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1. GENERAL

The Re-Fuse Braced Frame (RFBF) is a lateral load resisting system consisting of traditional, rolled section brace members that are connected to gusset plates via specially engineered, proprietary steel fuse elements. The fuse elements have unique geometry designed to accommodate large inelastic deformations under extreme seismic events while the remaining structure is intended to remain essentially elastic. Fuse elements are combined to form specific connection assemblies referred to as fuse configurations, which are used to connect vertical brace members to gusset plates (see Appendix A for details). Brace assemblies consist of vertical brace members, fuse configurations, and gusset plates. A maximum of two fuse configurations may be used in a single brace assembly, one at each end. The fuse elements are intended to be replaced if damaged in a significant seismic event. This design guide contains provisions and supplemental detail information and recommendations for analysis, design, and detailing of the RFBF system.

The fuse elements and their application as used within the RFBF system are proprietary under United States patent rights. The system may only be used through grant of license from Novel Structures, LLC of Wyoming. Design shall be performed in accordance with the provisions of this design guide and shall be peer reviewed by an entity approved by Novel Structures, LLC. Construction shall be performed in accordance with “Specification for Re-Fuse Braced Frame System,” Novel Structures, LLC, 2016, Cheyenne, WY.

References used in conjunction with this design guide include:


Except where noted otherwise in this document, the requirements of AISC 360 and AISC 341 as referenced above shall govern the general requirements, analysis, design, fabrication, erection, and quality control of the RFBF seismic force resisting system.
2. ANALYSIS AND MEMBER DESIGN

Required strength, limitations on structural height and irregularity, determination of seismic design category (SDC) and risk category, and other design requirements shall be as determined in the applicable building code.

The seismic design coefficients and factors for the RFBF system for use with provisions of the applicable code are consistent with those of the buckling-restrained braced frame (BRBF) system as follows:

- **Response Modification Coefficient**: \( R = 8 \)
- **Overstrength Factor**: \( \Omega_0 = 2\frac{1}{2} \)
- **Deflection Amplification Factor**: \( C_d = 5 \)

2.1 Analysis and System Requirements

2.1.1 Analysis and Deformations

Structural analysis shall be performed in accordance with the provisions of the applicable building code. The stiffness of the **fuse configurations** and brace assembly in the RFBF system shall be determined in accordance with Section 3.3.

The amplified brace deformation, \( \Delta_{\text{bm}} \), shall be taken as the work point-to-point deformation of the **fuse configuration** and brace and assembly corresponding to the design story drift determined in accordance with the applicable building code. The amplified brace deformation shall not exceed \( n_{\text{fuse}}\Delta_{\text{max,fuse}} \), where

\[
n_{\text{fuse}} = \text{number of specified fuse configurations between work points in series with a brace (} n_{\text{fuse}} = 1 \text{ for fuse at one end of brace, } n_{\text{fuse}} = 2 \text{ for fuses at both ends of brace)}
\]

\[
\Delta_{\text{max,fuse}} = \text{maximum design deformation of an individual fuse configuration taken as 0.75 in. (19 mm)}
\]

**Fuse configurations** shall not be considered as resisting gravity loads.

2.1.2 RFBF System Configurations

RFBF may utilize single diagonal, V, inverted-V, and X bracing configurations. **Fuse elements** shall be oriented concentric to the longitudinal axis of brace members. **Fuse configuration** and brace assemblies may be concentrically or eccentrically connected to beams and columns provided eccentricities do not exceed the beam depth. Member and connection forces resulting from eccentric connections shall be addressed in member design while maintaining the **fuse elements** as the expected source of inelastic deformation.

Brace connections at the intersection of braces in X-braced configurations shall develop the stiffness and available strength of the braces axially and in flexure about the strong and weak axis of the brace.

K-type braced frames shall not be used in RFBF.
Exception:

(1) Where structures are assigned to SDC A, B, or C as determined in accordance with the applicable building code, K-type braced frames and eccentricities exceeding the beam depth may be used provided the available strength of members exceeds the forces resulting from an analysis using a response modification factor, \( R = 3 \), but not less than the forces required to develop the adjusted fuse configuration strength as defined in Section 3.2.

A multi-tiered braced frame (MTBF) is defined as braced-framed configuration with two or more tiers of bracing between diaphragm levels or locations of out-of-plane bracing. The RFBF may be used in MTBF provided connections of horizontal struts to columns at each tier provide torsional bracing of the column in accordance with Appendix 6 of AISC 360.

2.2 Member and Connection Design

2.2.1 Strength of members and connections

The required strength of braces, columns, beams, and connections shall be based on the load combinations of the applicable building code including the overstrength seismic load.

In determining the overstrength seismic load for braces, columns, beams, and connections, the effect of horizontal forces including overstrength factor, \( E_{mh} \), shall be taken as the forces developed in the member assuming the forces in all brace assemblies correspond to their adjusted strength as defined in Section 3.2. Additionally, for braces \( E_{mh} \) shall not be taken as less than the effects of horizontal seismic forces determined in accordance with the applicable code multiplied by the overstrength factor, \( \Omega_b \).

