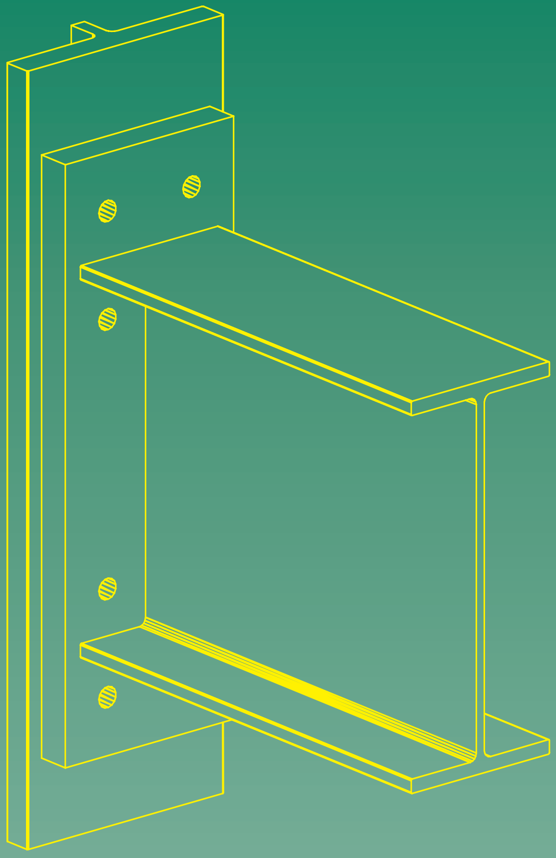
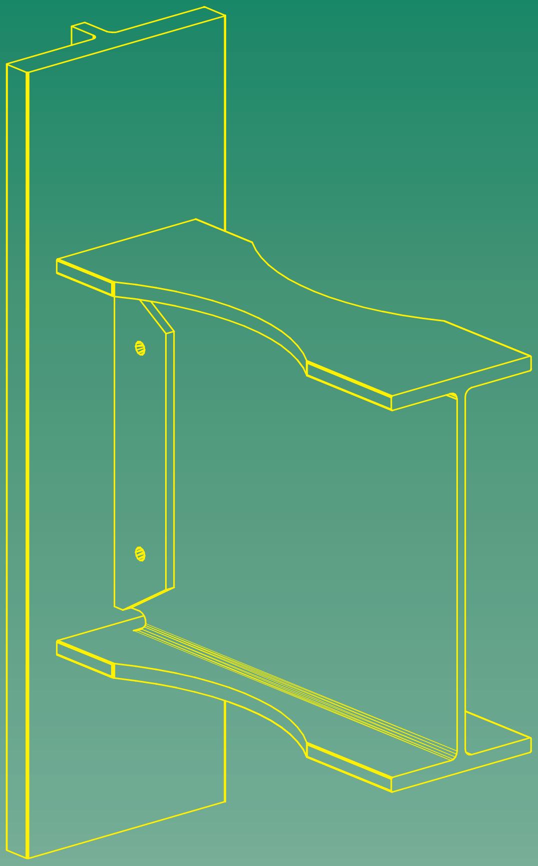
Moment Connections for Seismic Applications





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Canadian Institute of Steel Construction
Institut canadien de la construction en acier

MOMENT CONNECTIONS
FOR
SEISMIC APPLICATIONS

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FOREWORD

The Canadian Institute of Steel Construction is a national industry organization representing the structural steel, open-web steel joist and steel plate fabricating industries in Canada. Formed in 1930 and granted a Federal charter in 1942, the CISC functions as a nonprofit organization promoting the efficient and economic use of fabricated steel in construction.

As a member of the Canadian Steel Construction Council, the Institute has a general interest in all uses of steel in construction. CISC works in close co-operation with the Steel Structures Education Foundation (SSEF) to develop educational courses and programmes related to the design and construction of steel structures. The CISC supports and actively participates in the work of the Standards Council of Canada, the Canadian Standards Association, the Canadian Commission on Building and Fire Codes and numerous other organizations, in Canada and other countries, involved in research work and the preparation of codes and standards.

Preparation of engineering plans is not a function of the CISC. The Institute does provide technical information through its professional engineering staff, through the preparation and dissemination of publications, through the medium of seminars, courses, meetings, video tapes, and computer programs. Architects, engineers and others interested in steel construction are encouraged to make use of CISC information services.

This booklet has been prepared and published by the Canadian Institute of Steel Construction. It is an important part of a continuing effort to provide current, practical, information to assist educators, designers, fabricators, and others interested in the use of steel in construction.

Although no effort has been spared in an attempt to ensure that all data in this book is factual and that the numerical values are accurate to a degree consistent with current structural design practice, the Canadian Institute of Steel Construction does not assume responsibility for errors or oversights resulting from the use of the information contained herein. Anyone making use of the contents of this book assumes all liability arising from such use. All suggestions for improvement of this publication will receive full consideration for future printings.

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Revisions

Technical revsions to this publication are indicated in the body of the text by a vertical line and revision date in the left margin beside the altered item. This version of the publication supercedes all previous versions posted on the CISC website. Future revisions to this publication will be posted on the CISC website. Users are encouraged to visit the CISC website periodically for updates.

Table of Contents

Foreword iii

Background vi

Introduction viii

Scope ix

1 General

1.1 General Design Objective 1

2 Materials

2.1 Structural Steel 1

2.2 Bolts 1

2.3 Welds 1

3 General Design Procedures

3.1 Probable Moment Capacity at Plastic Hinges 2

3.2 Shear at Plastic Hinges 3

3.3 Strength Demands at Critical Sections 3

3.4 Panel Zone Shear 3

3.5 Continuity Plates 4

3.6 Lateral Support for Beams 4

4 Bolted Unstiffened End Plate Connection

4.1 Design Procedure 5

4.2 Welded Joints 8

5 Bolted Stiffened End Plate Connection

5.1 Design Procedure 11

5.2 Welded Joints 13

6 Reduced Beam Section Connection

6.1 Design Procedure 16

6.2 Welded Joint Details 18

6.2.1 Weld Access Holes 18

6.2.2 Backing Bars 19

6.2.3 Welding Sequence for Bottom Beam Flange 19

7 General Welded Joint Details

7.1 Run-off tabs 21
  7.1.1 Use of Run-off Tabs 21
  7.1.2 Run-off Tab Removal and Finish 21
7.2 Doubler Plate Details 21
7.3 Continuity Plate Details 22

References 23

Figures

3.2 – Shear at plastic hinges 25
3.3 – Strength demands at critical sections 25
4 – Bolted unstiffened end plate (BUEP) connection 26
5 – Bolted stiffened end plate (BSEP) connection 27
6 – Reduced beam section (RBS) connection 28
6.1 – Weld access hole dimensions (as provided in W59-03) 30
7.2 – Column web doubler plate details 31
7.3(a) – Continuity plates (shown as one-sided connection) 32
7.3(b) – Column with continuity plates and extended (single) web doubler plate 33
7.3(c) – Column with continuity plates and flush (single) web doubler plate 34

Background

Steel moment-resisting frames have become a popular seismic load-resisting system because steel is a well-known ductile material with a high strength-to-mass ratio, and because moment frames offer a high degree of redundancy. They became the most popular framing system for the construction of multi-storey buildings in the U.S. west coasts following the 1971 San Fernando earthquake in which steel moment-frame buildings demonstrated their superior performance versus other forms of construction.

Since the early 1970's, designers in California have routinely specified a type of beam-to-column moment connection for steel moment frames that features field-welded flanges and a shop-welded shear tab, field-bolted to the web (now referred to as the pre-Northridge connection). The 1988 Uniform Building Code adopted this connection type as the pre-qualified (prescriptive) moment connection design for seismic applications, representative of prevailing west coast practice at the time.

The Northridge earthquake of January 17, 1994, caused widespread building damage throughout some of the most heavily populated communities of Southern California. Following the earthquake, cracked welded moment connections were discovered in a number of steel moment frame buildings in the area. Since the detail description of these cracks is well documented in the literature they will not be repeated here. The Federal Emergency Management Agency, FEMA, in cooperation with others, conducted a comprehensive study of the situation and gave a concise report in a document, FEMA-354 (2000e): "…. *Much of the damage sustained was quite predictable, occurring in types of buildings that engineers had previously identified as having low seismic resistance and significant risk of damage in earthquakes. This included older masonry and concrete buildings, but not steel framed buildings. Surprisingly, however, a number of modern, welded, steel, moment-frame buildings also sustained significant damage……… Initial reports from this mandatory inspection program erroneously indicated that nearly every one of these buildings had experienced damage and in some cases, that this damage was extensive. It was projected that perhaps thousands of buildings had been damaged. It is now known that damage was much less widespread than originally thought and that many of the conditions that were originally identified as damage actually were imperfections in the original construction work. Of the nearly 200 buildings that were inspected under the City of Los Angeles ordinance, it now appears that only about 1/3 had any actual earthquake damage and that more than 90% of the total damage discovered occurred within a small group of approximately 30 buildings….."* .

In general, steel moment-frame buildings damaged by the Northridge earthquake met the basic intent of the building codes (FEMA 2000a). That is, they experienced limited structural damage, but did not collapse. However the structures did not behave as anticipated. Steel ductile moment-resisting frames (recognized as the most ductile system and commonly referred to as Special Steel Moment Frames in U.S.) are expected to undergo extensive yielding and inelastic deformation, without significant loss of strength. Damage is expected to consist of yielding and local buckling but not brittle fracture of connections. In response to the concerns raised by these damage discoveries, FEMA sponsored a programme of directed investigation and development to identify the cause of the damage, quantify the risk inherent in steel structures and develop practical and effective engineering criteria for mitigation of this risk. FEMA contracted with the SAC Joint Venture[*] to carry out this exhaustive study. The resulting FEMA/SAC project was conducted over a period of 6 years at a cost of $12 million (U.S.). Hundreds of leading practicing engineers, university researchers, industry associations, contractors, material suppliers, inspectors and building officials in the U.S. participated in the project. This project culminated with the publication of 4 engineering practice guideline documents (FEMA 2000a, b, c, d). These resource documents provide state-of-the-art recommendations for:

1) design of new steel moment-frame buildings, FEMA-350,
2) evaluation and upgrade of existing welded steel moment-frame buildings, FEMA-351,
3) post-quake evaluation and repair of welded steel moment-frame buildings, FEMA-352,
4) quality assurance guidelines for steel moment-frame construction for seismic applications, FEMA-353 (FEMA 2000a, b, c, d).

FEMA-350 and FEMA-353 propose guidelines for new construction. FEMA-350 also contains a detailed procedure for performance evaluation. Extensive studies in the FEMA/SAC project have lead to the conclusion that regular and well-configured steel moment-frame structures designed in accordance with FEMA design criteria and constructed in accordance with FEMA-353, provide in excess of 90% confidence of withstanding *Maximum Considered Earthquake* demands without global collapse. It should be noted that the application of these guidelines is expected to result in systems having performance capabilities that are desirable, and superior to other systems of all construction materials designed and built in accordance with the model code minimum requirements.

