OPTIMIZATION OF SPECIAL STEEL MOMENT FRAME CONNECTION DESIGN

by

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A REPORT

submitted in partial fulfillment of the requirements for the degree

MASTER OF SCIENCE

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KANSAS STATE UNIVERSITY
Manhattan, Kansas

2015

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Abstract

Special steel moment frames are one of the most common systems used to resist high seismic forces. Well-proportioned moment resisting connections are essential. Special steel moment frame connections must be capable of transferring moment and shear forces that are developed in the beams to the column. These connections must be designed as a highly ductile element in order to dissipate extensive energy thus undergo inelastic deformations. Doubler plates and continuity plates have been recommended by several design codes and standards in order to strengthen the column web and prevent the inelastic deformation of the panel zone due to high shear stress concentrations. However, doubler plates and continuity plates are very expensive due to the large amount of detailing and welding requirements. Furthermore, the extensive welding may affect the properties of the steel in which it may cause shrinkage, lower potential notch toughness and cracking. In any of these cases, there is high potential of losing the desirable inelastic performance required for these SMF. This report investigates the design of the special steel moment frame connections thus eliminating the use of doubler and continuity plates in these connections. Tables are provided that show all steel W-Shape beam sizes with all the adequate steel W-Shape column sizes used in special steel moment frames without the use of doubler and continuity plates in frame connections.

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Definitions

Complete joint penetration. It is a type of weld that extends completely through the thickness of components joined.

Continuity plates. Column stiffeners at the top and bottom of the panel zone; also known as transverse stiffeners.

Doubler plates. Plates that act as a reinforcement for the column web to prevent panel zone shear failure.

Drift. The displacement or deflection of a member due to lateral forces applied to it.

Highly ductile member. A member expected to undergo significant plastic rotation (more than 0.02 rad) from either flexure or flexural buckling under seismic loads

Inelastic Deformation. Deformation of a member in which it cannot return to its original shape; also known as non-recoverable deformation.

Inelastic Energy. Kinetic energy which is not conserved due to the action of internal friction.

k-area. The region of the web that extends from the tangent point of the web and the flange-web fillet (AISC "k" dimension) a distance of 1 ½ in. into the web beyond the k dimension.

Notch toughness. The ability of a material to absorb energy in the presence of a flaw.

P-Delta. The second order effect on shears and moments of frame members induced by axial loads on a laterally displaced building frame.

Plastic hinge. Yielded zone that forms in a structural member when the plastic moment is attained. The member is assumed to rotate further as if hinged, except that such rotation is restrained by the plastic moment.

Chapter 1 - Introduction

Steel moment frames were developed in the 19^{th} century in order to be used in high-rise structures. Angles and T-sections were commonly used to connect the beam flanges to the column creating moment frame connections. In the 20^{th} century, new welding techniques and high-strength bolts were introduced and used for creating moment frame connections. One of the most famous connections used in moment frames is Welded Flange connection with a shear tab for vertical loads and complete joint penetration welds for the beam-to-column connection. However, this standard connection didn't perform as expected in terms of ductility and inelastic deformation during the 1994 Northridge Earthquake. Failure in the moment frame connections occurred due to various causes. Moment frame connections were investigated and analyzed for the different types of failures and all causes were documented. Several new design requirements and codes were proposed based on investigations and testing.

Structural steel special moment frames (SMF) are mainly used as seismic force-resisting systems in buildings designed to resist severe earthquakes in which an extensive inelastic energy is dissipated. SMF are expected to undergo substantial yielding and inelastic deformation without significant strength loss. The American Society of Civil Engineers (ASCE) has defined SMF as one of the few systems that has no height limit under any seismic design category. According to National Earthquake Hazards Reduction Program (NEHRP), columns, beams and beam-column connections are designed and detailed to resist flexural, axial and shearing actions that results as a building sways through multiple inelastic displacement cycles during strong earthquake ground shaking.

A series of standards and codes provides the design requirements for steel special moment frames. ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures*

(ASCE, 2010) contains the seismic design criteria and requirements for steel special moment frames. ANSI/AISC 341-10, *Seismic Provisions for Structural Steel Buildings* (AISC, 2010) which is written to provide the design and construction of structural steel and composite structural steel/reinforced concrete building systems for high-seismic applications. ANSI/AISC 358-10, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (AISC, 2010) sets all the standards for the connection design in steel special moment frames, thus allowing their use without the need of special testing. ANSI/AISC 360-10, *Specification for Structural Steel Buildings* (AISC, 2010) is the main AISC specification which covers the design guide and specifications for all steel buildings. In addition to these standards, American Welding Society (AWS) has provided the standards and requirements for welding and fabrication of steel members and plates through AWS D1.1 *Structural Welding Code* and AWS D1.8 *Structural Welding Code Seismic* (AWS, 2005).

There are several types of rigid connections that have been prequalified by Federal Emergency Management Agency (FEMA) for seismic applications. FEMA 350, *Recommended Seismic Design Criteria for New Steel Moment-Frame Building* (FEMA, 2000) provides design criteria, size and other limitations for their use. The Canadian Institute of Steel Construction (CISC) states that the prequalified connections apply to frames consisting of wide-flange beams and columns subjected to strong axis bending. SMF connections must be capable of transferring the moment and shear forces that are developed in the beam to the column. Shear forces that occur at the panel zone of beam-to-column connection due to the transfer of moments from beams to columns, have resulted in serious problems. As the panel zone experiences large shear forces due to the large number of cycles of large inelastic distortions taking place in high seismic events, the panel zone starts to yield from the center towards the corners where it experiences failure and deformation. In order to reduce the chance of panel zone experiencing inelastic deformations, doubler plates and continuity plates are introduced.

Doubler plates are needed to strengthen the column web and provide extra strength to the panel zone to resist shear. They are fabricated and welded adjacent to the column web plate. While continuity plates are added between columns flanges to help transfer beam flange forces through the entire connection. Both plate types require proper detailing and welding to ensure transfer of forces through this highly stressed region and high ductile performance of the SMF connection. NEHRP seismic design discusses that adding doubler and continuity plates is very expensive due to the significant shop fabrication time that is needed to prepare the plates and welding them into the column (Hamburger et al., 2009). Understanding this information and working on designing special steel moment frames connections, the idea of eliminating both doubler and continuity plates without affecting the strength and performance of the beam-to-column connection, will be discussed.

The purpose of this report is to investigate the connection design of the special steel moment frames wherein the use of doubler and continuity plates in these connections is eliminated. Table 1 shows special steel moment frames comprised of W36x beams and various steel W-Shape columns that are considered adequate without the use of doubler and continuity plates in frame connections.

Table 1 – W36 Beams

Beam	Column			Include
W36x232	W36x652			-
	W14x730			-
W36x231	W14x730			-
W36x210	W40x593			-
	W36x652			-
	W14x665	up to	W14x730	-
W36x194	W40x593			-
	W36x652			-
	W14x665	up to	W14x730	-
W36x182	W40x593			-
	W36x652			-
	W14x605	up to	W14x730	-
W36x170	W40x503	up to	W40x593	-
	W36x529	up to	W36x652	-
	W27x539			-
	W14x605	up to	W14x730	-
W36x160	W40x503	up to	W40x593	-
	W36x529	up to	W36x652	-
	W27x539			-
	W14x550	up to	W14x730	-
W36x150	W40x431	up to	W40x593	W40x392
	W36x487	up to	W36x652	-
	W27x539			-
	W14x550	up to	W14x730	-

Chapter 2 - Special Steel Moment Frame Connection Design & Requirements

When designing SMF, three goals must be considered in order to provide ductile inelastic response. These primary goals are achieving strong column / weak beam frame, avoiding P-delta instability, and detailing connections and plastic hinges to dissipate energy. Sizing of beams and columns are typically controlled by drift. The concept of strong column / weak beam is achieved when the sum of column flexural strengths at each joint exceeds the sum of beam flexural strengths. This is essential to avoid any deformation of plastic hinges located at top and bottom of columns thus avoiding developments of P-delta instability of the frame. According to AISC *Seismic Provisions*, columns can be assumed to remain elastic when the column-beam moment ratio is 2 or greater.

Beam-to-column moment connections for SMF systems develop inelasticity in the beams and in the column panel zone as shown in figure 1. The efficiency of the moment connection is dependent on its ability to transfer the moment and shear forces that are developed in the beam to the column. In cases where the moment and shear forces are very high, moment connections may experience various failure modes such as fracture in or around welds, fracture in highly strained base material, fractures at weld access holes and net section fracture at bolt holes. Any of these failure modes may lead to panel zone deformation and/or failure.