User Note: The additional overstrength requirement for braces is in accordance with Section 12.4.3.1 of ASCE 7. This requirement is typically more stringent than the capacity limited requirement stated formally for braces, columns, beams, and connections because brace buckling is considered detrimental to the proper performance of the system.

Exception:

(1) The required strength of columns need not exceed the lesser of the following:
   a. Forces corresponding to the resistance of the foundation to overturning uplift.
   b. Forces as determined from nonlinear analysis as defined in Section C3 of AISC 341.

The frame forces and associated amplified drifts in orthogonal directions shall be determined and combined in accordance with the applicable building code. Where non-parallel seismic force resisting frames share a column, the adjusted brace assembly strength associated with the combined drift state for all brace assemblies framing into the shared column shall be determined and the vertical component of the adjusted brace assembly strengths shall be applied to the column at each level.

Columns shall be subject to the additional strength requirements of Section D1.4 of AISC 341. Column splices shall be subject to the additional requirements of Section D2.5 of AISC 341. Column bases shall be subject to the additional requirements of Section D2.6 of AISC 341.
Braces, columns, and beams shall meet the requirements for moderately ductile members as defined within AISC 341. Braces shall be wide flange members with 12 inch nominal depth (W12).

The connection of braces to gusset plates using *fuse elements* shall be in accordance with the connection details of Appendix A and specification of Appendix B.

There are no specific detailing requirements for the connection of gusset plates to columns and beams or struts.

Beam-to-column connections shall be simple connections meeting the requirements of section B3.6a of AISC 360 where the required rotation is taken to be 0.025 radians. Beam-to-column connections may be assumed pinned for the purposes of analysis. Alternatively, a reasonable estimation of the rotational stiffness characteristics of the beam-to-column connection shall be used in the analysis.

**User Note:** Ease of removal and reinstallation of *fuse elements* provides opportunity for enhanced reparability of the structural steel frame provided damage to the primary members is limited. Recommended measures to further limit damage to the primary structural frame are discussed in the Commentary. Pre-designed gusset-to-beam/column and beam-to-column connections meeting the recommendations discussed in the Commentary are available in Appendix D.

### 2.2.2 Protected Zones

The protected zone is portion of the *fuse elements* expected to undergo inelastic deformations as encapsulated by the retainer plates and web of the brace.

**User Note:** The Re-Fuse Brace Connection view in Figure 1 in Appendix A depicts the outline of the *fuse elements* expected to undergo inelastic deformation beneath the retainer plate.

### 3. STRENGTH AND STIFFNESS PROVISIONS

Respective brace strength and stiffness parameters shall be determined in accordance with this section.

#### 3.1 Available Fuse Configuration Strength

The available *fuse configuration* strength, $\phi P_n$, for a specified *fuse configuration* shall be taken as tabulated in Table 1. The available *fuse configuration* strength shall exceed the required strength as determined from analysis using the Load and Resistance Factor Design (LRFD) load combination of the applicable building code.

#### 3.2 Adjusted Fuse Strength

The adjusted *brace assembly* strength, $P_{pr}$, shall be determined by EQ 1,

$$P_{pr} = R_{y, fuse} \left( P_n + \phi \Delta_{assembly} \right)$$

EQ 1
where

\[ R_{y,\text{fuse}} = \text{ratio of expected yield stress to minimum specified yield stress of the fuse elements is 1.1} \ (\text{fuse elements are manufactured from A572 Gr. 50 material}) \]

\[ P_n = \text{nominal strength of a fuse configuration at the minimum specified yield stress provided in Table 1, kips (kN)} \]

\[ \omega = \text{strain and geometric hardening adjustment factor for a fuse configuration provided in Table 1, k/in. (kN/m)} \]

\[ \Delta_{\text{assembly}} = \text{approximate design deformation of a brace assembly,} \Delta_{\text{brm}}, \text{in. (m), regardless of the number of fuse configurations utilized} \]

Alternatively, for any brace assembly regardless of the calculated design deformation, the adjusted brace assembly strength may conservatively be taken as the expected strength at the maximum design deformation, \( P_{p,\text{max}} \), determined as \( R_{y,\text{fuse}}P_n \) as provided in Table 1

where

\[ P_{p} = \text{anticipated strength of the fuse configuration at the maximum design deformation is} \ P_n + \omega P_{n,\text{fuse}} \Delta_{\text{max, fuse}} \text{as provided in Table 1, kips (kN)} \]

**User Note:** While the total deformation within a brace assembly does distribute between multiple fuse configurations allowing for increased deformation capacity, testing has shown that assuming all deformation occurs in one fuse configuration provides a more accurate prediction of force within the assembly.