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[*] SAC is a joint venture of the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC), and California Universities for Research in Earthquake Engineering (CUREE).

Introduction

Clause 27 of CSA Standard S16-01 (herein and after referred to as S16-01) stipulates performance criteria for beam-to-column connections for *Ductile (Type D) Moment-Resisting Frames* and for *Moderately Ductile (Type MD) Moment-Resisting Frames* and requires that they be met by means of physical testing. Connections for *Limited-Ductility (Type LD) Moment-Resisting Frames* may either be qualified by testing or be proportioned and detailed in accordance with the specific requirements provided in Clause 27.4.

Over 150 connection assemblies have been tested in the U.S as part of the SAC Joint Venture Program (FEMA 2000a). Several connection types have been pre-qualified for *Special Moment-Resisting Frame* connections (FEMA 2000a) whose criteria of acceptance are similar to and somewhat exceed that for connections of *Type D* frames. Hence, these connections meet both the requirements for *Type D* and the less demanding requirements for *Type MD* connections. It has also been demonstrated in the SAC Program that when these connection types are used in *Ordinary Moment-Resisting Frame* connections (FEMA 2000a), certain restrictions of application can be relaxed. Since the criteria of acceptance for *Ordinary Moment-Resisting Frame* connections are similar to that for connections of *Type LD* frames, they are considered suitable for used in *Type LD* frames.

This CISC publication concentrates on the design criteria for some of the beam-to-column connections provided in FEMA 350 'Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings' (FEMA 2000a).

Scope

Several types of rigid connection (*fully-restrained*) have been prequalified by FEMA for seismic applications. FEMA-350 'Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings' (FEMA 2000a) provides design criteria, size and other limitations for their use. These prequalified connections apply to frames consisting of wide-flange beams and columns, subjected to strong axis bending only. They are also restricted for use in frames using columns within the depth of W360 sections[*]. This CISC publication covers three of these connection types:

1) Bolted unstiffened end plate connection
2) Bolted stiffened end plate connection and
3) Reduced Beam Section connection.

Extended end plate connections have been a common form of field-bolted moment connection used world-wide whereas the Reduced Beam Section connections have recently emerged to be a popular form of welded moment connection specially suited for seismic applications. While it is believed that the connection types included here cover most practical applications in Canada, the CISC does not intend to restrict or discourage the use of other connection types that are proven suitable by physical tests in accordance with Appendix J of S16-01. It should also be noted that the limitations and restrictions for their use given here have been established according to FEMA-350 and hence connections that do not comply with these limitations and restrictions but are proven appropriate by physical tests can also be used. Other connections such as column splices and column bases are beyond the scope of this publication.

In spite of fundamental differences in constructional welding practices between Canada and the U.S. which have existed historically, FEMA essentially ratifies the fundamentals in the Canadian welding system. The traditional third-party verification programmes stipulated in CSA standards W47.1 and W48 precede much of the *Quality Assurance* protocol, proposed by FEMA. On the other hand, resource documents, FEMA-350 and FEMA-353, attempt to cover the entire fabrication process. FEMA-353 proposes a matrix of obligations based on weld demand, weld failure consequence and direction of load effects under seismic loads, including weld testing obligations and weld inspector qualifications in a 'fitness-for-purpose' format. These FEMA documents cover matters beyond the scope of W47.1 and W48 and, reflecting U.S. practice, provide specific requirements that may differ from Canadian welding and weld inspection requirements. With a few exceptions, individual prequalified joints and procedural control on making each prequalified joint are essentially interchangeable between CSA standard W59 and its U.S. counterpart, AWS D1.1. But the procedural control for the entire connection, as proposed by FEMA, is beyond the scope of W59 and D1.1 standards.

QA and QC of welds are beyond the scope of this publication, except that the essence of FEMA recommendations for non-destructive testing (NDT) for welds is provided (in grey fonts).

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[*] For recent test results on deep column connections, see Ricles et al (Ricles 2004)

1 General

This publication provides detailed description, design requirements and limitations of application for three types of moment-resisting connections:

1) Bolted unstiffened end plate connection
2) Bolted stiffened end plate connection
3) Reduced Beam Section connection (field welded).

1.1 General Design Objective

Well-proportioned moment-resisting connections provide large, stable, plastic rotational capacity. Certain modes of behaviour, such as beam flange yielding and panel zone yielding, are ductile whereas others may not be. The design objective aims at mobilizing yielding of at least one ductile element while precluding any undesirable failure modes.

2 Materials

2.1 Structural Steel

Clause 27.1.5 of CSA Standard S16-01 (herein and after referred to as S16-01) generally applies. In addition, column steel should conform to ASTM A992, A572 Grade 50, A913 Grade 50 or A913 Grade 65 steel. All beams should be ASTM A992 or A572 Grade 50 steel, except A913 Grade 50 steel is also permitted, provided:

a) the mill-test value of yield strength does not exceed 85% of mill-test value of tensile strength and
b) the mill-test value of yield strength does not exceed 450 MPa.

Steel grades for end plates and end plate stiffeners should meet the specific requirements as given in each respective design procedure for bolted end plate connections.

2.2 Bolts

All bolts provided to resist seismic forces in these moment connections should conform to ASTM A325, A490 or A1852. A325M and A490M bolts are also permitted. They should be pretensioned and inspected in accordance with Clause 23.8 and Clause 23.9 of S16-01 respectively. The contact surfaces in bearing-type connections should be Class A or better.

2.3 Welds

Except where more stringent specific welding requirements are given in this publication, CSA Standard W59-03 (herein and after referred to as W59-03), excluding Clause 12, applies. Weld filler metals should conform to Clause 27.1.5.3 of S16-01. Section 7 outlines general welded joint details; specific requirements for the design of welded joints for each connection type are given in Sections 4 to 6.

3 General Design Procedures

Generally, the design procedures outlined in this document include the following:

1) Identify all undesirable failure modes as well as the primary yielding mechanism and any other yielding mechanisms
2) Determine the probable peak capacity of the primary yielding mechanism and in some cases the onset of yielding
3) Proportion the connection to ensure that the nominal resistances against all the undesirable failure modes (computed using a resistance factor, ϕ, = 1) at least equal the probable peak capacity of the primary yielding mechanism.
4) In order to achieve the most desirable sequence of yielding for connections in *Type D* and *Type MD* frames, proportion each connection to prevent yielding of any secondary yielding mechanism prior to the onset of yielding of the primary yielding mechanism.

NOTE: Units are in millimetres and Newtons unless otherwise noted.

This section describes the parameters and design procedures that apply to all 3 types of connections covered in this publication. Some of the parameters used in the equations contained in this Section are defined in Figure 3.3 and the remaining in S16-01.

3.1 Probable Moment Capacity at Plastic Hinges

For moment connections designed to develop plastic hinges in the beam or girder, the probable peak plastic moment capacity at the location of the plastic hinge should be determined as:

$$M_{pr} = C_{pr} R_y F_y Z_e$$

where:

M_{pr} = probable peak plastic hinge moment

C_{pr} = a factor to account for the effects of strain hardening, local restraint, additional reinforcement, and other connection conditions.

$$= \frac{F_y + F_u}{2F_y}$$

$R_y F_y$ = expected yield stress of the beam sections as defined in Clause 27.1.7 of S16-01.

Z_e = the effective plastic modulus of the beam section at the location of the plastic hinge. For Bolted end plate connections, Z_e = the plastic modulus of the unreduced beam section, Z_b.

3.2 Shear at Plastic Hinges

The shear at the plastic hinge should be determined by methods of statics, including the companion gravity loads acting on the beam. The shear at each plastic hinge may be determined by constructing a free-body diagram of the beam segment between plastic hinges. Figure 3.2 provides an example of such a simple analysis.

3.3 Strength Demands at Critical Sections

In order to complete the design of the connection, including, for example, selecting and proportioning the steel plates, bolts, and welds that make up the connection, it is necessary to determine the shear and flexural strength demands at each critical section. These forces may be calculated by taking a free-body diagram of that portion of the connection assembly located between the critical section and the plastic hinge. Figure 3.3 demonstrates this procedure for two critical sections.

3.4 Panel Zone Shear

Connections for *Type D* and *Type MD* frames should be proportioned to allow yielding to occur either as a combination of beam flexure yielding (primary yielding mechanism) and panel zone yielding (secondary yielding mechanism) or as beam flexure alone. Therefore, one-sided connections (as shown in Figure 7.3(a)) should have a panel zone thickness,

$$w' \geq \frac{C_y M_c \left(\frac{h - d_b}{h} \right)}{0.9(0.6\, R_{yc} F_{yc} d_c)(d_b - t_b)}$$

where:

h = average storey height of the storeys above and below the beam-to-column intersection, except

a) where the column below has a pinned base, h = sum of the storey height below and one-half the storey height above and

b) for top level connections, h = the storey height but twice the storey height where the column has a pinned base,

Mar 9, 2005

$$C_y = \frac{S_e}{C_{pr} Z_e}$$

M_c is defined in Figure 3.3,

$R_{yc}F_{yc}$ = the probable yield stress of the column section in accordance with Clause 27.1.7 of S16-01 and

S_e = effective elastic modulus of beam section at the plastic hinge location.

For two-sided connections (as shown in Figures 7.3(b) and (c)) simultaneous plastic hinging in both beams should be considered. When a web *doubler plate* having $F_y = F_{yc}$ is provided, w' is the sum of the column web thickness and the *doubler plate* thickness. Clause 27.2.4.3(a) of S16-01 applies and in calculating panel-size-to-thickness ratios the thickness may also be taken as w' provided the *doubler plate* is connected to the column web near the centre of the panel. (Sometimes, use of a column section with a thicker web instead of *doubler plates* leads to a more economical solution). For *doubler plate* details, see Section 7.

Mar 1, 2009

Panel zones in *Type LD* frames may be proportioned to meet either the requirement above or the requirements of Clauses 7.2 and 27.4.3 of S16-01.

3.5 Continuity Plates

Where continuity plates (also referred to as panel zone horizontal stiffeners and column transverse stiffeners) are required, the plates should, in addition to any specific requirements as stipulated in the design procedure for each connection type, meet the requirements below:

1) For two-sided connections, the thickness of the continuity plates should be at least equal to the thicker of the beam flanges (see Figure 7.3(b)).
2) For one-sided connections, the continuity plate thickness should be at least one-half of the thickness of the beam flange (see Figure 7.3(a)).

For continuity plate details, see Section 7. (Note: Use of a stronger column section that eliminates the need of continuity plates may lead to a more economical solution).

3.6 Lateral Supports for Beams

Beams should be laterally braced in accordance with Clause 27 of S16-01. Where the beam supports a concrete slab and is in direct contact with it along the beam span, lateral bracing provided directly or indirectly to the column flange may be considered an effective means of lateral support for the adjacent plastic hinge (FEMA 2000a and 2000i). Otherwise, supplemental bracing for the beam flanges near the plastic hinges should be provided. Where such supplemental bracing is provided, Clause 27.2.8 of S16-01 should also be considered.