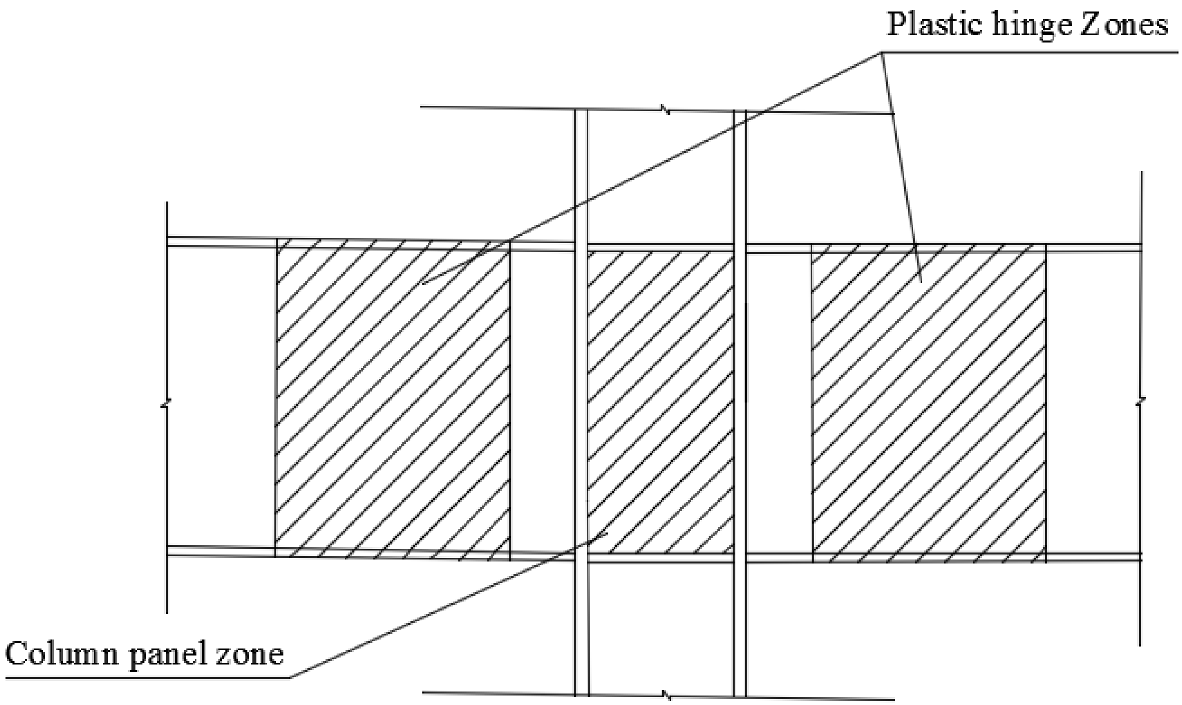

Figure 1 – Areas where inelastic deformation may be expected

Column panel zone deformation contributes a significant amount of ductility to the frame, thus the inelastic deformation of frames is controlled by the high ductility potential of the panel zone. Therefore the seismic design requirements focuses on the beam and column panel zone. The aim of the seismic design guide is to have a well-proportioned moment connection that can provide large, stable, plastic rotational capacity. This can be achieved by balancing the energy dissipation between the panel zone and plastic hinges located at the beam. AISC discusses two methods used to move plastic hinging of the beam away from the column so that the deformation doesn't occur in the column panel zone. One method is by reducing the cross-sectional properties of the beam at a defined location away from the column while the other method is by special detailing of the beam-to-column connection so that it can provide adequate strength and toughness in the connection to force inelasticity into the beam adjacent to column flange. In this report reduced beam section (RBS) connections are investigated.

Reduced Beam Section Connection

RBS connections are prequalified connections that are used in SMF systems consisting of wide flange beams and columns, subjected to strong axis bending. Moment frame connections are not limited to prequalified connections, however it is recommended by AISC and FEMA to use prequalified connections in moment frames due to the extensive analysis and testing on optimizing the design and behavior of these connections. AISC *Seismic Provisions*, Chapter 5, sets prequalification limits for RBS connections in which beams and columns are limited to certain sizes, weight, span-to-depth ratio, width-to-thickness ratio and lateral bracing. In addition to beam-to-column welding limitation and fabrication of flange cuts are outlined in details. The design procedure for the cut in the beam flange is set to the limits:

$$0.5b_{bf} \leq a \leq 0.75b_{bf}$$

$$0.65d \leq b \leq 0.85d$$

$$0.1b_{bf} \leq c \leq 0.25b_{bf}$$

$$R = \frac{4c^2 + b^2}{8c}$$

Where

b_{bf} = width of beam flange.

a = horizontal distance from face of column flange to the start of an RBS cut.

b = length of an RBS cut.

c = depth of cut at center of the reduced beam section.

d = depth of beam.

R = radius of cut.

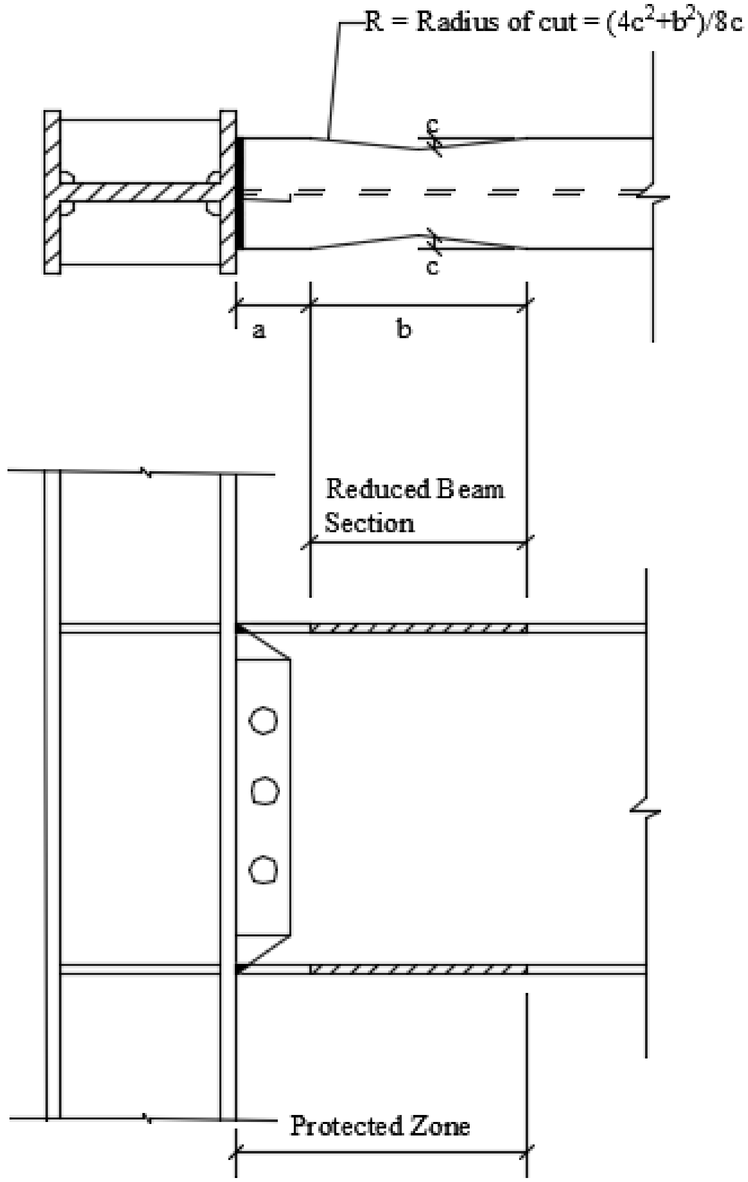

Figure 2 – Reduced beam section connection (As per AISC 358)

RBS connections for SMF must be able to develop at least 0.04 radians of interstory drift without significant strength loss, when subjected to cyclic loading, as specified in AISC 341. These connections are designed to develop plastic hinges in the beam in which the probable plastic moment capacity at the location of the plastic hinge is:

$$M_{pr} = C_{pr} R_y F_y Z_{RBS}$$

Where

$C_{pr} = \frac{F_y + F_u}{2F_y} \leq 1.2$

R_y = ratio of the expected yield stress to the specified minimum yield stress, F_y, as specified in the AISC *Seismic Provisions.*

F_y = specified minimum yield stress of the yielding element.

F_u = specified minimum tensile strength of the yielding element.

Z_{RBS} = effective plastic section modulus at the center of the reduced beam section.

It is computed with this equation:

$$Z_{RBS} = Z_x - 2ct_{bf}(d - t_{bf})$$

Where

Z_x = plastic section modulus about x-axis, for full beam cross section.

t_{bf} = thickness of beam flange.

c = depth of cut at center of the reduced beam section.

d = depth of beam.

AISC 341 define the location of the plastic hinge for beams with a RBS cutout to be assumed to occur at the center of the reduced beam section of the beam flange. The shear force at each plastic hinge is resulted due to the moment, $M_{pr,}$ at the center of the plastic hinge and calculated with the equation:

$$V_u = \frac{2M_{pr}}{L_h} + V_{gravity}$$

Where

L_h = distance between plastic hinge locations.

$V_{gravity}$ = beam shear force.

The Panel zone is required to behave in a very ductile mode of deformation. It can undergo many cycles of large inelastic distortions without deterioration in strength, while exhibiting cyclic hardening (Hamburger et al., 2009). It is recommended to design panel zones to share the energy dissipated through inelastic deformation with the plastic hinges located in the beams. However, panel zone experiencing large shear forces may start to yield at its center towards its corners leading to deformation of the panel zone. Therefore it's very important to check the magnitude of beam bending strength and panel zone shear strength. According to AISC *Seismic Provisions*, the required shear strength of panel zone is determined from the summation of the moments at the column faces as determined by projecting the expected moments at the plastic hinges to column faces, as shown in figure 3.