### 3.3 Brace Stiffness

The effective elastic axial stiffness of the brace, fuse configurations, and brace connection assembly, \( k_{e,\text{eff}} \), to be used for structural analysis is given by EQ 2.

\[
k_{e,\text{eff}} = \frac{1}{\frac{1}{k_{e,\text{brace}}} + \frac{n_{\text{fuse}}}{k_{e,\text{fuse}}}} \tag{EQ 2}
\]

where

\[ k_{e,\text{brace}} = \text{approximate axial stiffness of the brace and brace connection assembly less the fuse configurations} A_{\text{brace}}E_s/(L_{\text{wp}} - n_{\text{fuse}}L_{\text{fuse}}), \text{ k/in. (kN/m)} \]

\[ k_{e,\text{fuse}} = \text{axial stiffness of an individual fuse configuration as tabulated in Table 1 k/in, (kN/m)} \]

\[ A_{\text{brace}} = \text{gross cross sectional area of the brace member, in.}^2 \text{ (m}^2) \]

\[ E_s = \text{the elastic modulus of elasticity of steel taken as 29,000 ksi (200 GPa)} \]

\[ L_{\text{wp}} = \text{work point-to-work point dimension along the brace and fuse configuration assembly, in. (m)} \]

\[ L_{\text{fuse}} = \text{approximate effective length of a fuse element taken as 12 in. (0.3 m)} \]
Table 1. Re-Fuse Brace Connection Design Parameters

<table>
<thead>
<tr>
<th>Fuse Configuration</th>
<th>$\phi P_n$, kips</th>
<th>$P_n$, kips</th>
<th>$\omega$, kips/in.</th>
<th>$P_o$, kips</th>
<th>$P_{pr,\max}$, kips</th>
<th>$k_{e,fuse}$, kips/in.</th>
<th>$A_{equiv,fuse}^a$, in.$^2$</th>
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</thead>
<tbody>
<tr>
<td>RF-30</td>
<td>30</td>
<td>33</td>
<td>11</td>
<td>42</td>
<td>55</td>
<td>415</td>
<td>0.14</td>
</tr>
<tr>
<td>RF-60</td>
<td>60</td>
<td>67</td>
<td>22</td>
<td>83</td>
<td>110</td>
<td>830</td>
<td>0.29</td>
</tr>
<tr>
<td>RF-90</td>
<td>90</td>
<td>100</td>
<td>33</td>
<td>125</td>
<td>164</td>
<td>1245</td>
<td>0.43</td>
</tr>
<tr>
<td>RF-120</td>
<td>120</td>
<td>133</td>
<td>44</td>
<td>166</td>
<td>219</td>
<td>1660</td>
<td>0.57</td>
</tr>
<tr>
<td>RF-150</td>
<td>150</td>
<td>167</td>
<td>55</td>
<td>208</td>
<td>274</td>
<td>2075</td>
<td>0.71</td>
</tr>
<tr>
<td>RF-180</td>
<td>180</td>
<td>200</td>
<td>66</td>
<td>250</td>
<td>329</td>
<td>2490</td>
<td>0.86</td>
</tr>
<tr>
<td>RF-210</td>
<td>210</td>
<td>233</td>
<td>77</td>
<td>291</td>
<td>384</td>
<td>2905</td>
<td>1.00</td>
</tr>
</tbody>
</table>

$^a$ Area of a 12-inch length of section producing axial stiffness equivalent to $k_{e,fuse}$.

User Note: Recommended analytical modeling techniques to simulate effective brace assemblies within a variety of analytical software are discussed in Appendix C.
1. REPARABLE DESIGN AND DETAILING RECOMMENDATIONS

The recommendations of this section are not required for adequate stability and strength performance of the RFBF system, but are intended to limit damage to primary structural members and enhance the reparability. The details of Appendix C provide examples incorporating these recommendations.

1.1 Damage Mitigation of Primary Members and Connection Components

While inelastic deformations are substantially isolated to the fuse elements in the RFBF system, it is possible that damage to other elements, particularly gusset plates or primary members at gusset plate connections, can occur during any seismic event invoking inelastic behavior in the seismic force resisting system. Where gusset plates connect to both a beam and a column, it is recommended that member orientation and/or connection details be used that assist in alleviating “pinching forces” between the gusset plates and primary members due to large story drifts. Pinching forces, as referred to herein, are the forces resulting from the inherent tendency of the angle between a beam and column to reduce (close) as the structural frame drifts in a given direction.

Orienting primary members such that at least one side of the gusset plate is connected to the web of either the beam or the column without edge stiffeners has been shown to economically mitigate pinching forces due to the relative out-of-plane flexibility of the member web (McManus et al, 2013). Pre-designed braced frame connections meeting this intent are available in Appendix D. Commentary section C-F2.6b of AISC 341 provides an alternate approach whereby frame rotation is accommodated in a flexible, or pinned, beam connection located beyond the extends of the gusset connection. The cost of an additional shop-fabricated beam connection is inherent in the latter approach.

1.2 Reparable Gusset-to-Beam and Column Connections

Though welded construction is in no way precluded, it is recommended that gusset plates be bolted to beams and columns to enhance the ability to replace the connection components, thus enhancing the reparability of the system. Wide flange beams and columns are recommended to facilitate bolted connections.

Bearing bolts in standard holes should be used to connect gusset plates to double angle connection assemblies at beam/column interfaces, and at double angle or end plate connection assemblies to primary beams and columns.