4 Bolted Unstiffened End Plate Connection

The bolted unstiffened-end-plate (BUEP) connection is made by shop-welding the beam to an end plate, extended above and below the flanges. The beam flange-to-plate joints are complete-penetration-groove-welded joints whereas fillet welds or complete-joint-penetration-groove (CJPG) welds connect the beam web to the plate. The end plate is then field-bolted to the column, using eight bolts. The CJPG weld of each beam flange is made without a weld access hole and without using any backing bar. This type of connection can be used in *Type D, Type MD* and *Type LD* systems within the member size limitations given in Table 4. Figure 4 presents typical details for the connection.

4.1 Design Procedure

The connection should be proportioned to allow yielding to occur either as a combination of beam flexure and panel zone yielding or as beam flexure alone. Panel zone yielding that is not accompanied by beam flexure yielding is permitted for connections in *Type LD* frames provided the requirements of Clauses 7.2.6 and 27.4.3 of S16-01 are met. The end plate, bolts and welds should be designed so that no significant yielding occurs in these elements. The connection should be proportioned following the steps below. Most of the parameters used in the equations are defined in Figure 4 and the remaining in S16-01.

Mode 1: Bolt Tension

Bolt tension failure should be precluded by selecting bolt type and bolt size to resist M_{cf}, satisfying:

$$0.75 A_b F_u \geq \frac{M_{cf}}{2(d_1 + d_2)}$$

where:

M_{cf} is defined in Figure 3.3. and
d_1 and d_2 are defined in Figure 4.

Mode 2: Bolt Shear

Bolt shear failure should be precluded by satisfying:

$$3A_b(0.5F_u) \geq V_{cf}$$

where V_{cf} is defined in Figure 3.3.

The bolt threads should not intercept the shear plane.

Mode 3: End Plate Flexure

To preclude end plate flexural yielding, the end plate thickness t_p should satisfy:

$$t_p \geq \sqrt{\frac{M_{cf}}{0.8F_{yp}\left\{(d_b - p_t)\left[\frac{b_p}{2}\left(\frac{1}{p_f}+\frac{1}{s}\right)+(p_f+s)\frac{2}{g}\right]+\frac{b_p}{2}\left(\frac{d_b}{p_f}+\frac{1}{2}\right)\right\}}}$$

where $s = \sqrt{b_p g}$

The end plate should be CSA G40.21 300W or ASTM A36 steel. In either case, F_{yp} should be taken as 250 MPa.

Mode 4: End Plate Shear

End plate shear yielding should be precluded by selecting end plate thickness satisfying:

Mar 9, 2005

$$t_p \geq \frac{M_{cf}}{1.1F_{yp}b_p(d_b - t_b)}$$

where F_{yp} should be taken as 250 MPa.

Mode 5 a: Beam Flange Tension Effect on Column Flange without Continuity Plates

If the column flange thickness t_c satisfies the equation below, proceed to check Mode 6:

$$t_c \geq \sqrt{\frac{\left(\frac{M_{cf}}{d_b - t_b}\right)C_1}{2F_{yc}c}}$$

where:

c is defined in Figure 4,

$C_1 = \frac{g}{2} - k_1$

k_1 = distance from centreline of column web to flange toe of fillet as provided in Table 4.1 and in Part 6 of the *CISC Handbook.*

If the column flange is thinner than required, then continuity plates should be provided. Continuity plates, if required, should be proportioned in accordance with Section 3.5.

Mode 5 b: Beam Flange Tension Effect on Column Flange - Continuity Plates Provided (Requirement for absolute minimum column flange thickness)

If continuity plates are provided and the column flange thickness, t_c, satisfies the equation below, proceed to check Mode 7. Otherwise, select a column section with adequate flange thickness or consider other connection types.

$$t_c \geq \sqrt{\frac{\dfrac{M_{cf}}{2(d_b - t_b)}}{0.8F_{yc}Y_c}}$$

where:

$$Y_c = \left(\frac{c}{2} + s\right)\left(\frac{1}{C_2} + \frac{2}{C_1}\right) + (C_2 + C_1)\left(\frac{4}{c} + \frac{2}{s}\right)$$

$$C_1 = \frac{g}{2} - k_1$$

$$C_2 = \frac{b_c - g}{2}$$

$$s = \sqrt{\frac{C_1 C_2}{C_2 + 2C_1}(2b_c - 4k_1)}$$

Mode 6: Beam Flange Compression Effect on Column without Continuity Plates

Continuity plates in accordance with Section 3.5 should also be provided if the column web thickness,

$$w_c < \frac{M_{cf}}{(d_b - t_b)(6k_e + 2t_p + t_b)F_{yc}}$$

where k_e is the k-distance of the column section for engineering design. k_e-values for many column sections are provided in Table 4.1 (*Note: These tabulated values are consistent with the lower k-values provided for engineering design in the AISC Manual of Steel Construction, LRFD 3rd Edition*).

Mode 7: Panel Zone Shear

The panel zones should be proportioned in accordance with Section 3.4. For purposes of this calculation, d_b may be taken as the distance from one edge of the end plate to the centre of the beam flange at the opposite flange.

4.2 Welded Joints

The beam flange-to-plate joints should be complete-penetration-groove-welded joints. These joints have special requirements, as access holes are not permitted. Weld each single-bevel T-joint in the following sequence:

a) install an 8-millimetre fillet on the inner flange face, serving as backing,
b) gouge root of backing to sound metal, then
c) complete the groove weld in a horizontal or flat position.

Alternatively,

a) deposit the groove weld with a minimum root opening,
b) back-gouge root area, except the web-flange junction area, to sound metal, then
c) back weld with an 8-millimetre fillet weld.

The beam web-to-end plate joint should be either a CJPG welded joint or a fillet-welded joint. A fillet-welded web should have welds on both sides of the web and the welded web connection should be proportioned to resist the more severe load effect of the following:

a) flexural yielding capacity of the web,
b) $2V_{cf}$, where V_{cf} is defined in Figure 3.3.

Mar 9, 2005

In determining the fillet weld resistance against load effect (a), the value of V_r as defined in Clause 13.13.2.2 b) of S16-01 should be taken as $0.67\phi_w A_w X_u$.

Table 4
Bolted Unstiffened End Plate Connection - Summary of Requirements and Limitations

General	
Applicable systems	*Type D*, *Type MD* and *Type LD* frames
Hinge location distance, x	$t_p + d_b/3$
Critical Beam Parameters	
Maximum depth (nominal)	W610 for *Type D* and *Type MD* W760 for *Type LD*
Minimum span-to-depth ratio	*Type D and Type MD*: 7 *Type LD*: 5
Maximum flange thickness	19 mm
Permissible material specifications	Section 2.1
Critical Column Parameters	
Depth range (nominal)	*Type D* and *Type MD*: W200, W250, W310, W360 *Type LD*: not limited
Minimum flange thickness	Section 4.1, Modes 5a and 5b
Permissible material specifications	Section 2.1
Beam/Column Relations	
Panel zone strength	Section 4.1
Column/beam bending strength ratio	*Type D* and *Type MD*: Clause 27.2.3.2 of S16-01
Connection details	
Bolts	
Bolt diameter	$\leq 1^1/_2$ inch; see Section 4.1, Modes 1 and 2
Bolt types	Section 2.2
Installation requirements	Pretensioned, see Section 2.2
Washers	Single F436 when required
Oversized, slotted holes	Not permitted
End plate	
End plate thickness	Section 4.1, Modes 3 and 4
End plate material	CSA G40.21 300W, ASTM A36
Flange welds	
Weld type	CJPG weld, 10 mm fillet used as backing, root back gouged prior to start of groove weld. See Figure 4
Filler metal	Section 2.3
Weld access holes	Not permitted
Web connection	Figure 4
Continuity plates	Sections 3.5, 4.1 and 7

Table 4.1

k_e and k_1 distances for engineering calculations and othr cross-section dimensions

Wide flange column sections

Designation	d	b	t	w	k_e	k_1
	mm	mm	mm	mm	mm	mm
W360						
x1086	569	454	125	78.0	140	69
x990	550	448	115	71.9	130	66
x900	531	442	106	65.9	121	63
x818	514	437	97.0	60.5	112	60
x744	498	432	88.9	55.6	104	58
x677	483	428	81.5	51.2	97	56
W360						
x634	474	424	77.1	47.6	92	54
x592	465	421	72.3	45.0	88	53
x551	455	418	67.6	42.0	83	51
x509	446	416	62.7	39.1	78	50
x463	435	412	57.4	35.8	73	48
x421	425	409	52.6	32.8	68	46
x382	416	406	48.0	29.8	63	45
x347	407	404	43.7	27.2	59	44
x314	399	401	39.6	24.9	55	42
x287	393	399	36.6	22.6	52	41
x262	387	398	33.3	21.1	49	41
x237	380	395	30.2	18.9	45	39
x216	375	394	27.7	17.3	43	39
W360						
x196	372	374	26.2	16.4	41	38
x179	368	373	23.9	15.0	39	38
x162	364	371	21.8	13.3	37	37
W360						
x122	363	257	21.7	13.0	37	27
x110	360	256	19.9	11.4	35	26
x101	357	255	18.3	10.5	33	26
x91	353	254	16.4	9.5	31	25
W360						
x79	354	205	16.8	9.4	32	25
x72	350	204	15.1	8.6	30	25
x64	347	203	13.5	7.7	29	24

Designation	d	b	t	w	k_e	k_1
	mm	mm	mm	mm	mm	mm
W310						
x500	427	340	75.1	45.1	90	43
x454	415	336	68.7	41.3	84	41
x415	403	334	62.7	38.9	78	40
x375	391	330	57.2	35.4	72	38
x342	382	328	52.6	32.6	68	37
x313	374	325	48.3	30.0	64	35
W310						
x283	365	322	44.1	26.9	59	34
x253	356	319	39.6	24.4	55	33
x226	348	317	35.6	22.1	51	31
x202	341	315	31.8	20.1	47	30
x179	333	313	28.1	18.0	43	29
x158	327	310	25.1	15.5	40	28
x143	323	309	22.9	14.0	38	27
x129	318	308	20.6	13.1	36	27
x118	314	307	18.7	11.9	34	26
x107	311	306	17.0	10.9	32	26
W310						
x86	310	254	16.3	9.1	32	23
x79	306	254	14.6	8.8	30	22
W310						
x74	310	205	16.3	9.4	29	23
x67	306	204	14.6	8.5	27	22
x60	303	203	13.1	7.5	26	22
W250						
x167	289	265	31.8	19.2	45	25
x149	282	263	28.4	17.3	41	24
x131	275	261	25.1	15.4	38	23
x115	269	259	22.1	13.5	35	22
x101	264	257	19.6	11.9	32	21
x89	260	256	17.3	10.7	30	21
x80	256	255	15.6	9.4	28	20
x73	253	254	14.2	8.6	27	20
W250						
x67	257	204	15.7	8.9	28	20
x58	252	203	13.5	8.0	26	19

5 Bolted Stiffened End Plate Connection

The bolted stiffened-end-plate (BSEP) connection is made by shop-welding the beam to an end plate. The beam flange-to-plate joints are complete-penetration-groove-welded joints whereas fillet welds or CJPG welds connect the beam web to the plate. The end plate is then field-bolted to the column, using sixteen bolts. The CJPG weld of each beam flange is made without using a weld access hole and without backing. The end plate extensions at the top and bottom of the beam are each stiffened by a vertical stiffener plate that extends outward from the beam flanges. This type of connection can be used in *Type D, Type MD* and *Type LD* systems within the member size limitations given in Table 5. Figure 5 presents a typical detail for the connection.