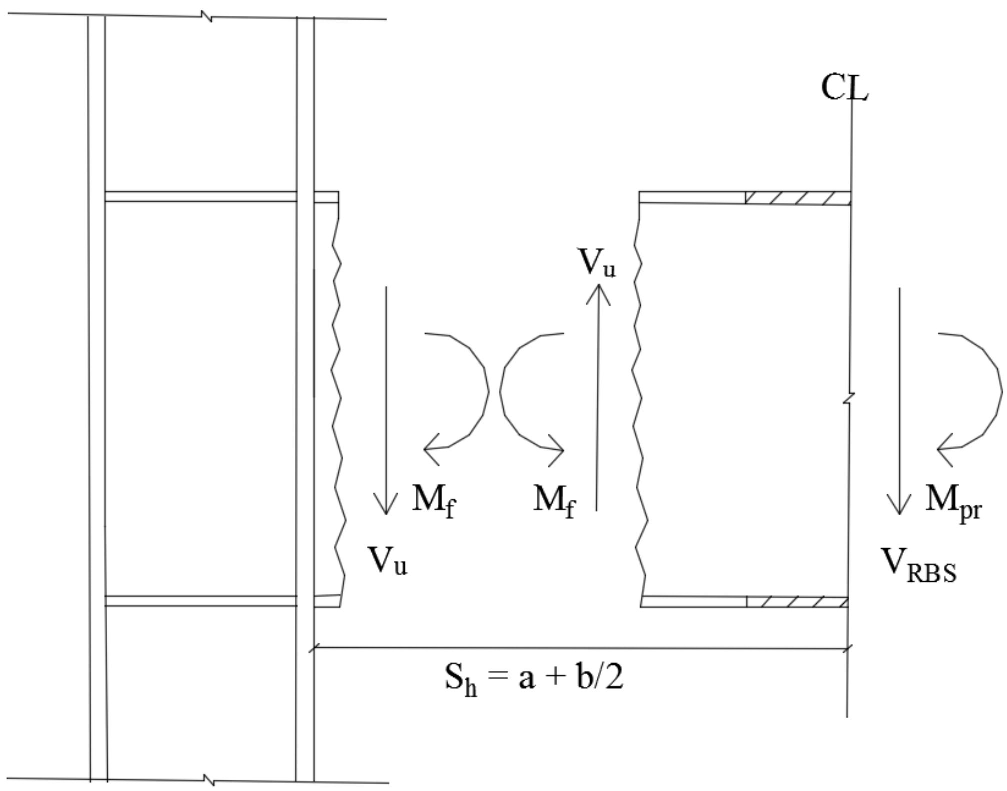

Figure 3 - Free-body diagram between center of RBS and face of column

The moment at the face of the column, M_f, is computed using this equation:

$$M_f = M_{pr} + V_{RBS}S_h$$

Where

S_h = distance from face of the column to the plastic hinge.

V_{RBS} = shear force at the center of the reduced beam section.

As mentioned before, the required column panel zone shear strength must be determined from the summation of the moments at the column faces. However, the application of the moments at the column face to determine the required shear strength of the panel zone recognizes that beam hinging will take place at a location away from the beam-to-column connection, which will result in amplified effects on the panel zone shear. The ultimate shear force acting on the panel zone, $R_{u,}$ is computed using this equation:

$$R_u = \frac{\sum M_f}{d_b - t_f}$$

Where

d_b = depth of the beam.

t_f = beam flange thickness.

AISC *Steel Specifications* chapter J states that typical applications where panel zone deformation in a beam-to-column connection is considered, and considering that the column and beam sizes are relatively large due to drift and frame stability requirements, the required axial strength at the beam-to-column connection falls under this equation:

$$P_r \leq 0.75\, P_c$$

Where

$P_r = P_{u,}$ the required axial strength.

P_c = column available axial compressive strength, $P_c = F_y A_g$

In this case, AISC 360 sets the design shear strength, $\phi_v R_n$, at the panel zone to be computed using this equation:

$$R_n = 0.6 F_y d_c t_{cw} \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_{cw}}\right)$$

Where

ϕ_v = 0.9 for load and resistance factor load

F_y = specified minimum yield stress of the column web.

d_c = depth of column.

t_{cw} =thickness of column web.

b_{cf} =width of column flange.

t_{cf} = thickness of column flange.

In order to have an adequate panel zone at the beam-to-column connection, $R_u \leq \phi_v R_n$, must be satisfied. However, most of the SMF connections, designed to undergo extensive yielding and inelastic deformation during high seismic events, fail to meet this requirement thus doubler plates are required.

Chapter 3 - Doubler & Continuity Plates

Doubler and continuity plates are very essential elements for a better performance of the beam-to-column connection in a special steel moment frame. Both plates are typically welded to column web and flanges to help transferring forces from beam to column through panel zone thus strengthening panel zone and preventing it from deformation. Figure 4 shows columns with both transverse stiffeners and web doubler plates, in which they are fillet welded to the column flanges. According to the AISC *Design Guide*, fillet-welded details is preferable over groove-welded details in the majority of cases, however, preference should be given to the use of details that require the least amount of weld metal with due consideration of the material preparation requirements. It is important to provide proper detailing of the welds between doubler plates and column web, column flanges, and/or continuity plates to ensure that forces transfers through this highly stressed region can be achieved (NEHRP, 2009).

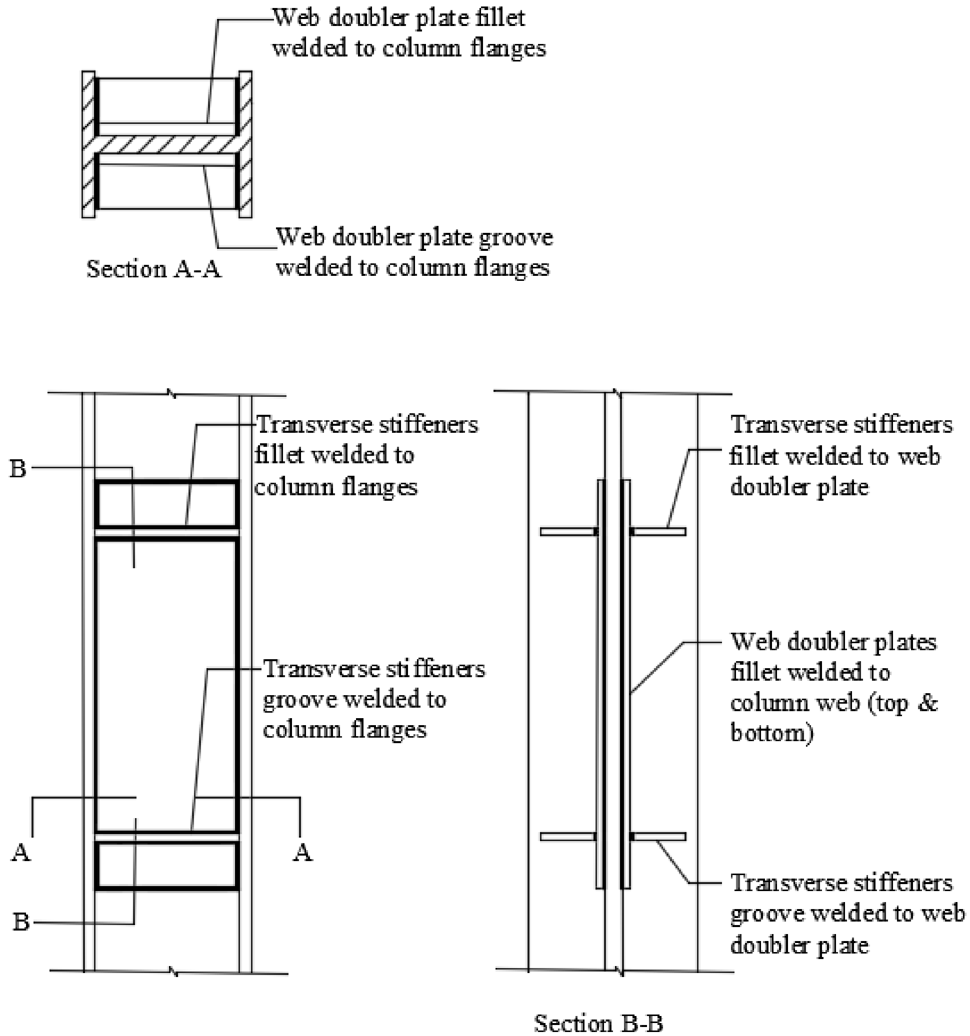

Figure 4 - Column with transverse stiffeners & web doubler plate

Doubler Plates

Equating the required shear strength with design shear strength will determine the need for panel zone doubler plates, due to the high shear forces occurring in the joint panel zones of special steel moment frames. Adding doubler plates is needed to strengthen the column web, thus meeting the panel zone shear strength requirements. When the column web thickness is inadequate to resist tensile or compressive flange forces, web doubler plates and/or transverse stiffeners are required, extending at least one-half the depth of the column web. When they are used, shear is dispersed between them in the column panel zone, hence, axial forces in the column flanges balance this shear. Therefore, they can be used to resist local web yielding, web crippling, and/or compression buckling of the web per LRFD Specification Section K1 (AISC Design Guide, 2003). The required strength of the web doubler plate is:

$$V_{u_{dp}} = V_u - \phi R_{v_{cw}}$$

Where

V_u = factored panel-zone shear force.

$ØR_{v\ cw}$ = column web design shear strength

When designing web doubler plate, there are various ways to determine the width, depth and thickness of the web doubler plate. In case transverse stiffeners are used, the web doubler plate shall be extended to the transverse stiffeners or even more preferable to extend it past the transverse stiffeners so that the top and bottom edges of the web doubler plate can be square-cut and the corners of the transverse stiffeners do not need to be clipped so that the column flange-to-web fillets are cleared. In case that transverse stiffeners are not used, the web doubler plate shall extend past the beam flange to clear the zone where the column web subject to crippling and buckling. AISC *Design Guide* set a minimum distance of 2.5 times the column k-distance for directly welded flange or flange plated moment connection and 3 times the column k-distance plus the end-plate thickness for an extended end-plate moment connection. Figure 6 shows the various ways of welding either one or two web doubler plates to column web.