Staggered bolts are recommended in double angle connection assemblies to allow for reduced bolt gauges on the flanges of the primary members.

Where an end plate or double angle connection assembly is connected to the flange of a primary member, the bending capacity of the primary member flange, including the effects of prying action, should exceed that of the outstanding legs of the end plate or connection angles to reduce the possibility of inducing yield in the primary member flange.
APPENDIX A – RE-FUSE BRACED FRAME BRACE-TO-GUSSET CONNECTION DETAILS

RE-FUSE BRACE CONNECTION

GUSSET PL

BRACE FLANGE

1/4" CLR OF WEB ±1/8"
Figure 1. Re-Fuse Braced Frame – Brace-to-Gusset Connection Configuration
APPENDIX B – RE-FUSE BRACED FRAME SPECIFICATION

SECTION 051201 – RE-FUSE BRACED FRAME SYSTEM

PART 1 - GENERAL

1.1 SUMMARY

A. Section Includes:
   1. Prefabricated fuse elements.
   2. Vertical brace-to-gusset connection components and fasteners.

B. Related Sections:
   1. Section 051200 "Structural Steel Framing" for definitions and structural steel elements within the braced frame not addressed in this Section.

1.2 DEFINITIONS

A. Re-Fuse (RF) Fuse: Proprietary structural steel components of unique geometry by Novel Structures, LLC of Cheyenne, Wyoming providing specific inelastic deformation capacity and predictable elastic and inelastic strength.

B. Re-Fuse Braced Frame (RFBF): Seismic force resisting system utilizing RF Fuses to connect vertical braces to gusset plates.

C. Retainer Plates: U-shaped plates used to encapsulate RF Fuses in RFBF connection assemblies.

1.3 ACTION SUBMITTALS

A. Product Data: For each type of product indicated.

B. Shop and Erection Drawings: Show location, fabrication, and assembly of structural-steel braces and/or gusset plates equipped with RF Fuses.

   1. Location of each piece or detail within the structure.
   2. Include details of cuts, connections, splices, holes, and other pertinent data.
   3. Indicate type, size, and length of bolts, distinguishing between shop and field bolts. Identify pretensioned and slip-critical high-strength bolted connections.
   4. Identify braces and connections as being part of the seismic force resisting system.
   5. Indicate locations and dimensions of protected zones.
1.4 INFORMATIONAL SUBMITTALS

A. Mill test reports for braces, fuses, and connection material including chemical and physical properties.

B. Product Test Reports: For the following if present on project:
   1. Bolts, nuts, and washers including mechanical properties and chemical analysis.
   2. Direct-tension indicators.
   3. Tension-control, high-strength bolt-nut-washer assemblies.

1.5 QUALITY ASSURANCE

A. Comply with applicable provisions of the following specifications and documents:
   1. AISC 303.
   2. AISC 341.
   3. AISC 360.
   4. RCSC's "Specification for Structural Joints Using ASTM A 325 or A 490 Bolts."

B. Preinstallation Conference: Conduct conference at Project site.

1.6 DELIVERY, STORAGE, AND HANDLING

A. Store materials to permit easy access for inspection and identification. Keep steel members off ground and spaced by using pallets, dunnage, or other supports and spacers. Protect steel members and packaged materials from corrosion and deterioration.
   1. Do not store materials on structure in a manner that might cause distortion, damage, or overload to members or supporting structures. Repair or replace damaged materials or structures as directed.

B. Store fasteners in a protected place in sealed containers with manufacturer's labels intact.
   1. Fasteners may be repackaged provided Owner's testing and inspecting agency observes repackaging and seals containers.
   2. Clean and relubricate bolts and nuts that become dry or rusty before use.
   3. Comply with manufacturers' written recommendations for cleaning and lubricating ASTM F 1852 fasteners and for retesting fasteners after lubrication.

1.7 COORDINATION

A. Where components are to be painted, coordinate selection of shop primers with topcoats to be applied over them. Comply with paint and coating manufacturers' recommendations to ensure that shop primers and topcoats are compatible with one another.
PART 2 - PRODUCTS

2.1 STRUCTURAL-STEEL MATERIALS

A. W-Shapes for Braces: ASTM A 992/A 992M unless indicated otherwise.

B. Gusset, Connection, and Retainer Plates: ASTM A 36/A 36M or ASTM A 572/A 572M, Grade 50 unless indicated otherwise.

C. RF Fuse: ASTM A 572/A 572M, Grade 50.

2.2 BOLTS, CONNECTORS, AND ANCHORS

A. High-Strength Bolts or Threaded Rods, Nuts, and Washers: ASTM A 325 (ASTM A 325M), Type 1, heavy-hex steel structural bolts or ASTM A 449 threaded rods; ASTM A 563, Grade C, (ASTM A 563M, Class 8S) heavy-hex carbon-steel nuts; and ASTM F 436 (ASTM F 436M), Type 1, hardened carbon-steel washers; all with plain finish.