5.1 Design Procedure

The connection should be proportioned to allow yielding to occur either as a combination of beam flexure and panel zone yielding or as beam flexure alone. Panel zone yielding that is not accompanied by beam flexure yielding is permitted for connections in *Type LD* frames provided the requirements of Clauses 7.2.6 and 27.4.3 of S16-01 are met. The end plate, bolts and welds should be designed so that no significant yielding occurs in these elements. The connection should be proportioned following the steps below. Most of the various parameters used in the equations are defined in Figure 5 and the remaining in S16-01.

Mode 1: Bolt Tension

Bolt tension failure should be precluded by selecting bolt type and bolt size to resist M_{cf}, satisfying:

$$0.75 A_b F_u \geq \frac{M_{cf}}{3.4(d_2 + d_3)} \quad \text{and}$$

$$0.75 A_b F_u \geq \frac{3.25 \times 10^{-6}\, p_f^{0.591} P_{cf}^{2.58}}{t_p^{0.895}\, d_{bt}^{1.91}\, t_s^{0.327}\, b_p^{0.965}} + T_b$$

where

Mar 9, 2005

T_b is the minimum bolt tension given in Table 7 of S16-01,

$$P_{cf} = \frac{M_{cf}}{d_b - t_b} \quad \text{and}$$

d_{bt} is the diameter of the bolts.

The stiffeners should be shaped and dimensioned as shown on Figure 5.

Mode 2: Bolt Shear

Bolt shear failure should be prevented by satisfying

$$6A_b(0.5F_u) \geq V_{cf}$$

where V_{cf} is defined in Figure 3.3.

The bolt threads should not intercept the shear plane.

Mode 3: End Plate Flexure

To preclude end plate flexural yielding, the end plate thickness t_p should at least equal the larger of

$$\frac{154 \times 10^{-6} p_f^{0.9} g^{0.6} P_{cf}^{0.9}}{d_{bt}^{0.9} t_s^{0.1} b_p^{0.7}} \quad \text{and}$$

$$\frac{267 \times 10^{-6} p_f^{0.25} g^{0.15} P_{cf}}{d_{bt}^{0.7} t_s^{0.15} b_p^{0.3}} ,$$

where d_{bt} is the diameter of the bolts.

The end plate should be CSA G40.21 300W or ASTM A36 steel, and the stiffener plates should be as least as thick as the beam web.

The column flanges should be at least as thick as the required end plate thickness.

Mode 4: End Plate Shear

End plate shear yielding is precluded as stiffener plates are provided. These stiffener plates should be dimensioned as shown in Figure 5. They should be clipped at the intersection of the end plate and the beam flange.

Mode 5: Beam Flange Tension Effect on Column Flange without Continuity Plates

Proceed to check Mode 6 if the column flange thickness t_c satisfies the equation below:

$$t_c \geq \sqrt{\frac{\alpha_m P_{cf} C_3}{0.9F_{yc}(3.5p_b + c)}} ,$$

where

$$\alpha_m = C_a \left(\frac{A_f}{A_w} \right)^{1/3} \left(\frac{C_3}{d_{bt}} \right)^{1/4}$$

$C_3 = \frac{g}{2} - \frac{d_{bt}}{4} - k_1$ and

C_a = 1.45 for A325 bolts and 1.48 for A490 bolts.

A_f = Area of beam tension flange

A_w = Area of beam web, clear of flanges

If the column flange is thinner than required, provide continuity plates in accordance with Section 3.5 and proceed to check Mode 7.

Mode 6: Beam Flange Compression Effect on Column without Continuity Plates

Continuity plates in accordance with Section 3.5 should also be provided if the column web thickness,

$$w_c < \frac{M_{cf}}{(d_b - t_b)(6k_e + 2t_p + t_b)F_{yc}}$$

where k_e is the k-distance of the column section suitable for engineering calculations. k_e-values for many column sections are provided in Table 4.1 (*Note: These tabulated values are consistent with the lower k-values provided for engineering design in the AISC Manual of Steel Construction, LRFD 3rd Edition*).

Mode 7: Panel Zone Shear

The panel zone should be proportioned in accordance with Section 3.4. For purposes of this calculation, d_b may be taken as the distance from one edge of the end plate to the centre of the beam flange at the opposite flange.

5.2 Welded Joints

The beam flange-to-plate joints should be complete-penetration-groove-welded joints. These joints have special requirements as access holes are not permitted. Weld each single-bevel T-joint in the following sequence:

a) install an 8-millimetre fillet on the inner flange face, serving as backing,
b) gouge root of backing to sound metal, then
c) complete the groove weld in a horizontal or flat position.

Alternatively,

a) deposit the groove weld with a minimum root opening,
b) backgouge root area, except the web-flange junction area, to sound metal, then
c) backweld with an 8-millimetre fillet weld.

The stiffeners should be welded to the end plate and the beam flange using CJP double-bevel groove welds. The beam web-to-end plate joint should be either a CJPG welded joint or a fillet-welded joint. A fillet-welded web should have welds on both sides of the web and the welded web connection should be proportioned to resist:

a) the flexural yielding capacity of the web and,
b) in combination with the welded stiffener-to-end-plate joints, $2V_{cf}$, where V_{cf} is defined in Figure 3.3.

Mar 9, 2005

In determining the fillet weld resistance against load effect (a), the value of V_r as defined in Clause 13.13.2.2 b) of S16-01 should be taken as $0.67\phi_w A_w X_u$.

Table 5
Bolted Stiffened End Plate Connection – Summary of Requirements and Limitations

General	
Applicable systems	*Type D*, *Type MD* and *Type LD* frames
Hinge location distance, x	$t_p + L_s$
Critical Beam Parameters	
Maximum depth (nominal)	W920
Minimum span-to-depth ratio	*Type D* and *Type MD*: 7 *Type LD*: 5
Maximum flange thickness	25 mm
Permissible material specifications	Section 2.1
Critical Column Parameters	
Depth range (nominal)	*Type D* and *Type MD*: W310, W360 *Type LD*: not limited
Minimum flange thickness	Section 5.1, Mode 5
Permissible material specifications	Section 2.1
Beam/Column Relations	
Panel zone strength	Section 5.1, Mode 7
Column/beam bending strength ratio	*Type D* and *Type MD*: Clause 27.2.3.2 of S16-01
Connection details	
Bolts	
Bolt diameter	$\leq 1^1/_2$ inch; see Section 5.1, Modes 1 and 2
Bolt grades	Section 2.2
Installation requirements	Pretensioned, see Section 2.2
Washers	Single F436 when required
Oversized, slotted holes	Not permitted
End plate	
End plate and stiffener dimensions	Section 5.1, Mode 3
End plate and stiffener material	CSA G40.21 300W, ASTM A36
Flange welds	
Weld type	CJPG weld, 10 mm fillet used as backing, root backgouged prior to start of groove weld. See Figure 5
Filler metal	Section 2.3
Weld access holes	Not permitted
Web connection	Figure 5
Continuity plates	Sections 3.5, 5.1 and 7

6 Reduced Beam Section Connection

The forces at a reduced beam section (RBS) connection are kept within its resistance by reducing the flexural capacity of the beam at a strategically selected location to ensure that yielding and plastic hinging occur in the beam. These connections feature circular radius cuts in both top and bottom flanges of the beam thereby reducing the flange area over a short segment near each end of the beam. No reinforcement other than weld metal is used to join the flanges of the beam to the column. Each beam flange is welded to the column using complete joint penetration groove welds. If a steel backing bar is used in the top flange joint it may be left in place whereas any backing bar used in the bottom flange joint should be removed. The web connection features a shear tab that may be bolted or welded in the field. Figure 6 provides detail requirements for this connection type. These connections can be used in *Type D* and *Type MD* frames within the limitations given in Table 6. When this type of connection is used, the elastic frame drift calculations should take into account the reduction in beam stiffness. This increase in drift varies typically from 7% to 9% for flange reductions of 40% and 50% respectively. In lieu of a detailed analysis that accounts for this stiffness reduction, increase the elastic drift (determined based on unreduced beam section properties) by 7% to 9% accordingly.

6.1 Design Procedure

The connection should be proportioned to allow yielding to occur either as a combination of flexural yielding of the reduced section and panel zone yielding or as reduced section yielding alone. The beam-flange-to-column joints and the beam web connection should be proportioned so that no significant yielding occurs in these elements. The connection should be proportioned following the steps below. Some of the parameters used in the equations are defined in Figure 3.3 and Figure 6 and the remaining in S16-01.

Mode 1: Connection Flexure

a) Select the length and location of the beam flange reduction, according to the following limits:

$$0.5b \leq a \leq 0.75b$$
$$0.65d \leq s \leq 0.85d$$

where a and s are as shown in Figure 6, and b and d are the beam flange width and depth, respectively. Try $a = 0.5b$ and $s = 0.65d$.

b) Select the depth of the cut, c, within the range of $0.20b$ and $0.25b$. Try

$$c = 0.20b.$$

The reduced flange width, taken as the flange width at the ends of the middle 2/3 of the reduced section, should be equal to or less than 14.6t *(Note that wide-flange sections* *whose gross flange dimensions meet the Class 2 limit as defined in S16-01 also satisfy this requirement, except that where plastic hinging away from the reduced section is anticipated, usually due to an unusual gravity load configuration, a Class 1 beam is required).*

c) Calculate Z_e of the reduced section *(For* $c = 0.2b$, $b_e = 0.6b$*).*

d) Determine the factored resistance of the reduced section and check its adequacy for the effect of the factored loads as stipulated in S16-01. Select a stronger beam section and repeat previous steps if necessary.

e) Calculate M_{cf} according to the method of Section 3 and Figure 3.3 using $C_{pr} = 1.15$.

Flexural failure of the connection should be precluded by satisfying

$$M_{cf} \leq R_y F_y Z_b$$

where

$R_y F_y = 385$ MPa and

Z_b is the plastic section modulus of the gross beam section.

If M_{cf} is greater than the limit, increase c and repeat previous steps. The value of c should not exceed $0.25b$.

f) Calculate M_{cf} and M_c based on the final RBS dimensions according to the methods of Section 3.