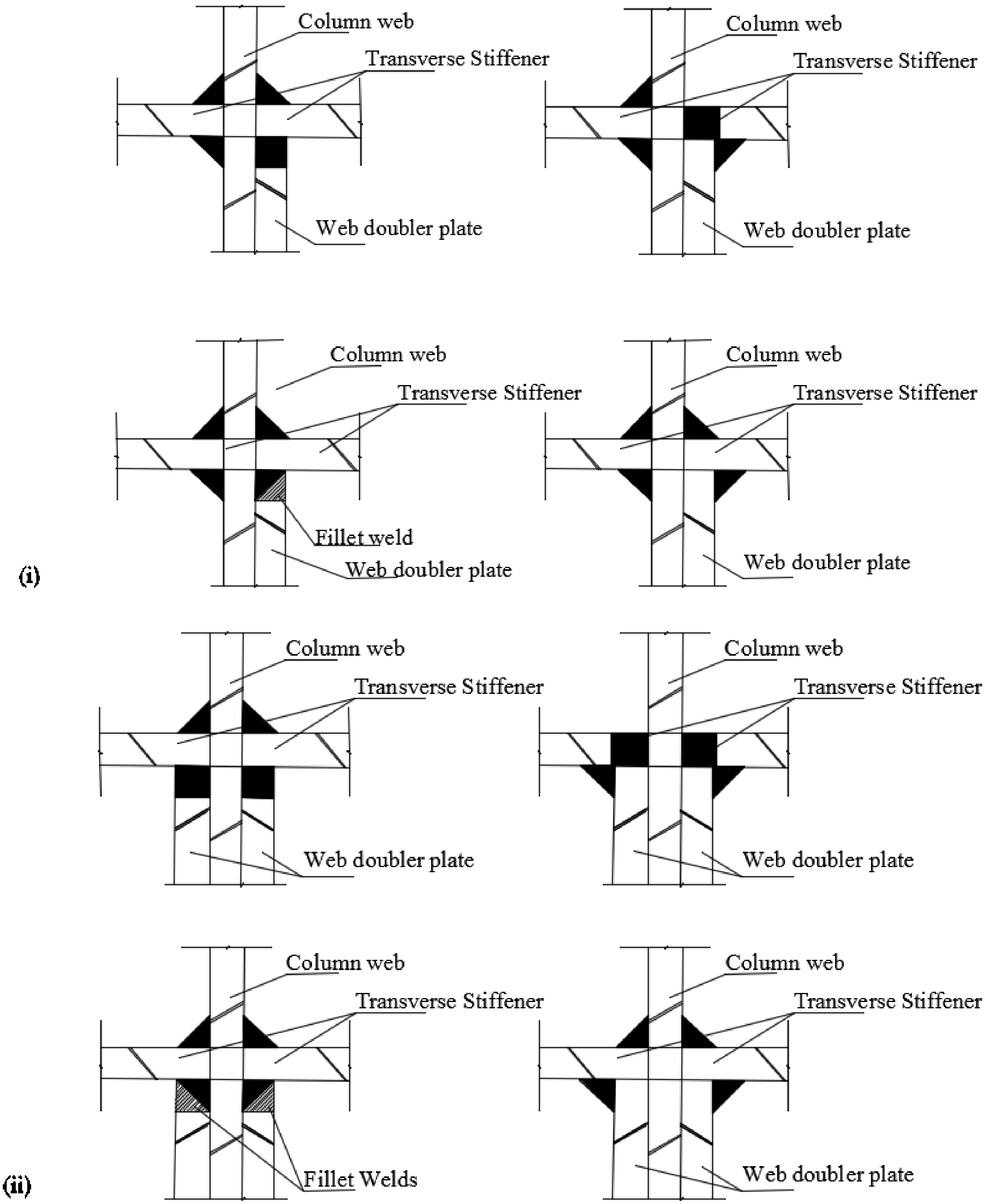

Figure 5 - Welded joint details at top & bottom edges with (i) one doubler plate (ii) two doubler plates

The web doubler plate thickness is selected to provide an additional thickness to the column web so that it meets the required column web thickness needed to resist panel zone web shear. This required web doubler plate thickness is:

$$t_p \geq \frac{V_{u_{dp}}}{0.9(0.6F_y d_c)}$$

Where

$V_{u\,dp}$ = shear carried by the web doubler plate.

F_y = web doubler plate specified minimum yield strength.

d_c = column depth.

It is important to check for shear buckling of the web doubler plate in high seismic application. The minimum thickness of both the column web and web doubler plate per AISC *Seismic Provisions* is:

$$\mathrm{t}_{\mathrm{p}_{min}} = \frac{d_m - t_s + d_c - 2t_f}{90} \geq \frac{h\sqrt{F_y}}{418}$$

Where

d_m = moment arm between concentrated flange forces.

t_s = transverse stiffener thickness

d_c = column depth

t_f = column flange thickness

h = d_c – 2k

k = distance from outside face of column flange to the web toe of flange-to-web fillet.

AISC *Design Guide* states that when using SMF, web doubler plates shall be welded along their column-flange edges to develop the shear strength of the full web doubler plate thickness. It is recognized that welding in the flange-to-web fillet region of wide flange columns carries the potential for shrinkage distortions and subsequent cracking due to restraint and low notch toughness (AISC, 2010)

Continuity Plates

Continuity plates, also referred to as panel zone horizontal stiffeners and column transverse stiffeners, are typically added between column flanges to help transfer forces from beam flanges through the entire connection to the column. When the column flange thickness is inadequate to resist tensile or compressive forces, continuity plates shall be used in which they are welded to the inside face of the column flanges. The continuity plates stiffen the column web to prevent local crippling under the concentrated beam flange forces and minimize the stress concentrations that can occur in the joint between the beam flange and the column due to non-uniform stiffness of the column flange (AISC, 2010). As a result, continuity plates can be used to resist local flange bending, local web yielding, web crippling and compression buckling of the web. Continuity plates are required if the column flange thickness is less than the greater of

$$t_{cf} \leq 0.4\sqrt{1.8 b_{bf} t_{bf}\left(\frac{R_{yb}F_{yb}}{R_{yb}F_{yc}}\right)}$$

And

$$t_{cf} \leq \frac{b_{bf}}{6}$$

Where

b_{bf} = beam flange width.

t_{bf} = beam flange thickness.

R_{yb} = ratio of the expected yield stress of the column flange.

F_{yb} = specified minimum yield stress of the beam flange.

F_{yc} = specified minimum yield stress of the column flange.

Like the web doubler plates, proper detailing of continuity plates is essential to have the desired ductile performance of SMF connections. AISC *Seismic Provisions* sets minimum dimensions for the continuity plates. The minimum width of each continuity plate is

$$b_s = \frac{b_{bf}}{3} - \frac{t_{cw}}{2}$$

Where

b_s = width of continuity plate.

b_{bf} = width of beam flange.

t_{cw} = thickness of column web.

The minimum thickness of each continuity plate is

$$t_s = \frac{t_{bf}}{2} \geq \frac{b_s\sqrt{F_{y_{st}}}}{95}$$

Where

t_s = thickness of continuity plate.

t_{bf} = thickness of beam flange.

b_s = width of continuity plate.

$F_{y\,st}$ = specified minimum yield strength of continuity plate.

Continuity plates are typically used in pairs to avoid creating eccentricity of forces, and additional moment on the member. They are usually placed top and bottom for resisting tension and compression forces. They can be extend to full or partial depth of the column web, depending on the amount of concentrated forces from beam flanges acting on the column web. AISC *Design Guide* states that the length of a full-depth continuity plate shall be the distance between the column flanges with consideration of column cross-sectional tolerances and welding used. The length of a partial-depth continuity plate should be selected to minimize the thickness of the continuity plate and the size of the fillet weld used. Note that continuity plates should be clipped to clear the rolling fillets of the column section. The clip should be details to facilitate suitable weld terminations for both the flange weld and the web weld (CISC, 2008). Continuity plates are welded to column flanges using a Complete Joint Penetration (CJP) groove weld. They can be also welded to the column web or the web doubler plate.