1. Finish: Plain unless indicated to be galvanized. Hot-dip zinc coating where indicated to be galvanized.

2. Direct-Tension Indicators where indicated: ASTM F 959, Type 325 (ASTM F 959M, Type 8.8), compressible-washer type with plain finish unless indicated to be galvanized. Mechanically deposited zinc coating where indicated to be galvanized.

B. Tension-Control, High-Strength Bolt-Nut-Washer Assemblies: ASTM F 1852, Type 1, round head assemblies consisting of steel structural bolts with splined ends, heavy-hex carbon-steel nuts, and hardened carbon-steel washers.

1. Finish: Plain unless indicated to be galvanized. Hot-dip zinc coating where indicated to be galvanized.

2.3 FABRICATION

A. Structural Steel: Fabricate and assemble in shop to greatest extent possible. Fabricate according to AISC 303, AISC 360 and AISC 341.

1. Mark and match-mark materials for field assembly.
2. Complete structural-steel assemblies before starting shop-priming operations, if applicable.

B. Thermal Cutting of Braces and Connection Components other than RF Fuses: Perform thermal cutting by machine to greatest extent possible.

C. Cutting of RF Fuses:

1. Perimeter cut paths and bolt hole cut paths may be thermal cut.
2. Interior cut paths other than bolt holes shall be cut by waterjet. Overall tolerance of interior cut paths is ±0.01 in. Thermal cutting of interior cut paths is not permitted.
D. Bolt Holes: Cut, drill, mechanically thermal cut, or punch bolt holes perpendicular to metal surfaces. Do not enlarge bolt holes by burning.

E. Cleaning: Clean and prepare steel surfaces that are to remain unpainted according to SSPC-SP 2, "Hand Tool Cleaning" or SSPC-SP 3, "Power Tool Cleaning."

F. Splices: Splicing of brace members to obtain required lengths is not permitted without prior approval of Structural Engineer-of-Record unless indicated otherwise.

G. Substitutions: Where exact sizes and weights indicated are not readily available, secure approval of alternate sizes from Structural Engineer-of-Record in time to prevent project delay.

2.4 SHOP CONNECTIONS

A. High-Strength Bolts: Shop install high-strength bolts according to RCSC's "Specification for Structural Joints Using ASTM A325 or A490 Bolts" for type of bolt and type of joint specified.

1. Joint Type at RF Fuses: Slip critical.
2. Joint Type at Slotted Plate Connections to Braces: Pretensioned.

2.5 SHOP PRIMING AND PAINTING

A. Where steel is indicated to be primed, shop prime steel surfaces except the following:

1. Surfaces to be high-strength bolted with slip-critical connections.
2. Surfaces to receive sprayed fire-resistant materials (applied fireproofing).

B. Surface Preparation: Clean surfaces to be painted. Remove loose rust and mill scale and spatter, slag, or flux deposits. Prepare surfaces according to either of the following specifications and standards unless an alternate specification or standard is required for the paint process provided:

1. SSPC-SP 2, "Hand Tool Cleaning."
2. SSPC-SP 3, "Power Tool Cleaning."

C. Priming and Painting: Where components are required to be painted and/or primed for corrosion resistance, the surface preparation, primer, and paint process shall achieve a Class A faying surface in accordance with RCSC's "Specification for Structural Joints Using ASTM A325 or A490 Bolts" between RF Fuses and at interfaces with braces, gusset plates, and retainer plates.

2.6 GALVANIZING

A. Hot-Dip Galvanized Finish: Apply zinc coating by the hot-dip process to braces and connection plates according to ASTM A123/A123M where indicated. Galvanizing of RF Fuses is not permitted.
2.7 SOURCE QUALITY CONTROL

A. Testing and Inspection:

1. Perform testing and inspection in accordance with AISC 360 and AISC 341.
2. Inspect cut paths of 100% of RF Fuses to verify cut paths are free of notches and defects.
3. Polish and etch the exterior edge 2% of RF Fuses to verify grain direction of steel plate material is parallel to short direction of RF Fuse. Where rejection rate exceeds 1% frequency of inspection shall be increased to 100% of RF Fuses until rejection rate is reduced to 1% whereby inspection frequency may then be reduced to 10% of RF Fuses.

PART 3 - EXECUTION

3.1 ERECTION

A. Set structural steel accurately in locations and to elevations indicated and according to AISC 303 and AISC 360.

B. Maintain erection tolerances of structural steel within AISC's "Code of Standard Practice for Steel Buildings and Bridges."

C. Align and adjust various members that form part of complete frame or structure before permanently fastening. Before assembly, clean surfaces that will be in permanent contact with members. Perform necessary adjustments to compensate for discrepancies in elevations and alignment.

1. Level and plumb individual members of structure.
2. Make allowances for difference between temperature at time of erection and mean temperature when structure is completed and in service.

D. Splice members only where indicated.

E. Do not use thermal cutting during erection unless approved by Structural Engineer-of-Record. Finish thermally cut sections within smoothness limits in AWS D1.1/D1.1M. Thermal cutting of RF Fuses is not permitted.

