICE HERA MEMBERS	NON-MEMBERS	TICK IF REQD	TOTAL PRICE
18	*Prediction of site-specific corrosion rates in New Zealand...*	No Charge	$ 15.00		
23	*Structural assessment of corrosion-damaged steel beams...*	No Charge	$ 15.00		
				Subtotal brought forward from front of sheet	
				GRAND TOTAL	

Items Available from Other Sources:

1	*Draft Acceptable Solutions for Fire Safety* - Standards New Zealand, Private Bag 2439, Wellington
2	*Acceptable Solutions for Fire Safety* - Standards New Zealand, Private Bag 2439, Wellington
3	*New Zealand Building Code:1992* - Standards New Zealand, Private Bag 2439, Wellington
28	*Structural sections to BS4:Part 1 and BS4848:Part 4* - British Steel NZ Ltd, Stephen Stickland, phone 0-9-634 1179, fax 0-9-634 2901

Post to: HERA Information Centre, P.O. Box 76-134, Manukau City; or Fax to: 0-9-262 2856
Please return both sides of this form so that we can identify you.