Mode 2: Connection Shear

Calculate the shear at the column face, V_{cf} according to the equation:

$$V_{cf} = \frac{2M_{cf}}{L - d_c} + V_g$$

where V_g = shear due to companion gravity loads factored in accordance with Clause 7.2.6 of S16-01

When a welded web joint is selected, the beam web should be welded to the column using complete-joint-penetration (CJP) groove welds. For a field-welded joint, the shear tab serves as a backing bar as well as a connection plate to receive erection bolts. In this application, the erection bolts and shop-applied shear-tab-to-column welds should be proportioned for erection loads as described in Figure 6. Erection bolts need not be pretensioned.

If a field-bolted shear tab is to be used, the shear tab connection should be designed for V_{cf}. The resistance of the bolted connection may be taken as the nominal resistance of bearing-type connections (ϕ = 1). The tab should be connected to the column using either CJP groove welds or full-depth fillets of 0.75 times the tab plate thickness on each side of the tab plate.

Mode 3: Panel Zone Shear

The panel zone should be proportioned in accordance with Section 3.4.

Continuity Plates

Continuity plates should be provided if the column flange thickness, in millimetres, is less than the greater of

$$0.4\sqrt{1.8\, b_b\, t_b \frac{R_{yb}F_{yb}}{R_{yc}F_{yc}}}, \quad \text{and} \quad \frac{b_b}{6}$$

where

b_b = beam flange width (unreduced), mm,
t_b = beam flange thickness, mm,
$R_{yb}F_{yb}$ = the probable yield stress of the beam section as defined in Clause 27.1.7 of S16-01, MPa, and
$R_{yc}F_{yc}$ = the probable yield stress of the column section as defined in Clause 27.1.7 of S16-01, MPa.

Continuity plates, when required, should be proportioned in accordance with Section 3.5.

A typical RBS connection in detail is shown in Figure 6.

6.2 Welded Joint Details

6.2.1 Weld Access Holes

Weld access holes should meet either the dimensional requirements stipulated in Figure 5.1 of W59-03 (also shown in Figure 6.1) or the dimensional details recommended in Figure 3-5 of FEMA-350 (FEMA 2000a). The access holes should be finished to a surface roughness not to exceed 13 µm, and should be free of notches and gouges. For this purpose, a notch or gouge is any depression deeper than the overall surface roughness. Measurement of surface finish values by visual appearance or tactile comparison is acceptable. Weld access holes should be inspected by means of magnetic particle testing (*MT*) or liquid penetrant testing (*PT*) and should be free of cracks.

Notches or gouges present from thermal cutting should be removed by grinding, faired to a slope of not more that 1:5 against a straight cut surface, or to a radius of not less than 10 mm if they are in the curved portion of the cut surface. The depth of notches and gouges that may be repaired by grinding is not limited, provided the required dimensions, including tolerances, of the access hole are maintained. Other defects may be repaired by welding. Prior to welding, the notch or gouge should be ground to provide a smooth contour with a radius not less than 10 mm. The repair area should be preheated to a temperature between 200°C and 290°C. Controlled hydrogen electrodes that meet H16 designator requirements should be used. A written repair welding procedure specification for the application should be followed. Following completion of welding, the area should be ground smooth and flush to meet the contour and finish requirements for the access holes, with fairing of the welded surface to adjoining surfaces. Access holes that have been repaired by welding should be inspected by means of *MT*.

6.2.2 Backing Bars

Backing bars used for welding the bottom beam flange to the column should not be left in place. Steel backing may be removed by air carbon arc cutting (AAC), grinding, chipping, or thermal cutting. The Process should be controlled to minimize gouging. Following backgouging, the root should be backwelded. A reinforcing fillet weld with a minimum leg size of 8 mm or the root opening plus 2 mm, whichever is larger, should be provided. The reinforcing fillet weld need not be ground. Following completion of the reinforcing fillet weld, the fillet weld and the immediately adjacent area should be inspected by means of *MT*. (*Note: W59-03 also requires that a fillet weld, not less than one-fourth the beam flange thickness but need not exceed 10 mm, be placed on top of each side of the groove weld in a T-joint subject to tension.*)

Non-fusible backing may be used in applications and locations where qualified. The backing material should be removed. If visual inspection of the root shows no unacceptable discontinuities, no backgouging and backwelding is required. Any unacceptable weld discontinuities should be removed by backgouging, the root backwelded as needed and the reinforcing fillet added as described above. The completed fillet weld and the immediately adjacent area should be inspected by means of *MT*.

6.2.3 Welding Sequence for Bottom Beam Flange

When welding the bottom beam flange to the column in the field, the following sequence should be followed:

1) Weld starts-stops directly under the beam web should be avoided.
2) Each layer should be completed across the full width of the flange before the next layer begins.
3) The starts-stops of any two consecutive layers should not fall on the same side of the beam web.

Table 6
Reduced Beam Section Connection – Summary of Requirements and Limitations

General	
Applicable systems	*Type D* and *Type MD* frames
Hinge location distance, *x*	$a + s/2$
Critical Beam Parameters	
Maximum depth (nominal)	W920 (maximum mass 446 kg/m)
Minimum span-to-depth ratio	7
Maximum $b/2t$	7.3, where b is taken as the flange width at the ends of the middle 2/3 of the reduced section
Maximum flange thickness	44 mm
Permissible material specifications	Section 2.1
Flange reduction parameters	Section 6.1
Critical Column Parameters	
Depth range (nominal)	W310, W360
Permissible material specifications	Section 2.1
Beam/Column Relations	
Panel zone strength	Section 3.4
Column/beam bending strength ratio	Clause 27.2.3.2 of S16-01
Connection details	
Web connection	Section 6.1 and Figure 6
Continuity plates	Sections 3.5, 6.1 and 7
Flange welds	Figure 6
Welding parameters	Sections 2.3, 6.2 and 7
Weld access holes	Figure 6.1 or as shown in Figure 3-5 of FEMA-350

7 General Welded Joint Details

7.1 Run-off tabs

7.1.1 Use of Run-off Tabs

When run-off tabs, also commonly referred to as extension bars and weld tabs, are used they should be oriented parallel to the joint preparation and to the weld direction. Where practicable, run-off tabs should extend beyond the edge of the joint a distance not less than the greater of the thickness of the part and 25 mm. Use of weld dams is prohibited. Non-fusible weld tabs may be used in applications and locations where qualified.

7.1.2 Run-off Tab Removal and Finish

Steel run-off tabs should be removed to within 3 mm of the base metal surface, except when used for welding continuity plates where removal to within 6 mm of the plate edge is acceptable, and the end of the finished weld. Acceptable methods of removal are air carbon arc cutting (AAC), grinding, chipping, and thermal cutting. The process should be controlled to minimize gouging.

The edges where the run-off tabs have been removed should be finished to a surface roughness of 13 µm or better. Measurement of surface finish values by visual appearance or tactile comparison is acceptable. Grinding to a flush condition is not required. The contour of the weld at the ends should provide a smooth transition, free of gouges, notches and sharp corners. A minimum radius at the corner need not be provided. Weld defects not more than 2 mm deep should be repaired to a transition slope not steeper than 1:5. Other weld defects should be repaired by welding in accordance with an applicable welding procedure specification.

Following removal, finishing and completion of any necessary repairs, the exposed ends of the weld should be inspected by means of magnetic particle testing (*MT*).

7.2 Doubler Plate Details

Web *doubler plates*, where required, may be welded in accordance with one of the two details shown in Figure 7.2.

When detailed in accordance with Figure 7.2(a), the edges of the *doubler plate* should be chamfered to an adequate bevel to facilitate access to the root of the weld. A detail that features a square-edge plate and square groove weld between the *doubler plate* and the column is not permitted. No grinding of the completed weld is required.

When detailed in accordance with Figure 7.2(b), the plate should be chamfered to accommodate the rolling fillets of the column section. The fillet weld, in both throat and leg size should not be less than the chamfer dimensions used for the *doubler plate*. No grinding of the completed weld is required.

7.3 Continuity Plate Details

Continuity plates should be clipped to clear the rolling fillets of the column section. Along the column flange, the clip should not exceed a distance of 13 mm beyond the published k_1 dimension. Along the web, the clip should extend a distance of at least 38 mm beyond the published *k* dimension for detailing. The clip should be detailed to facilitate suitable weld terminations for both the flange weld and the web weld. Except for straight cut bevel clips as shown in Figure 7.3(a), a re-entrant corner should be provided in accordance with W59-03.

The weld between the continuity plate and the column flange should be a CJP groove weld. If backing bars are used and remain in place, they should receive a reinforcing fillet weld between the backing bar and column flange. The fillet weld size need not exceed the minimum size requirements stipulated in W59-03. No fillet weld should be placed between the continuity plate and the backing bar.

Weld terminations near the column flange tips may be completed using run-off tabs. For welds connecting column flanges to continuity plates, weld terminations near the rolling fillets of the column section need not be made using run-off tabs. The use of small non-fusible run-off tabs, where qualified, to assist in weld terminations is permitted. Run-off tabs should be removed following completion of welding, but no grinding is required.

For one-sided connections (as shown in Figure 7.3(a)), the continuity plates may be fillet welded to the column flange at the free side.

Continuity plates may be connected to the column web or *doubler plate* using groove welds, fillet welds, or a combination of the two (See Figure 7.3). The welds should terminate at a distance of approximately 6 mm from the cut ends as shown on Figure 7.3(a).

Note: Thick continuity plates require large CJP plate-to-flange welds. The associated heat input and shrinkage may be undesirable with respect to connection performance as well as production. While research in continuity plate requirements continues, results of experimental tests conducted since the publication of FEMA-350 indicate that some of the requirements given above may be relaxed. Users are advised to stay abreast of the most current findings reported in the literature.