F. Do not enlarge unfair holes in members by burning or using drift pins. Ream holes that must be enlarged to admit bolts. Welding of RF Fuses is not permitted unless approved by Structural Engineer-of-Record and Novel Structures, LLC.

3.2 FIELD CONNECTIONS

A. High-Strength Bolts and/or Threaded Rods: Install high-strength bolts according to RCSC's "Specification for Structural Joints Using ASTM A 325 or A 490 Bolts" for type of bolt and type of joint specified.

1. Joint Type at RF Fuses: Slip critical.
2. Joint Type at Slotted Plate Connections to Braces: Pretensioned.
3.3 FIELD QUALITY CONTROL

A. Testing and Inspection: Perform testing and inspection in accordance with AISC 360 and AISC 341.

3.4 REPAIRS

A. Repairs of damaged materials or surfaces: All repairs shall be approved by the Structural Engineer-of-Record and Novel Structures, LLC.

END OF SECTION 051201
Using the Equivalent Lateral Force Procedure of ASCE 7 with a Response Modification Coefficient, $R$, of 8, preliminary analysis of a five story building results in the frame forces shown in Figure C-1 under the critical load combination including earthquake loads. It assumed the critical load combination including earthquake loads governs the design over load combinations with wind.
Step 1 – Strength-Based Fuse Selection

The ultimate forces in each brace resulting from the preliminary analysis are summarized in Table C-1, for which a corresponding fuse is selected such that the fuse capacity, $\phi P_n$, taken from Table 1 meets or exceeds the ultimate force in the brace member from the analysis, $P_u$.

<table>
<thead>
<tr>
<th>Story</th>
<th>Brace Force $P_u$, kips</th>
<th>Fuse Configuration</th>
<th>Fuse Capacity $\phi P_n$, kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>209</td>
<td>RF-210</td>
<td>210</td>
</tr>
<tr>
<td>2</td>
<td>195</td>
<td>RF-210</td>
<td>210</td>
</tr>
<tr>
<td>3</td>
<td>161</td>
<td>RF-180</td>
<td>180</td>
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<td>4</td>
<td>107</td>
<td>RF-120</td>
<td>120</td>
</tr>
<tr>
<td>5</td>
<td>32</td>
<td>RF-60</td>
<td>60</td>
</tr>
</tbody>
</table>

Step 2 – Brace Design

The required strength of the brace, $P_{ubr}$, is the brace force determined from the preliminary analysis multiplied by the overstrength factor, $\Omega_0$, but not less than the adjusted brace assembly strength at the design story drift, which can conservatively be taken as $P_{pr,max}$ from Table 1. For the brace at story 1, the required strength is determined as follows:

$$P_{ubr1} = (209 \text{ kips})(2.5) = 523 \text{ kips} \geq P_{pr,max} = 384 \text{ kips}, \quad P_{ubr} = 523 \text{ kips}$$

The work point-to-work point length of each brace, $L_{br}$, is determined from the frame geometry as:

$$L_{br} = \sqrt{H^2 + L^2} = \sqrt{(17.5 \text{ ft})^2 + (8.75 \text{ ft})^2} = 19.6 \text{ ft}$$

Using Table 4-1 of the AISC Steel Construction Manual, 14th Edition for an unbraced length about the weak axis of 20 ft, a moderately ductile W12x72 is selected for which $\phi P_n = 602$ kips. The brace design for each story is summarized in Table C-2.

<table>
<thead>
<tr>
<th>Story</th>
<th>Required Brace Strength $P_u$, kips</th>
<th>Brace Section</th>
<th>Brace Axial Capacity $\phi P_n$, kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>523</td>
<td>W12x72</td>
<td>602</td>
</tr>
<tr>
<td>2</td>
<td>488</td>
<td>W12x72</td>
<td>602</td>
</tr>
<tr>
<td>3</td>
<td>403</td>
<td>W12x72</td>
<td>602</td>
</tr>
<tr>
<td>4</td>
<td>268</td>
<td>W12x53</td>
<td>354</td>
</tr>
<tr>
<td>5</td>
<td>81</td>
<td>W12x40</td>
<td>173</td>
</tr>
</tbody>
</table>

Step 3 – Column Design

The maximum required compressive strength of the column, $P_{uc}$, is determined from load combination 5 of ASCE 7 section 12.4.3.2 given as $(1.2 + 0.2S_{D0})D + \Omega_0 Q_E + L + 0.2S$. The overstrength seismic load, $\Omega_0 Q_E$ is the axial force in the column tier under consideration associated with the adjusted fuse force in all brace assemblies framing into and above the upper end of that tier. The axial forces in each column tier attributed to dead, live and snow loads $[(1.2 + 0.2S_{D0})D + L + 0.2S]$ are given in Table C-3. The adjusted fuse strength can be calculated based on the amplified brace deformation, $\Delta_{fbr}$, associated with the design story drift determined in accordance with ASCE 7.
Using this approach for the column tier at story 1, the required strength of one brace at story 2 is determined as follows:

\[ k_{e,\text{brace}} = \frac{A_{\text{brace}}E}{L_{wp} - n_{\text{fuse}}L_{\text{fuse}}} = \frac{(21.1 \text{ in.}^2)(29,000 \text{ ksi})/(19.6 \text{ ft})(12 \text{ in.}/\text{ft}) - 2(12 \text{ in.})} = 2,897 \text{ k/in.} \]

\[ k_{e,\text{fuse}} = 2,905 \text{ k/in.} \]

\[ k_{e,\text{eff}} = \frac{1}{1/(k_{e,\text{brace}}) + 2/(k_{e,\text{fuse}})} = 967 \text{ k/in.} \]