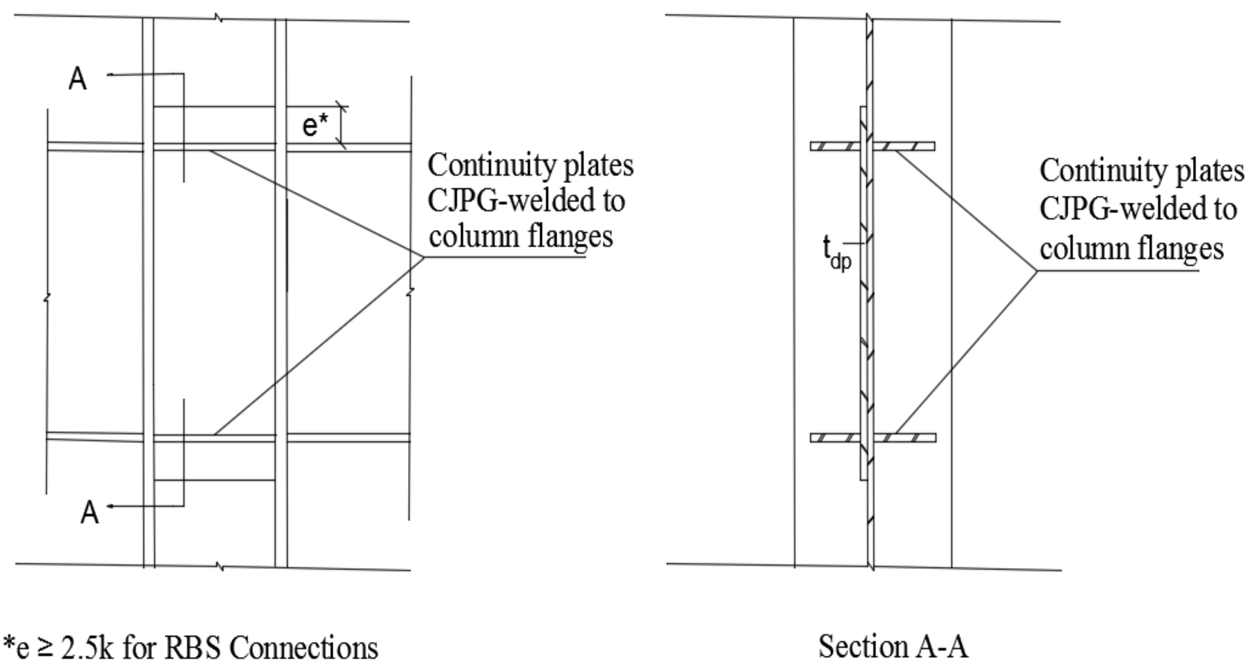

Figure 6 - Column with continuity plates & single doubler plate
(See Figure 4 for welding details)

Chapter 4 - Eliminating Doubler & Continuity Plates

Doubler plates and continuity plates are extremely labor-intensive detail materials. The amount of detailing and welding required for these plates make them very expensive and time consuming. The various weld types and sizes, in addition to, the thickness and overall dimensions of the reinforcing plates that involve measuring, cutting, profiling and welding, require extensive amount of work and time. Multiplying this effort by the number of plates needed for every moment frame in the structure may lead to a significant increase in the cost of the structure. Another factor considered is the effect of welding on the properties of the steel. Web doubler plates require significant welding into the column flange-to-web fillet region (k-area), which is an area of potentially lower notch toughness (AISC, 2010). The welding involved, can consequently cause shrinkage and affect the properties of the steel resulting in high potential of cracking. Heat produced by welding has high potential of affecting the crystal structure of steel resulting in becoming more of brittle material thus losing the main advantage of using steel which is ductility. Using thick plates require large weld sizes, the associated heat input and shrinkage may be undesirable with respect to connection performance as well as production (CISC, 2008). Therefore, eliminating doubler plates and continuity plates can save significant cost and time, as well as, maintain properties and performance of material used. In order to eliminate reinforcing plates, larger columns should be used in special steel moment frames. Larger columns have thicker webs and flanges that can resist concentrated forces from beam flanges without causing any local yielding, buckling, crippling or panel zone deformation.

AISC *Design Guide* includes table with estimated costs for different transverse stiffeners and web double plates with the required types and sizes of welds. It compares different examples and applications in order to elaborate the concept of achieving balance between increase in material cost and reduction in labor cost. However, NEHRP explains that typically when designing a SMF a rule of thumb is applied. Assuming story heights on the order of 15 ft and beam spans of approximately 30 ft, if the designer can increase the mass per foot of the column by less than 100 lb/ft and avoid the need for doubler plates, the cost of the frame will be reduced (NEHRP, 2009).

In this report, the main concern is to eliminate both doubler plates and continuity plates in SMF connections without affecting the inelastic performance of the overall SMF in a high seismic event. Assumptions and computations were involved in order to design SMF using unreinforced columns. Engineers should evaluate whether to use unreinforced columns or reinforcing plates, as there are various factors that may affect with the cost such as inflation rate, fabricators prices, location cost modification, labor cost per ton, etc.

Assumptions & Computations

Using AISC *Steel Construction Manual*, Table 1-1, W-Shapes beams and columns are selected. While following the beam and column limitations mentioned in AISC *Seismic Provisions* for special steel moment frames, the beam shapes are selected with the suitable unreinforced column shapes. It's important to note that Table 1-1 in AISC *Steel Construction Manual* contains all different W-Shape members with their properties. Some W-Shapes have superscripts that identify certain issues or requirements. For example, W24x103^c, the superscript "c" means that the W-Shape is slender for compression, "h" means that flange thickness is greater than 2 inches where special requirements may apply per AISC *Specification* Section A3.1c and "v" means that shape does not meet the slenderness ratio (h/t_w) limit for shear in AISC *Specification* Section G2.1(a). Therefore any W-Shapes with superscript "v" are not used for both beams and columns, and W-Shapes with superscript "c" are not used for columns.

Since RBS connections are used in this report for SMF connection, assumptions are made concerning the dimensions of the cut of the beam flange to limit the variables. Following the limits for the cut dimensions, the dimensions of the beam flange reduction are selected to be

$$a = 0.7b_f$$

$$b = 0.8d$$

$$c = 0.2b_f$$

Where

a = horizontal distance from face of column flange to the start of an RBS cut.

b = length of an RBS cut.

c = depth of cut at center of the reduced beam section.

b_f = width of beam flange.

d = depth of beam.

The frame span, L, is assumed to be 40 feet as typically used in industry. The radius of cut, R, plastic section modulus at center of the reduced beam section, Z_{RBS}, distance from face of the column to plastic hinge, S_h and distance between plastic hinge locations, $L_{h,}$ are calculated using the equations mention in RBS connection section or following the AISC *Seismic Provisions*, Chapter 5. Using these calculated values, we are able to compute the moments at the reduced beam section, $M_{pr,}$ the maximum moment at the column face, $M_{f,}$ and expected shear at plastic hinge, $V_{RBS.}$ It is important to ensure that maximum moment at the face of the column does not exceed the plastic moment of the beam so that the beam has adequate flexural strength. In addition to confirming that the beam has adequate shear strength by checking that the expected shear at the plastic hinge does not exceed the design shear strength of the beam. It is assumed that the ratio of required axial strength, $P_{r,}$ and nominal compressive strength, $P_{c,}$ is equal to 0.2 to meet AISC *Specification,* Chapter H, designing members for combined forces and torsion. When designing a SMF connection, it is necessary to meet the concept strong-column / weak-beam. In order to ensure this, the sum of the projections of the nominal flexural strengths of the column above and below the joint to the beam centerline with a reduction for the axial force in the column, $\sum M_{pc,}$ must be greater than the sum of the projections of the expected flexural strengths of the beams at the plastic hinge locations to the column centerline, $\sum M_{pb.}$ M_{pc} and M_{pb} are computed and specified per AISC *Seismic Provisions* Chapter E3.4a.

While using the properties of the selected W-shapes in Table 1-1 and the values of the maximum moments at the face of the columns, the ultimate shear, R_u, at column panel zone is computed. The aim is to have a design nominal strength, $\phi_v R_{n,}$ at the web panel zone larger than the ultimate shear in order to avoid having doubler plates. In addition to satisfying equations E3-8 and E3-9 per AISC *Seismic Provisions,* to avoid using continuity plates. In the following pages, an example has been provided to guide the reader through the process of a typical SMF connection design without the use of doubler plates and continuity plates.

Design Example

The following example covers the process of designing a reduced beam section connection for a special steel moment frame using an unreinforced column:

1. Select beam and column sizes.

Beam: W21x50

A =14.7 in^2 d = 20.8 in t_w = 0.38 in b_f = 6.53 in t_f = 0.535 in

I_x = 984 in^4 r_x = 8.18 in Z_x = 110 in^3

Column: W24x207

A =60.7 in^2 d = 25.7 in t_w = 0.87 in b_f = 13 in t_f = 1.46 in

I_x = 6820 in^4 r_x 10.6 in Z_x = 606 in^3

Material: A992

F_y = 50 ksi F_u = 65 ksi

2. Design reduced beam section.

RBS Dimensions:

$a = 0.7b_f$ = 4.571 in

$b = 0.8d$ = 16.64 in

$c = 0.2b_f$ = 1.306 in

$R = \frac{4c^2+b^2}{8c}$ = 27.155 in

Span:

$S_h = a + 0.5b$ = 12.9 in

$L = 40\ ft\ (assumed)$

$L_h = L - 2S_h$ = 35.71 ft

$Z_{RBS} = Z_x - 2ct_{bf}(d - t_{bf})$ = 81.68 in^3

3. Compute Moment at reduced beam section.

$C_{pr} = \frac{F_y+F_u}{2F_y} \leq 1.2$ = 1.15

$M_{pr} = C_{pr} R_y F_y Z_{RBS}$ = 430.53 kips-ft

4. Compute expected shear at plastic hinge and check beam shear strength.

$V_{RBS} = \frac{2M_{pr}}{L_h} + \frac{wL_h}{2}$ = 41.97 kips

$V'_{RBS} = \frac{2M_{pr}}{L_h} - \frac{wL_h}{2}$ = 6.26 kips

$V_{req} = 0.5wL_h = 17.85\ kips$ (Assuming uniform unit load w)

$ØV_n = 237\ kips$ (From Table 3-2, AISC 360)

$V_u \leq ØV_n$; Beam is adequate for shear

5. Compute maximum moment at column face and check beam flexural strength.

$M_g = 0.5wS_h^2$ = 0.577 kip-ft

$M_f = M_{pr} + \frac{V_{RBS}S_h}{12} + M_g$ = 476.19 kip-ft

$M'_f = -M_{pr} - \frac{V'_{RBS}S_h}{12} + M_g$= -436.67 kip-ft

$Ø_d M_{pe} = R_y F_y Z_x$ = 504.167 kip-ft

$M_f \leq Ø_d M_{pe}$; Beam is adequate for flexure

6. Compute column-beam moment ratio and meet the strong-column/weak-beam concept.

$\sum M_{pc} = \sum Z_c(F_{yc} - \frac{P_{uc}}{A_g})$ = 4040 kip-ft

$M_{uv} = (V_{RBS} + V'_{RBS})(a + 0.5b + 0.5d_c)$ = 103.45 kip-ft

$\sum M_{pb} = \sum(1.1R_y F_{yb} Z_{RBS} + M_{uv})$ = 515.26 kip-ft

$\frac{\sum M_{pc}}{\sum M_{pb}} \geq 1.0$ = 7.84; Ok

7. <u>Compute maximum shear at panel zone and check panel zone shear strength.</u>

Having an adequate panel zone shear strength, doubler plates are not required.