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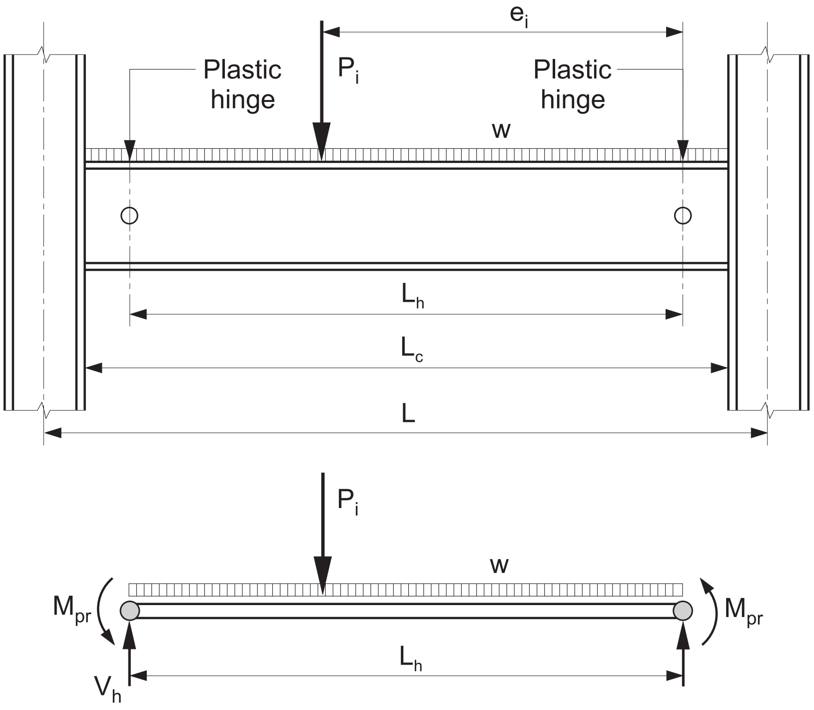
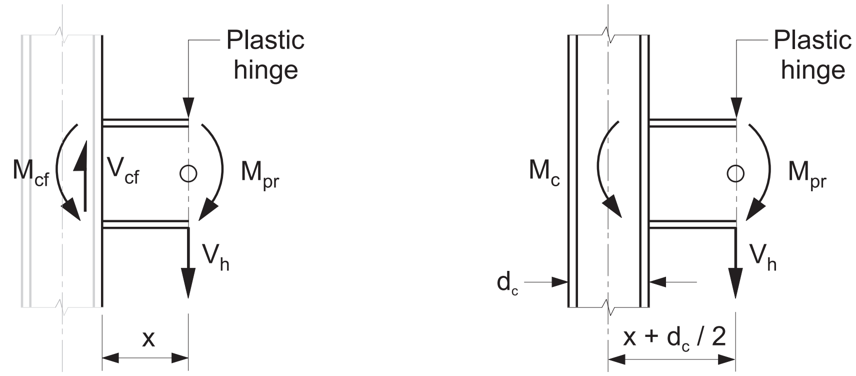


Free-Body Diagram of Beam Segment between Plastic Hinges

$$V_h = V_E + V_G$$
$$= \frac{2M_{pr}}{L_h} + \left(\frac{\Sigma e_i P_i}{L_h} + \frac{w L_h}{2} \right)$$

Figure 3.2
Shear at Plastic Hinges



$$M_{cf} = M_{pr} + V_h x$$
$$V_{cf} = V_h + xw$$

Critical Section at Column Face

$$M_c = M_{pr} + V_h (x + d_c / 2)$$

Critical Section at Column Centreline

Figure 3.3
Strength Demands at Critical Sections

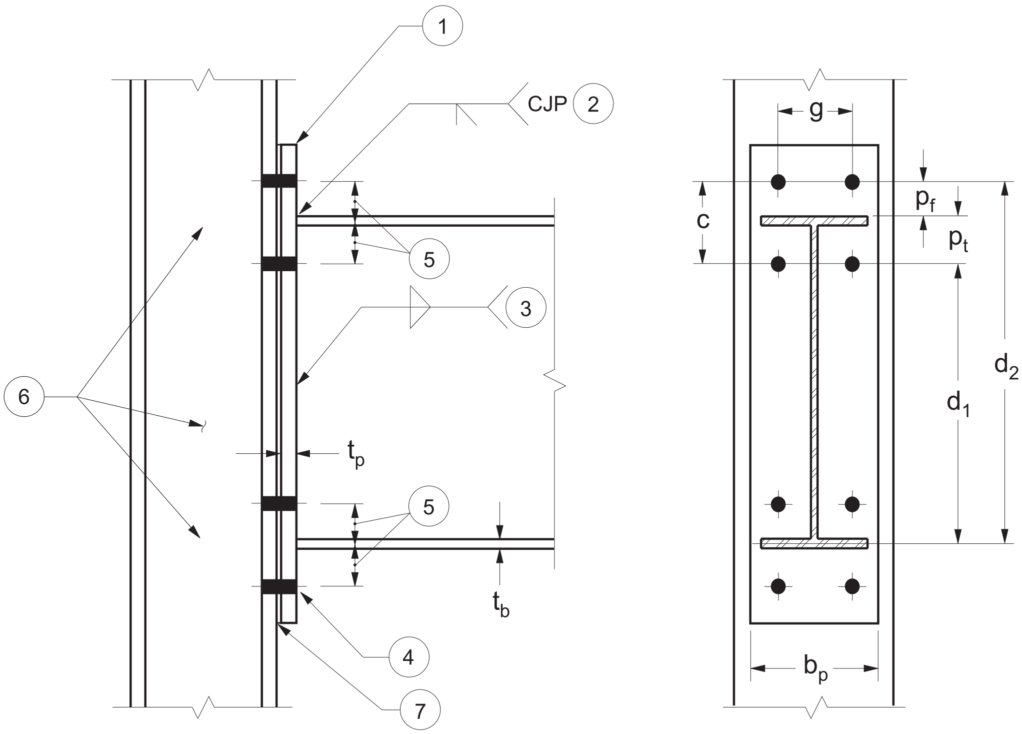


Figure 4
Bolted Unstiffened End Plate (BUEP) Connection

Footnotes:

1. End plate: See Section 4 and Table 4
2. This weld has special requirements. See Section 4.2. FEMA recommendation for NDT of weld: MT 100% of CJP joints, full length; UT 100% of CJP joints, full length. Discontinuities located at the weld root, at the web-to-flange intersection, should not be the cause for rejection.
3. Fillet weld both sides, or CJP weld. See Section 4.2 for sizing requirements. FEMA recommendation for NDT of weld: MT 100% of CJP joint; UT 100% of CJP joints (when acceptance rate is high reduce to 25% of joints); MT 25% of fillets. Test full length of welds, except for welds longer than 600 mm partial length testing applies.
4. Pretensioned ASTM A325 or A490 bolts. Diameter not to exceed $1^{1}/_{2}$ inches. See Section 4.1 for sizing requirements.
5. Bolt location is part of the end plate design. See Section 4.1.
6. For continuity plates, see Figure 7.3 and Sections 4.1 and 7. For web doubler plates and calculations of panel zone strength, see Sections 3.4 and 4.1.
7. Shim as required (Finger shims shall not be placed with fingers pointing up).

Feb 6, 2008

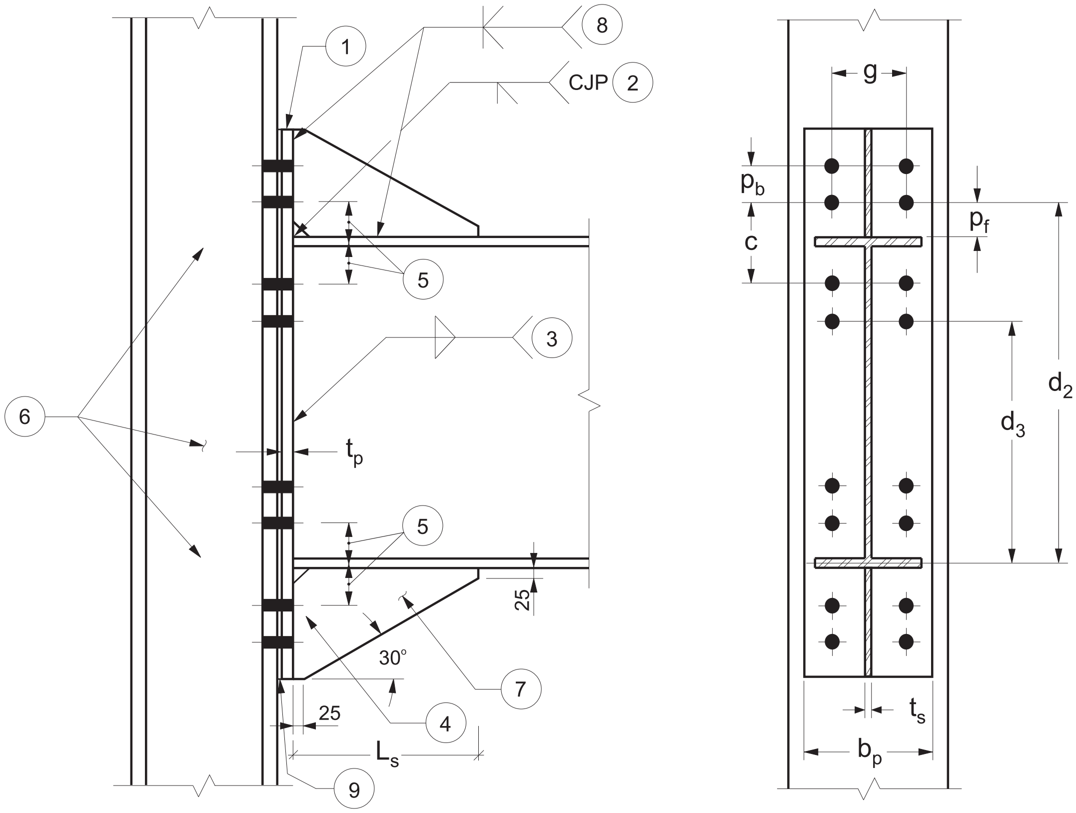


Figure 5
Bolted Stiffened End Plate (BSEP) Connection

Footnotes:

1. End plate: See Section 5 and Table 5
2. This weld has special requirements. See Section 5.2. FEMA recommendation for NDT of weld: MT 100% of CJP joints, full length; UT 100% of CJP joints, full length. Discontinuities located at the weld root, at the web-to-flange intersection, should not be the cause for rejection.
3. Fillet weld both sides, or CJP weld. See Section 5.2 for sizing requirements. FEMA recommendation for NDT of weld: MT 100% of CJP joint; UT 100% of CJP joints (when acceptance rate is high reduce to 25% of joints); MT 25% of fillets. Test full length of welds, except for welds longer than 600 mm partial length testing applies.
4. Pretensioned ASTM A325 or A490 bolts. Diameter not to exceed $1^{1}/_{2}$ inches. See Section 4.1 for sizing requirements.
5. Bolt location is part of the end plate design. See Section 5.1.
6. For continuity plates, see Figure 7.3 and Sections 5.1 and 7. For web doubler plates and calculations of panel zone strength, see Sections 3.4 and 5.1.
7. Stiffener is shaped as shown. Stiffener should not be thinner than the beam web.
8. Stiffener welds are CJP double-bevel groove welds to both the beam flange and the end plate. FEMA recommendation for NDT of weld: MT 100% and UT 100% for full length (for weld to beam, when acceptance rate is high reduce to 25% of joint).
9. Shim as required (Finger shims shall not be placed with fingers pointing up).