Typically the calculated stiffness of all the braces would be utilized within an analysis model that includes P-Δ effects to determine elastic story drifts, which would then be amplified in accordance with ASCE 7 to determine design story drifts. Representing the stiffness of the fuse and brace assembly can be accomplished using many methods, three of which are as follow:

1. Some software allow for an axial spring or partial-fixity restraint to be applied to the end of frame elements. In such an instance, the brace may modeled using the member size as determined in Step 2 with rotationally unrestrained end conditions ("pinned" ends). The stiffness of the fuse element, \( k_{e,\text{fuse}} \), taken from Table 1 can then be assigned as a partial-fixity axial restraint at each end of the brace.

2. The brace can be modeled using the member size as determined in Step 2. At each end of the brace, a 12 inch long steel element can modeled to connect the end of the brace member to the beam-column intersection. The cross-sectional area, \( A_{\text{equiv, fuse}} \), taken from Table 1 can then be assigned to the 12 inch long steel elements to represent the fuse stiffness. The connectivity between the 12 inch long steel elements and the brace member shall be fully restrained in all directions to maintain stability. The connectivity between the 12 inch long steel elements and beam-column intersections shall be rotationally unrestrained ("pinned").

3. The total effective stiffness of the brace and fuse assembly, \( k_{e,\text{eff}} \), may be calculated as shown previously in Step 3. The brace may be modeled as an element with modulus of elasticity and cross-sectional area proportioned to provide a stiffness equal to \( k_{e,\text{eff}} \).

Recognizing the contribution of axial stiffness of the beams and columns to the inelastic drift of the frame is minimal, and that P-Δ effects are also relatively small for braced frame systems, these can be neglected for the illustrative purposes of this example. In doing so \( \Delta_{\text{def}} \) can be approximated as the elastic deformation of the brace assembly multiplied by the deflection amplification factor, \( C_d \), assuming the importance factor, \( I_e \), is taken as 1.0.

\[ \Delta_{\text{def}} = 5(209 \text{ kips})/(967 \text{ k/in.}) = 1.08 \text{ in.} \]

\[ P_{pr,2} = R_{y,\text{fuse}} (P_{n2} + \omega_2\Delta_{\text{assembly,2}}) = 1.1(233 \text{ kips} + (77 \text{ k/in.})(1.08 \text{ in.})) = 348 \text{ kips} \]

Alternatively, the adjusted fuse strength can conservatively be taken as \( P_{pr,\text{max}} = 384 \text{ kips} \) as tabulated in Table 1. While more conservative, using \( P_{pr,\text{max}} \) does not require calculation of the design story drift to determine the force to the columns. The force in the column tier at story 1 associated with the adjusted brace force taken as \( P_{pr,\text{max}} \) for all braces is:

\[ P_{uc1} = (1.2 + 0.2S_{DS})D + L + 0.2S + (P_{pr,\text{max}2} + P_{pr,\text{max}3} + P_{pr,\text{max}4} + P_{pr,\text{max}5})(H/H_{le}) \]

\[ P_{uc1} = 546 \text{ kips} + 343 \text{ kips} + 294 \text{ kips} + 196 \text{ kips} + 98 \text{ kips} = 1477 \text{ kips} \]

Using Table 4-1 of the AISC Steel Construction Manual, 14th Edition with an unbraced length about the weak axis of 18 ft, a moderately ductile W14x145 is selected for which \( \phi P_n = 1550 \text{ kips} \). The column design for each story is summarized in Table C-3 assuming a splice above level 3.
### Table C-3. Column Design Summary

<table>
<thead>
<tr>
<th>Story / Tier</th>
<th>(1.2 + 0.2SDS)D + L + 0.2S</th>
<th>Ω0QE</th>
<th>Required Column Strength Pu, kips</th>
<th>Column Section</th>
<th>Column Axial Capacity φPn, kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>546</td>
<td>343</td>
<td>1477</td>
<td>W14x145</td>
<td>1550</td>
</tr>
<tr>
<td>2</td>
<td>420</td>
<td>343</td>
<td>1008</td>
<td>W14x145</td>
<td>1550</td>
</tr>
<tr>
<td>3</td>
<td>294</td>
<td>294</td>
<td>588</td>
<td>W14x82</td>
<td>620</td>
</tr>
<tr>
<td>4</td>
<td>168</td>
<td>196</td>
<td>266</td>
<td>W14x82</td>
<td>620</td>
</tr>
<tr>
<td>5</td>
<td>42</td>
<td>98</td>
<td>42</td>
<td>W14x82</td>
<td>620</td>
</tr>
</tbody>
</table>