$R_u = \frac{\sum M_f}{d_b - t_f}$ = 540.56 kips

$\emptyset R_n = 0.6F_y d_c t_{cw}(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_{cw}})$ = 711.6 kips

$R_u \leq \emptyset R_n$; Doubler Plate is not required.

8. Check that the selected column flange is greater than equations E3-8 and E3-9 per AISC *Seismic Provisions*, to avoid using continuity plates.

$t_{cf} \leq 0.4\sqrt{1.8b_{bf}t_{bf}\left(\frac{R_{yb}F_{yb}}{R_{yb}F_{yc}}\right)}$ = 1.0

$t_{cf} \leq \frac{b_{bf}}{6}$ = 1.09

$t_{cf} = 1.46\ in$

Continuity Plates are not required.

Following the same procedure using different beams and the columns sizes that meet all the design and strength requirements, Tables 1, and Tables A-1 through A-6 are designed, in which it shows all the different beam & column sizes that fit together without using doubler and continuity plates in SMF connection. It is very important to note that the results indicate that at a certain point columns sizes are limited to slenderness not only shear strength at the panel zone.

Therefore, smaller columns sizes and weights couldn't be connected to certain beams due to their inadequate slenderness ratio.

Chapter 5 - Discussion & Conclusion

The design of steel special moment frame connections is very crucial. Testing and experiments have been conducted by many researchers in order to understand and optimize the behavior of SMF connection. Testing and design requirements concluded that SMF connections must be capable of transferring moment and shear forces that are developed in the beams to the column. These shear forces, produced due to large cyclic loading, cause serious problems to the SMF connections when they occur at the panel zone, as the panel zone starts to yield and experience failure and deformation at its corners. In order to avoid this problem, the design codes and requirements have specified doubler plates and continuity plates to provide additional strength to the panel zone. It is required to design panel zone with high ductility and strength capacity so that it can share the dissipation of energy and provide large, stable and plastic rotational capacity. Doubler plates and continuity plates are typically welded to column web and flanges to help transferring forces from beam to column through the panel zone and prevent it from deformation. They minimize stress concentrations and resist various types of web and flange failures. However, doubler plates and continuity plates are extremely labor-intensive to detail and fabricate. They typically require large amount of detailing and welding which make them very expensive and time consuming. All the welding specifications, dimensioning, locating, and fabricating these plates consume time and money. In addition to, the extensive welding may have an effect on the properties of the steel in which it may cause shrinkage, lower potential notch toughness and cracking. In any of these cases, there will be a high risk of losing the desirable inelastic performance required for these SMF. This report covers the process of eliminating both doubler and continuity plates in SMF connections without affecting the inelastic performance of the overall SMF in a high seismic event. Considering the cost, the engineers/designers have the responsibility to evaluate whether to use unreinforced columns or reinforcing plates. Nevertheless, a rule of thumb can be applied in which assuming story height of 15 ft and beam spans 30 ft, if the designer can increase the mass per foot of the column by less than 100 lb/ft and avoid the need of doubler plates and continuity plate, the overall cost of the frame will be reduced. In this report, Table 1, and Tables A-1 through A-6 are designed to show all different beam and column sizes that can fit together without the need of doubler and continuity plates in SMF connection. Note that the beam and column sizes are picked from Table 1-1 in AISC *Steel Construction Manual*. W-Shapes with subscript “v” indicating failure due to shear are not considered for beams and columns, while W-shapes with subscript “c” indicating failure due to compression are not used for columns. It is important to mention that the results indicate that at a certain point columns sizes are limited to slenderness and not only to shear strength at panel zone, meaning that smaller column sizes and weights couldn’t be connected to certain beams due to their in adequate slenderness ratio.

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Hamburger, R., Krawinkler, H., Malley, J. & Adan, S. (2009). NEHRP Seismic Design Technical Brief No. 2. *Seismic Design of Steel Special Moment Frames: A Guide for Practical Engineers.* National Institute of Standards and Technology.

Appendix A - Beam/Column Connections without Doubler & Continuity Plates

List of Tables

Table A- 1 W33x Beams

Beam	Column			Include
W33x241	W14x730			-
W33x221	W36x652			-
	W14x730			-
W33x201	W40x593			-
	W36x652			-
	W14x665	up to	W14x730	-
W33x169	W40x503	up to	W40x593	-
	W36x529	up to	W36x652	-
	W27x539			-
	W14x605	up to	W14x730	-
W33x152	W40x431	up to	W40x593	W40x392
	W36x487	up to	W36x652	-
	W27x539			-
	W14x550	up to	W14x730	-
W33x141	W40x431	up to	W40x593	W40x392
	W36x441	up to	W36x652	-
	W27x539			-
	W14x500	up to	W14x730	-
W33x130	W40x397	up to	W40x593	-
	W40x331	up to	W40x392	-
	W36x441	up to	W36x652	-
	W27x539			-
	W14x500	up to	W14x730	-

Table A- 2 W30x Beams

<table>
<tr><td>Beam</td><td colspan="3">Column</td><td>Include</td></tr>
<tr><td rowspan="2">W30x235</td><td colspan="3">W36x652</td><td>-</td></tr>
<tr><td colspan="3">W14x730</td><td>-</td></tr>
<tr><td rowspan="3">W30x211</td><td colspan="3">W40x593</td><td>-</td></tr>
<tr><td colspan="3">W36x652</td><td>-</td></tr>
<tr><td>W14x665</td><td>up to</td><td>W14x730</td><td>-</td></tr>
<tr><td rowspan="4">W30x191</td><td colspan="3">W40x593</td><td>-</td></tr>
<tr><td colspan="3">W36x652</td><td>-</td></tr>
<tr><td colspan="3">W27x539</td><td>-</td></tr>
<tr><td>W14x605</td><td>up to</td><td>W14x730</td><td>-</td></tr>
<tr><td rowspan="4">W30x173</td><td>W40x503</td><td>up to</td><td>W40x593</td><td>-</td></tr>
<tr><td>W36x529</td><td>up to</td><td>W36x652</td><td>-</td></tr>
<tr><td colspan="3">W27x539</td><td>-</td></tr>
<tr><td>W14x550</td><td>up to</td><td>W14x730</td><td>-</td></tr>
<tr><td rowspan="4">W30x148</td><td>W40x431</td><td>up to</td><td>W40x593</td><td>W40x392</td></tr>
<tr><td>W36x441</td><td>up to</td><td>W36x652</td><td>-</td></tr>
<tr><td colspan="3">W27x539</td><td>-</td></tr>
<tr><td>W14x500</td><td>up to</td><td>W14x730</td><td>-</td></tr>
<tr><td rowspan="5">W30x132</td><td>W40x397</td><td>up to</td><td>W40x593</td><td>-</td></tr>
<tr><td>W40x331</td><td>up to</td><td>W40x392</td><td>-</td></tr>
<tr><td>W36x441</td><td>up to</td><td>W36x652</td><td>-</td></tr>
<tr><td colspan="3">W27x539</td><td>-</td></tr>
<tr><td>W14x455</td><td>up to</td><td>W14x730</td><td>-</td></tr>
</table>

Table A-2 W30x Beams (Continued)

Beam	Column			Include
W30x124	W40x362	up to	W40x593	-
	W40x327	up to	W40x392	-
	W36x395	up to	W36x652	-
	W33x387			-
	W30x391			-
	W27x539			-
	W24x370			-
	W14x455	up to	W14x730	-
W30x116	W40x362	up to	W40x593	-
	W40x327	up to	W40x392	-
	W36x361	up to	W36x652	-
	W33x354	up to	W33x387	-
	W30x357	up to	W30x391	-
	W27x368	up to	W27x539	-
	W24x370			-
	W14x426	up to	W14x730	-
W30x108	W40x324	up to	W40x593	-
	W40x294	up to	W40x392	-
	W36x361	up to	W36x652	-
	W33x354	up to	W33x387	-
	W30x357	up to	W30x391	-
	W27x336	up to	W27x539	-
	W24x335	up to	W24x370	-
	W14x398	up to	W14x730	-
W30x99	W40x324	up to	W40x593	-
	W40x278	up to	W40x392	-
	W36x330	up to	W36x652	-

Table A-2 W30x Beams (Continued)

Beam	Column			Include
W30x99	W33x318	up to	W33x387	-
	W30x326	up to	W30x391	-
	W27x307	up to	W27x539	-
	W24x306	up to	W24x370	-
	W18x311			-
	W14x370	up to	W14x730	-