Feb 6, 2008

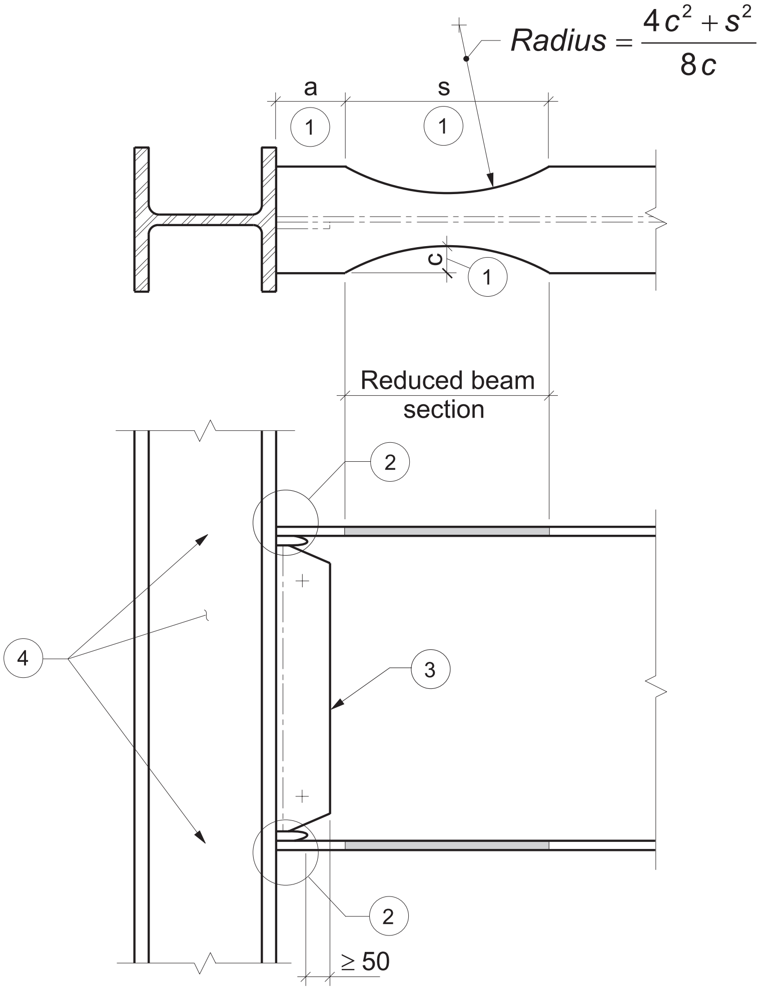


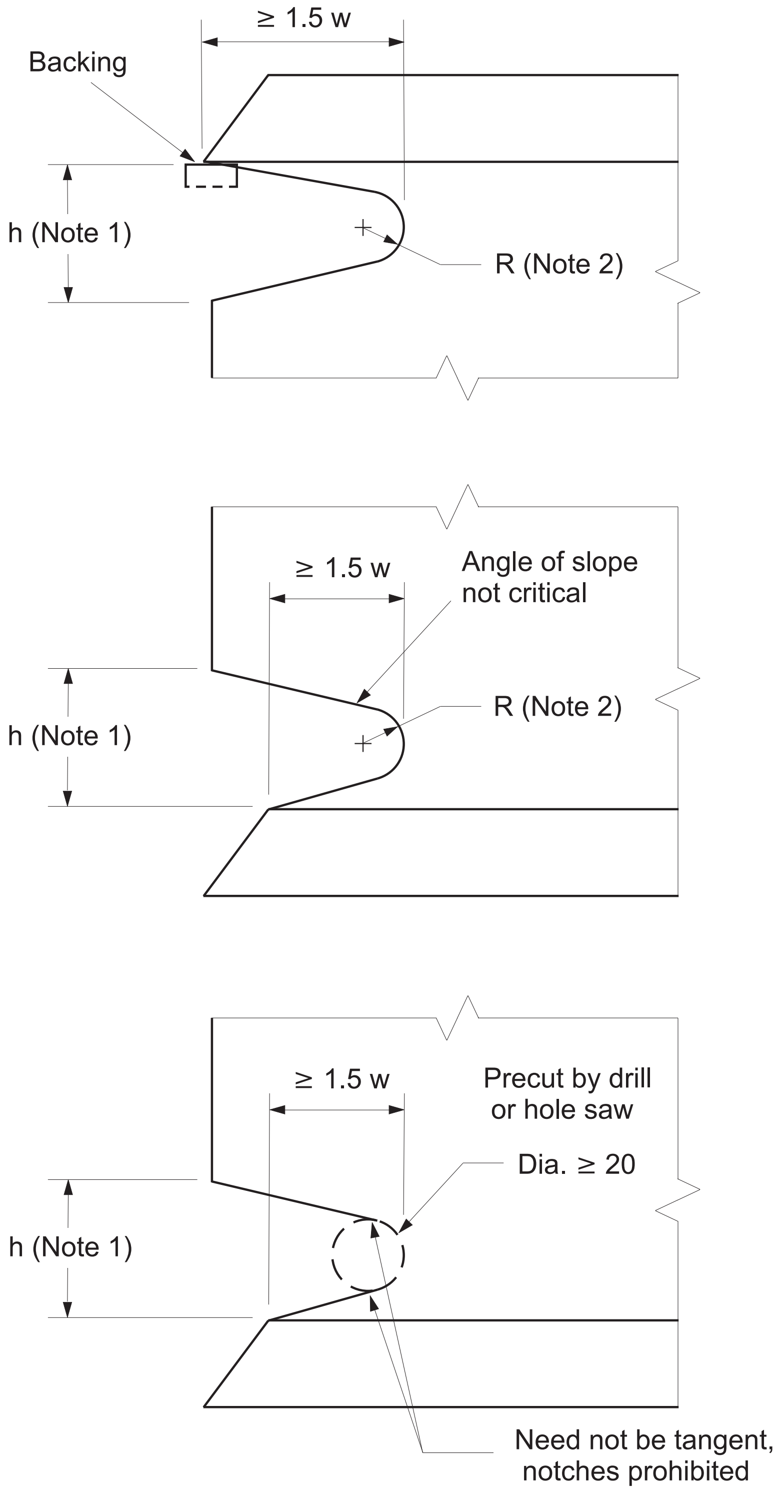
Figure 6
Reduced Beam Section (RBS) Connection

Footnotes:

1. See Section 6.1 for calculation of RBS dimensions and Section 6.2 for fabrication details.
2. CJP groove weld at top and bottom flanges. At bottom flange, remove backing bar, backgouge to sound metal, and add an 8 millimetre or larger fillet weld. At top flange, the backing bar may be left in place and a continuous 8 mm fillet weld added under the backing. FEMA recommendation for NDT of weld: MT 100% of joints, full length; UT 100% of joints, full length. The weld access holes should be proportioned in accordance with either Figure 6.1 or Figure 3.5 of FEMA 350 (FEMA 2000a).
3. Web connection: The web connection shall be proportioned in accordance with Section 6.1.
   a. Alternative 1: Field-bolted shear tab. Bolts shall be pretensioned and inspected in accordance with Cl. 23.8 and Cl. 23.9 of S16-01 respectively. FEMA recommendation for NDT of weld: UT 10% of CJP joints, full length; MT 10% of fillet-welded joints, 150 mm spot at random.

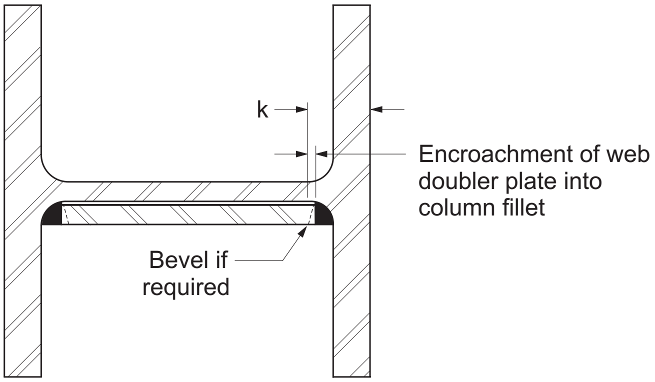
   b. Alternative 2: CJP welded web. Shear tab length is equal to the distance between the weld access holes plus 6 mm (The access holes height, h, given in Figure 6.1 should be the clear height, excluding this 6 mm shear tab extension). Shear tab thickness is as required for erection and the tab serves as backing for CJP weld (10 mm min. thickness). Shear tab may be cut square, or tapered as shown. Weld of shear tab to column flange is minimum 5 mm fillet on the side of the beam web, and a fillet sized for erection loads (8 mm minimum) on the side away from the beam web. No weld tabs are required at the ends of the CJP weld and no welding of the shear tab to the beam web is required. Erection bolts: number, type and size shall be selected for erection loads. FEMA recommendation for NDT of weld: MT 100% of CJP joint; UT 100% of CJP joints (when acceptance rate is high reduce to 25% of joints); MT 25% of fillets. Test full length of welds, except for welds longer than 600 mm partial length testing applies.

4. For continuity plates and web doubler plates see Section 6.1 and Figures 7.2 and 7.3.

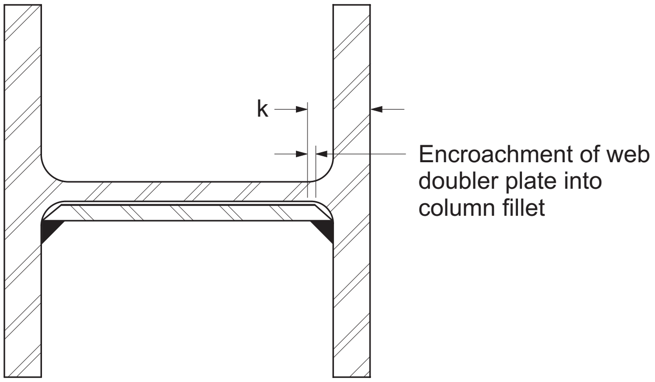


Notes:
1. Hole height h ≥ w and ≥ 20mm. See note 3 in Figure 6.
2. Radius shall provide smooth notch-free transition;
   R ≥ 10 mm (typical 12 mm)

Figure 6.1
Weld Access Hole Dimensions
(as provided in W59-03)