Table A- 3 W27x Beams

Beam	Column			Include
W27x258	W14x730			-
W27x235	W36x652			-
	W14x730			-
W27x217	W40x593			-
	W36x652			-
	W14x665	up to	W14x730	-
W27x194	W40x593			-
	W36x652			-
	W27x539			-
	W14x605	up to	W14x730	-
W27x178	W40x503	up to	W40x593	-
	W36x529	up to	W36x652	-
	W27x539			-
	W14x605	up to	W14x730	-

Table A-3 W27x Beams (Continued)

Beam	Column			Include
W27x161	W40x503	up to	W40x593	-
	W36x487	up to	W36x652	-
	W27x539			-
	W14x550	up to	W14x730	-
W27x146	W40x431	up to	W40x593	W40x392
	W36x441	up to	W36x652	-
	W27x539			-
	W14x500	up to	W14x730	-
W27x129	W40x372	up to	W40x593	-
	W40x327	up to	W40x392	-
	W36x395	up to	W36x652	-
	W33x387			-
	W30x391			-
	W27x539			-
	W24x370			-
	W14x455	up to	W14x730	-
W27x114	W40x362	up to	W40x593	-
	W40x294	up to	W40x392	-
	W36x361	up to	W36x652	-
	W33x354	up to	W33x387	-
	W30x357	up to	W30x391	-
	W27x368	up to	W27x539	-
	W24x370			-
	W14x398	up to	W14x730	-

Table A-3 W27x Beams (Continued)

<table>
<tr><th>Beam</th><th colspan="3">Column</th><th>Include</th></tr>
<tr><td rowspan="10">W27x102</td><td>W40x324</td><td>up to</td><td>W40x593</td><td>-</td></tr>
<tr><td>W40x264</td><td>up to</td><td>W40x392</td><td>-</td></tr>
<tr><td>W36x302</td><td>up to</td><td>W36x652</td><td>-</td></tr>
<tr><td>W33x318</td><td>up to</td><td>W33x387</td><td>-</td></tr>
<tr><td>W30x326</td><td>up to</td><td>W30x391</td><td>-</td></tr>
<tr><td>W27x307</td><td>up to</td><td>W27x539</td><td>-</td></tr>
<tr><td>W24x306</td><td>up to</td><td>W24x370</td><td>-</td></tr>
<tr><td colspan="3">W18x311</td><td>-</td></tr>
<tr><td>W14x370</td><td>up to</td><td>W14x730</td><td>-</td></tr>
<tr><td colspan="3">W12x336</td><td>-</td></tr>
<tr><td rowspan="10">W27x94</td><td>W40x324</td><td>up to</td><td>W40x593</td><td>-</td></tr>
<tr><td>W40x264</td><td>up to</td><td>W40x392</td><td>-</td></tr>
<tr><td>W36x302</td><td>up to</td><td>W36x652</td><td>W36x256</td></tr>
<tr><td>W33x318</td><td>up to</td><td>W33x387</td><td>-</td></tr>
<tr><td>W30x326</td><td>up to</td><td>W30x391</td><td>-</td></tr>
<tr><td>W27x307</td><td>up to</td><td>W27x539</td><td>-</td></tr>
<tr><td>W24x306</td><td>up to</td><td>W24x370</td><td>-</td></tr>
<tr><td>W18x283</td><td>up to</td><td>W18x311</td><td>-</td></tr>
<tr><td>W14x370</td><td>up to</td><td>W14x730</td><td>-</td></tr>
<tr><td colspan="3">W12x336</td><td>-</td></tr>
<tr><td rowspan="5">W27x84</td><td>W40x324</td><td>up to</td><td>W40x593</td><td>-</td></tr>
<tr><td>W40x264</td><td>up to</td><td>W40x392</td><td>-</td></tr>
<tr><td>W36x302</td><td>up to</td><td>W36x652</td><td>W36x256</td></tr>
<tr><td>W33x291</td><td>up to</td><td>W33x387</td><td>-</td></tr>
<tr><td>W30x292</td><td>up to</td><td>W30x391</td><td>-</td></tr>
</table>

Table A-3 W27x Beams (Continued)

Beam	Column			Include
W27x94	W27x281	up to	W27x539	-
	W24x279	up to	W24x370	-
	W18x283	up to	W18x311	-
	W14x342	up to	W14x730	-
	W12x305	up to	W12x336	-

Table A- 4 W24x Beams

Beam	Column			Include
W24x279	W14x730			-
W24x250	W36x652			-
	W14x730			-
W24x229	W40x593			-
	W36x652			-
	W14x665	up to	W14x730	-
W24x207	W40x593			-
	W36x652			-
	W14x605	up to	W14x730	-
W24x192	W40x593			-
	W36x529	up to	W36x652	-
	W27x539			-
	W14x605	up to	W14x730	-
W24x176	W40x503	up to	W40x593	-
	W36x529	up to	W36x652	-
	W27x539			-
	W14x550	up to	W14x730	-

Table A-4 W24x Beams (Continued)

Beam	Column			Include
W24x162	W40x503	up to	W40x593	-
	W36x487	up to	W36x652	-
	W27x539			-
	W14x500	up to	W14x730	-
W24x146	W40x431	up to	W40x593	W40x392
	W36x441	up to	W36x652	-
	W27x539			-
	W14x455	up to	W14x730	-
W24x131	W40x397	up to	W40x593	W40x392
	W36x395	up to	W36x652	-
	W33x387			-
	W30x391			-
	W27x539			-
	W14x426	up to	W14x730	-
W24x117	W40x397	up to	W40x593	-
	W40x327	up to	W40x392	-
	W36x395	up to	W36x652	-
	W33x387			-
	W30x357	up to	W30x391	-
	W27x368	up to	W27x539	-
	W24x370			-
	W14x398	up to	W14x730	-

Table A-4 W24x Beams (Continued)

Beam	Column			Include
W24x104	W40x397	up to	W40x593	-
	W40x327	up to	W40x392	-
	W36x395	up to	W36x652	-
	W33x387			-
	W30x357	up to	W30x391	-
	W27x336	up to	W27x539	-
	W24x335	up to	W24x370	-
	W18x311			-
	W14x370	up to	W14x730	-
	W12x336			-
W24x103	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x330	up to	W36x652	-
	W33x318	up to	W33x387	-
	W30x326	up to	W30x391	-
	W27x336	up to	W27x539	-
	W24x306	up to	W24x370	-
	W18x311			-
	W14x370	up to	W14x730	-
	W12x336			-
W24x94	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x291	up to	W33x387	-
	W30x292	up to	W30x391	-
	W27x307	up to	W27x539	-
	W24x306	up to	W24x370	-
	W18x283	up to	W18x311	-
	W14x370	up to	W14x730	-
	W12x336			-

Table A-4 W24x Beams (Continued)

Beam	Column			Include
W24x84	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x263	up to	W33x387	-
	W30x261	up to	W30x391	-
	W27x281	up to	W27x539	-
	W24x279	up to	W24x370	-
	W18x258	up to	W18x311	-
	W14x342	up to	W14x730	-
	W12x305	up to	W12x336	-
W24x76	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x263	up to	W33x387	-
	W30x261	up to	W30x391	-
	W27x258	up to	W27x539	-
	W24x250	up to	W24x370	-
	W18x258	up to	W18x311	-
	W14x311	up to	W14x730	-
	W12x279	up to	W12x336	-
W24x68	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x263	up to	W33x387	-
	W30x235	up to	W30x391	-
	W27x235	up to	W27x539	-
	W24x229	up to	W24x370	-
	W18x234	up to	W18x311	-
	W14x283	up to	W14x730	-
	W12x252	up to	W12x336	-

Table A-4 W24x Beams (Continued)

Beam	Column			Include
W24x62	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x263	up to	W33x387	-
	W30x211	up to	W30x391	-
	W27x217	up to	W27x539	-
	W24x207	up to	W24x370	-
	W21x201			-
	W18x211	up to	W18x311	-
	W14x257	up to	W14x730	-
	W12x230	up to	W12x336	-

Table A- 5 W21x Beams

Beam	Column			Include
W21x201	W40x593			-
	W36x652			-
	W27x539			-
	W14x605	up to	W14x730	-
W21x182	W40x503	up to	W40x593	-
	W36x529	up to	W36x652	-
	W27x539			-
	W14x550	up to	W14x730	-
W21x166	W40x503	up to	W40x593	-
	W36x487	up to	W36x652	-
	W27x539			-
	W14x500	up to	W14x730	-

Table A-5 W21x Beams (Continued)

Beam	Column			Include
W21x147	W40x431	up to	W40x593	W40x392
	W36x441	up to	W36x652	-
	W27x539			-
	W14x455	up to	W14x730	-
W21x132	W40x397	up to	W40x593	W40x392
	W36x395	up to	W36x652	-
	W33x387			-
	W30x391			-
	W27x539			-
	W14x426	up to	W14x730	-
W21x122	W40x397	up to	W40x593	-
	W40x327	up to	W40x392	-
	W36x395	up to	W36x652	-
	W33x387			-
	W30x357	up to	W30x391	-
	W27x368	up to	W27x539	-
	W24x370			-
	W14x398	up to	W14x730	-
W21x111	W40x372	up to	W40x593	-
	W40x327	up to	W40x392	-
	W36x395	up to	W36x652	-
	W33x354	up to	W33x387	-
	W30x357	up to	W30x391	-
	W27x336	up to	W27x539	-
	W24x335	up to	W24x370	-
	W14x398	up to	W14x730	-