(a) CJP Groove-Welded Detail



(b) Fillet-Welded Detail

Note: Avoid welding in the k-area

Figure 7.2
Column Web Doubler Plate Details

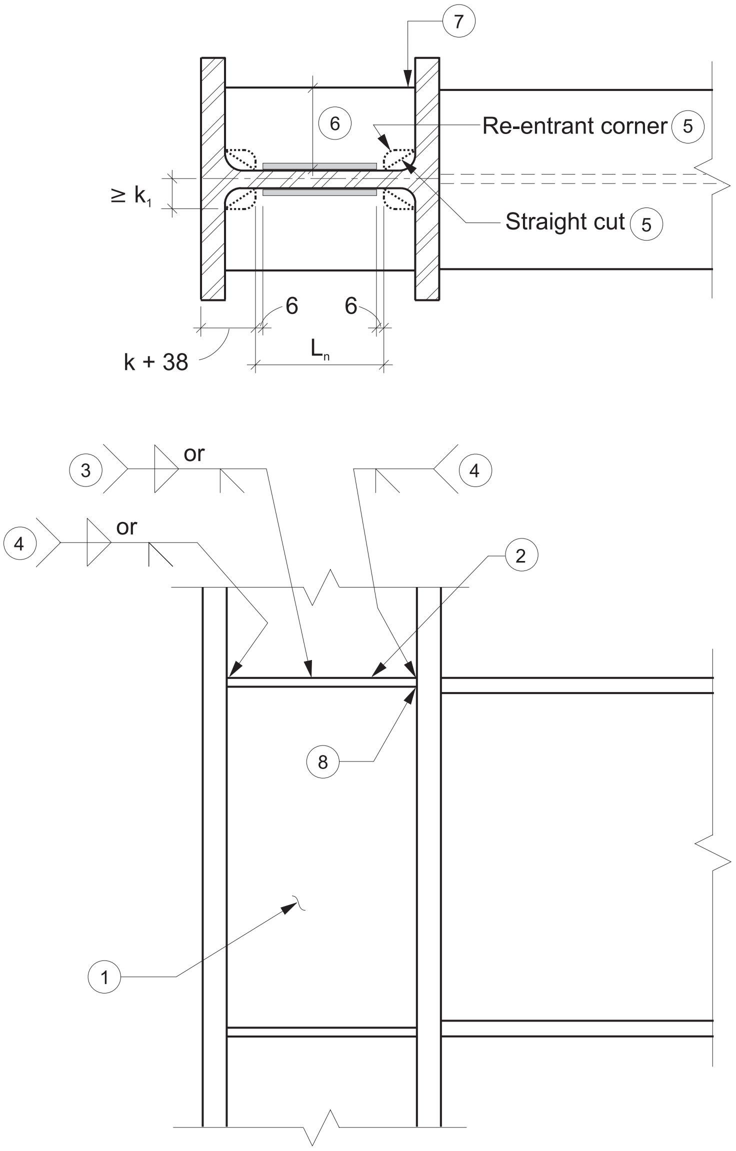
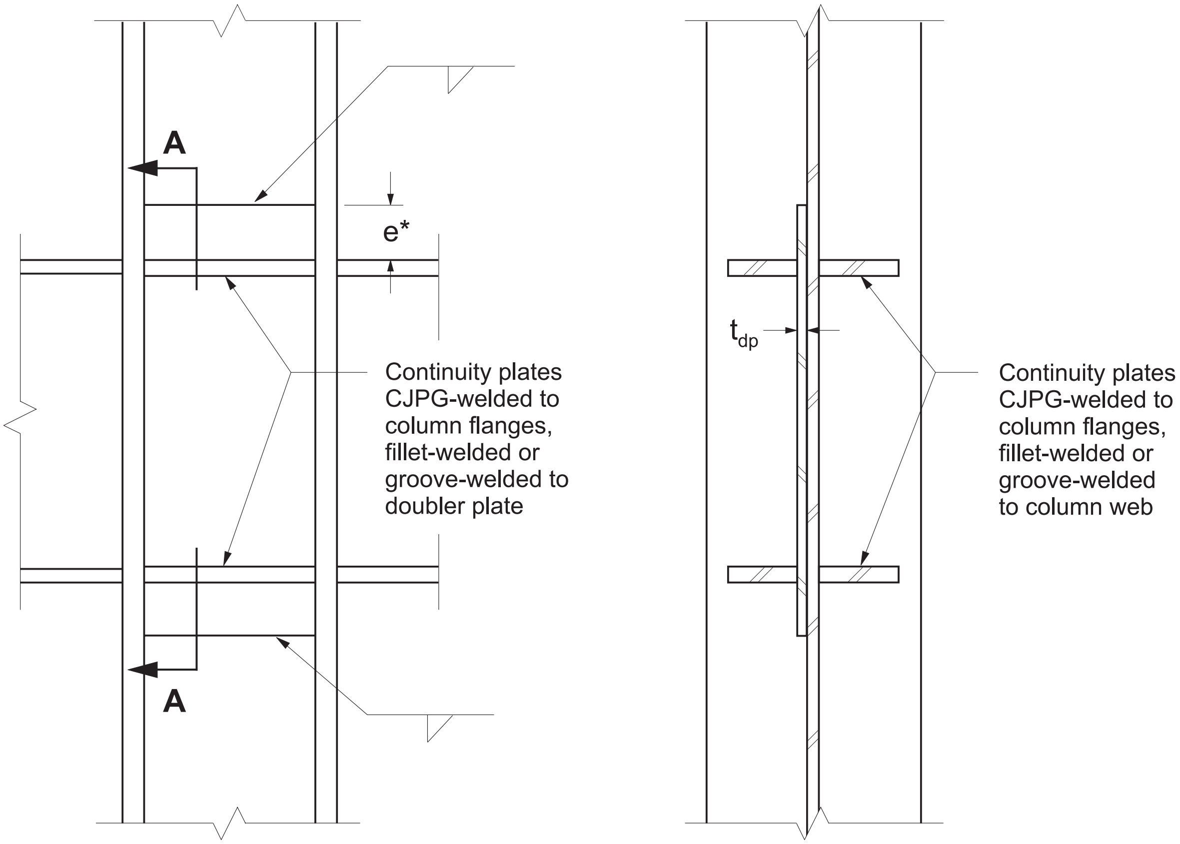


Figure 7.3(a)
Continuity Plates (Shown as One-sided Connection)

Footnotes:

1. Web doubler plate where required in accordance with Sections 3.4, 4 and 5.
2. Continuity plate where required in accordance with Sections 4, 5 and 6.
3. Required total weld strength = $0.6t_p(L_n)F_{yp}$, where F_{yp} is F_y of the continuity plate. *FEMA recommendation for NDT of weld: MT 10% of joints, full length; except for CJP joints, UT 10% of joints, full length.*
4. CJPG, typical. For one-sided connections (as shown in Fig. 7.3(a)), the continuity plates may be fillet welded to the column flange at the free side. *FEMA recommendation for NDT of weld: MT 100% of CJP joints, full length; UT 100% of CJP joints, full length (when acceptance rate is high reduce to 25% of joints). MT 25% of fillets, full length.*
5. Except for straight cut bevel clips, provide a re-entrant corner in accordance with W59-03.
6. Minimum width to match beam flange. Plates flush with column flange tips preferred.
7. Remove extension bar beyond 6 mm from edge of continuity plate. Grind end of weld smooth.
8. Steel backing bar, if used, should be connected to column flange using a continuous fillet weld under backing. Minimum single-pass fillet size should conform to W59-03.



Continuity plates CJPG-welded to column flanges, fillet-welded or groove-welded to doubler plate

Continuity plates CJPG-welded to column flanges, fillet-welded or groove-welded to column web

*e $\geq 2.5k$ for RBS connections,
$\geq 3k + t_{dp}$ for end-plate connections

Section A-A

Figure 7.3(b)
Column with Continuity Plates and Extended (single) Web Doubler Plate

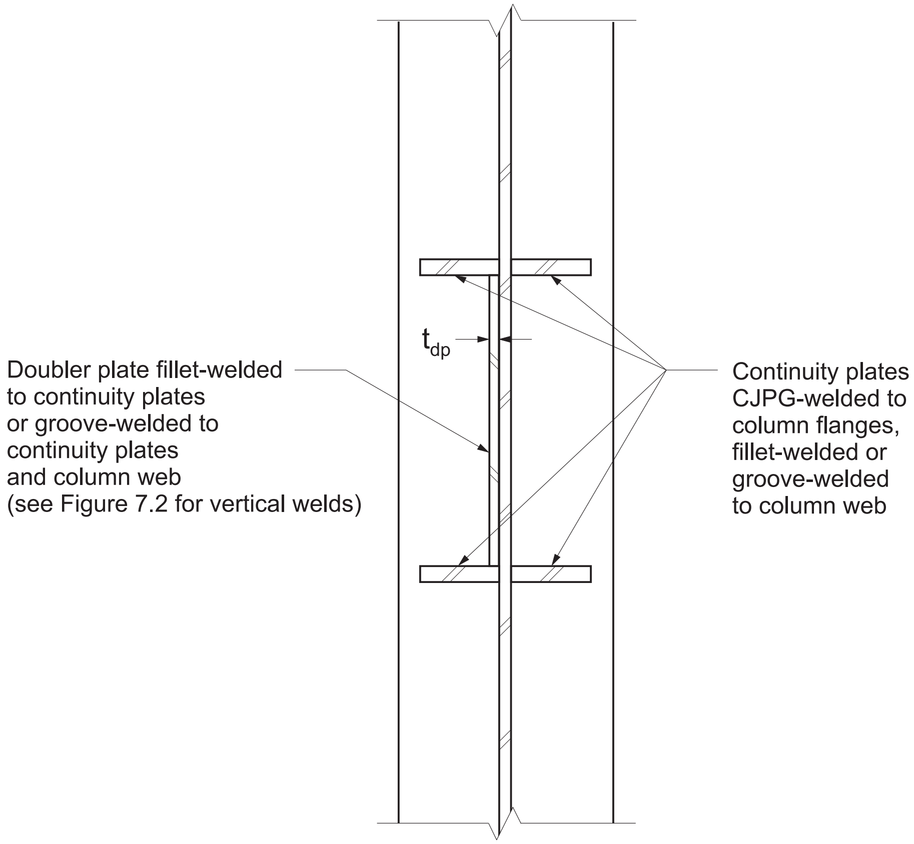
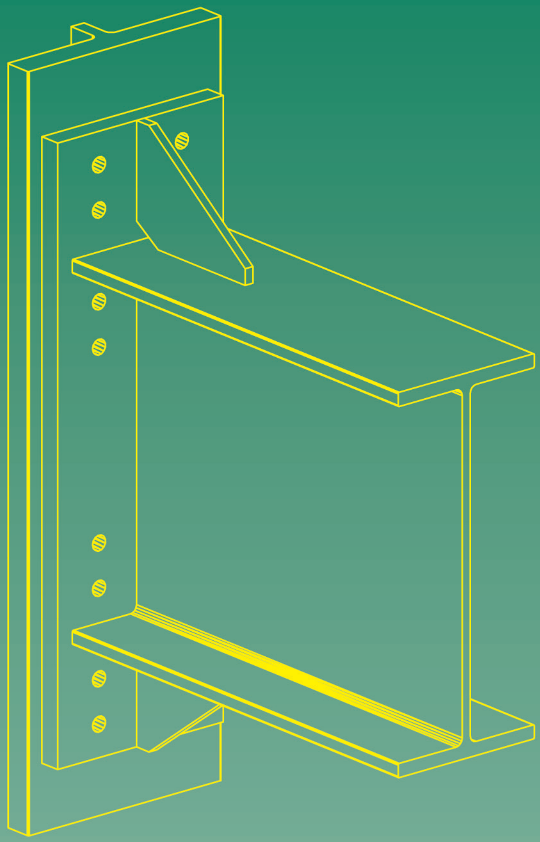


Figure 7.3(c)
Column with Continuity Plates and Flush (single) Web Doubler Plate



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