Beam	Column			Include
W21x101	W40x372	up to	W40x593	-
	W40x327	up to	W40x392	-
	W36x395	up to	W36x652	-
	W33x354	up to	W33x387	-
	W30x326	up to	W30x391	-
	W27x307	up to	W27x539	-
	W24x306	up to	W24x370	-
	W18x311			-
	W14x370	up to	W14x730	-
	W12x336			-
W21x93	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x291	up to	W33x387	-
	W30x292	up to	W30x391	-
	W27x281	up to	W27x539	-
	W24x279	up to	W24x370	-
	W18x283	up to	W18x311	-
	W14x342	up to	W14x730	-
	W12x305	up to	W12x336	-
W21x83	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x263	up to	W33x387	-
	W30x261	up to	W30x391	-
	W27x258	up to	W27x539	-
	W24x250	up to	W24x370	-
	W18x258	up to	W18x311	-
	W14x311	up to	W14x730	-
	W12x279	up to	W12x336	-

Table A-5 W21x Beams (Continued)

Beam	Column			Include
W21x73	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x263	up to	W33x387	-
	W30x235	up to	W30x391	-
	W27x235	up to	W27x539	-
	W24x229	up to	W24x370	-
	W18x234	up to	W18x311	-
	W14x283	up to	W14x730	-
	W12x252	up to	W12x336	-
W21x68	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x263	up to	W33x387	-
	W30x235	up to	W30x391	-
	W27x217	up to	W27x539	-
	W24x229	up to	W24x370	-
	W18x211	up to	W18x311	-
	W14x283	up to	W14x730	-
	W12x252	up to	W12x336	-
W21x62	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x263	up to	W33x387	-
	W30x235	up to	W30x391	-

Table A-5 W21x Beams (Continued)

Beam	Column			Include
W21x62	W27x217	up to	W27x539	-
	W24x207	up to	W24x370	-
	W21x201			-
	W18x211	up to	W18x311	-
	W14x257	up to	W14x730	-
	W12x230	up to	W12x336	-
W21x57	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x263	up to	W33x387	-
	W30x211	up to	W30x391	-
	W27x178	up to	W27x539	-
	W24x192	up to	W24x370	-
	W21x201			-
	W18x192	up to	W18x311	-
	W14x233	up to	W14x730	-
	W12x210	up to	W12x336	-
W21x55	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x263	up to	W33x387	-
	W30x235	up to	W30x391	-
	W27x217	up to	W27x539	-
	W24x207	up to	W24x370	-

Table A-5 W21x Beams (Continued)

Beam	Column			Include
W21x55	W21x201			-
	W18x211	up to	W18x311	-
	W14x257	up to	W14x730	-
	W12x230	up to	W12x336	-
W21x50	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x263	up to	W33x387	-
	W30x211	up to	W30x391	-
	W27x178	up to	W27x539	-
	W24x162	up to	W24x370	-
	W21x166	up to	W21x201	-
	W18x175	up to	W18x311	-
	W14x211	up to	W14x730	-
	W12x190	up to	W12x336	-
W21x44	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x263	up to	W33x387	-
	W30x211	up to	W30x391	-
	W27x178	up to	W27x539	-
	W24x146	up to	W24x370	-
	W21x147	up to	W21x201	-
	W18x158	up to	W18x311	-
	W14x193	up to	W14x730	-
	W12x170	up to	W12x336	-

Table A- 6 W18x Beams

Beam	Column			Include
W18x283	W14x730			-
W18x258	W36x652			-
	W14x665	up to	W14x730	-
W18x234	W40x593			-
	W36x652			-
	W14x665	up to	W14x730	-
W18x211	W40x593			-
	W36x652			-
	W27x539			-
	W14x605	up to	W14x730	-
W18x192	W40x503	up to	W40x593	-
	W36x529	up to	W36x652	-
	W27x539			-
	W14x550	up to	W14x730	-
W18x175	W40x503	up to	W40x593	-
	W36x487	up to	W36x652	-
	W27x539			-
	W14x500	up to	W14x730	-
W18x158	W40x431	up to	W40x593	W40x392
	W36x441	up to	W36x652	-
	W27x539			-
	W14x500	up to	W14x730	-
W18x143	W40x397	up to	W40x593	W40x392
	W36x441	up to	W36x652	-
	W27x539			-
	W14x455	up to	W14x730	-

Table A-6 W18x Beams (Continued)

Beam	Column			Include
W18x130	W40x372	up to	W40x593	-
	W40x331	up to	W40x392	-
	W36x395	up to	W36x652	-
	W33x387			-
	W30x391			-
	W27x368	up to	W27x539	-
	W24x370			-
	W14x426	up to	W14x730	-
W18x119	W40x362	up to	W40x593	-
	W40x294	up to	W40x392	-
	W36x361	up to	W36x652	-
	W33x354	up to	W33x387	-
	W30x357	up to	W30x391	-
	W27x336	up to	W27x539	-
	W24x335	up to	W24x370	-
	W14x398	up to	W14x730	-
W18x106	W40x362	up to	W40x593	-
	W40x294	up to	W40x392	-
	W36x361	up to	W36x652	-
	W33x318	up to	W33x387	-
	W30x326	up to	W30x391	-
	W27x307	up to	W27x539	-
	W24x306	up to	W24x370	-
	W18x311			-
	W14x370	up to	W14x730	-
	W12x336			-

Table A-6 W18x Beams (Continued)

Beam	Column			Include
W18x97	W40x362	up to	W40x593	-
	W40x294	up to	W40x392	-
	W36x330	up to	W36x652	-
	W33x318	up to	W33x387	-
	W30x292	up to	W30x391	-
	W27x307	up to	W27x539	-
	W24x306	up to	W24x370	-
	W18x283	up to	W18x311	-
	W14x342	up to	W14x730	-
	W12x305	up to	W12x336	-
W18x86	W40x362	up to	W40x593	-
	W40x294	up to	W40x392	-
	W36x330	up to	W36x652	-
	W33x318	up to	W33x387	-
	W30x292	up to	W30x391	-
	W27x281	up to	W27x539	-
	W24x279	up to	W24x370	-
	W18x258	up to	W18x311	-
	W14x311	up to	W14x730	-
	W12x279	up to	W12x336	-
W18x86	W40x362	up to	W40x593	-
	W40x294	up to	W40x392	-
	W36x330	up to	W36x652	-
	W33x318	up to	W33x387	-
	W30x292	up to	W30x391	-
	W27x281	up to	W27x539	-
	W24x250	up to	W24x370	-
	W18x234	up to	W18x311	-
	W14x283	up to	W14x730	-
	W12x252	up to	W12x336	-

Table A-6 W18x Beams (Continued)

Beam	Column			Include
W18x71	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x263	up to	W33x387	-
	W30x235	up to	W30x391	-
	W27x217	up to	W27x539	-
	W24x229	up to	W24x370	-
	W18x211	up to	W18x311	-
	W14x283	up to	W14x730	-
	W12x252	up to	W12x336	-
W18x65	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x263	up to	W33x387	-
	W30x211	up to	W30x391	-
	W27x217	up to	W27x539	-
	W24x207	up to	W24x370	-
	W21x201			-
	W18x211	up to	W18x311	-
	W14x257	up to	W14x730	-
	W12x230	up to	W12x336	-
W18x60	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x263	up to	W33x387	-
	W30x211	up to	W30x391	-
	W27x194	up to	W27x539	-
	W24x192	up to	W24x370	-

Table A-6 W18x Beams (Continued)

Beam	Column			Include
W18x60	W21x201			-
	W18x192	up to	W18x311	-
	W14x233	up to	W14x730	-
	W12x210	up to	W12x336	-
W18x55	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x263	up to	W33x387	-
	W30x211	up to	W30x391	-
	W27x178	up to	W27x539	-
	W24x176	up to	W24x370	-
	W21x182	up to	W21x201	-
	W18x175	up to	W18x311	-
	W14x233	up to	W14x730	-
	W12x210	up to	W12x336	-
W18x50	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x263	up to	W33x387	-
	W30x211	up to	W30x391	-
	W27x178	up to	W27x539	-
	W24x162	up to	W24x370	-
	W21x166	up to	W21x201	-
	W18x175	up to	W18x311	-
	W14x211	up to	W14x730	-
	W12x190	up to	W12x336	-

Table A-6 W18x Beams (Continued)

Beam	Column			Include
W18x46	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x263	up to	W33x387	-
	W30x211	up to	W30x391	-
	W27x178	up to	W27x539	-
	W24x146	up to	W24x370	-
	W21x147	up to	W21x201	-
	W18x158	up to	W18x311	-
	W14x193	up to	W14x730	-
	W12x170	up to	W12x336	-
W18x40	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x263	up to	W33x387	-
	W30x211	up to	W30x391	-
	W27x178	up to	W27x539	-
	W24x131	up to	W24x370	-
	W21x132	up to	W21x201	-
	W18x143	up to	W18x311	-
	W14x176	up to	W14x730	-
	W12x152	up to	W12x336	-
W18x35	W40x324	up to	W40x593	-
	W40x264	up to	W40x392	-
	W36x302	up to	W36x652	W36x256
	W33x263	up to	W33x387	-
	W30x211	up to	W30x391	-

Table A-6 W18x Beams (Continued)

Beam	Column			Include
W18x35	W27x178	up to	W27x539	-
	W24x131	up to	W24x370	-
	W21x132	up to	W21x201	-
	W18x119	up to	W18x311	-
	W14x159	up to	W14x730	-
	W12x152	up to	W12x336	-