**Step 4 – Beam Design**

The maximum required strength of the beam are the shear, moment, and axial forces determined from load combination 5 of ASCE 7 section 12.4.3.2 given as 

\[(1.2 + 0.2SDS)D + \Omega_0 Q_E + L + 0.2S\]

Taking the uniformly distributed load along the beam attributed to dead, live and snow loads 

\[(1.2 + 0.2SDS)D + L + 0.2S\]

as \(w_u = 1.5\) klf, the required shear and moment strength of the beams are at the floor levels are determined as follows:

\[V_{ub} = (1.5 \text{ klf})(17.5 \text{ ft})/2 = 13 \text{ kips}\]

\[M_{ub} = (1.5 \text{ klf})(17.5 \text{ ft})^2/8 = 57 \text{ k-ft}\]

Investigating the beam at level 5, W21x44 with flanges meeting the requirements for moderately ductile numbers is selected preliminarily based on strength and stiffness requirements from gravity loading. The force delivered to the frame by the diaphragm at level 5 is assumed to be 67 kips, and is conservatively assumed to be applied entirely through a collector connection at one end of the beam. Thus the overstrength axial load to be used for design of the beam and beam connection, as well as collector beams and collector connections outside braced frame, is determined as:

\[P_{ub5} = \Omega_0 Q_E = 2.5(67 \text{ kips}) = 168 \text{ kips}\]

Investigating the beam at level 2, the capacity-limited maximum axial force in the inverted V configuration is the horizontal component delivered by the first story braces, which is determined as:

\[P_{ub2} = P_{pr,max1}(L/L_{br}) = (384 \text{ kips})(8.75 \text{ ft})/(19.6 \text{ ft}) = 171 \text{ kips (governs over level 5)}\]

In accordance with Table 1-3 of the AISC Seismic Design Manual, 2nd Edition, the axial force limit for the web of a W21x44 to meet the requirements for moderately ductile members is \(P_{u,max} = 201 \text{ kips}\). This is greater than maximum applied axial force of 171 kips, therefore the web of the section meets the requirement for moderately ductile members.

The slenderness ratio of the beam about the strong axis is determined as:

\[KL/r_x = 1.0(17.5 \text{ ft})(12\text{ in./ft})/8.06 \text{ in.} = 26.1\]

Table 1-3 of the AISC Seismic Design Manual, 2nd Edition provides the maximum unbraced length to meet moderately ductile requirements, \(L_{br,max} = 10.4 \text{ ft}\). Therefore, the beam must be braced at the midpoint, which would be inherent in this case because bracing is present at the brace intersection. The resulting unbraced length is 8.75 ft. The slenderness ratio of the beam about the weak axis is determined as:

\[KL/r_y = 1.0(8.75 \text{ ft})(12\text{ in./ft})/1.26 \text{ in.} = 83.3 \text{ (governs)}\]

Utilizing Table 4-22 of the AISC Steel Construction Manual, 14th Edition, the compressive strength about the weak axis is determined as:
\[ \phi F_{crx} \geq 26.9 \text{ ksi}, \text{ therefore } F_{crx} = (26.9 \text{ ksi})/0.9 = 29.9 \text{ ksi} \]

Checking the web for slenderness in accordance with the Specification as:

\[ \frac{h}{t_w} = \frac{(20.7 \text{ in.} - 2(0.45 \text{ in.}))}{(0.35 \text{ in.})} = 56.6 \geq 1.49\sqrt{\left(\frac{29,000 \text{ ksi}}{29.9 \text{ ksi}}\right)} = 1.49(31.1) = 46.4 \]

\[ b_x = 1.92(0.35 \text{ in.})(31.1)(1-(0.34)(31.1)/(56.6)) = 17.0 \text{ in.} \]

\[ A_x = 13.0 \text{ in.}^2 - (0.35 \text{ in.})(20.7 \text{ in.} - 17.0 \text{ in.}) = 11.7 \text{ in.}^2 \]

\[ Q_x = (11.7 \text{ in.}^2)/(13.0 \text{ in.}^2) = 0.90 \]

\[ 4.71\sqrt{(29,000 \text{ ksi})(0.9(50 \text{ ksi}))} = 119.6 > 83.3 \]

\[ F_e = \pi^2(29,000 \text{ ksi})/(83.3)^2 = 41.2 \text{ ksi} \]

\[ \phi P_{nx} \geq \phi F_{cr} A_g = 0.9(0.9)(0.658 \frac{0.9(50 \text{ ksi})}{41.2 \text{ ksi}})(50 \text{ ksi})(13.0 \text{ in.}^2) = 333 \text{ kips} \]

From Table 3-10 of the AISC Steel Construction Manual, 14th Edition, the moment capacity of the beam with an unbraced length of 8.75 ft is \( \phi M_n = 290 \text{ k-ft} \).

The compression and moment interaction is determined as:

\[ P \chi \phi P_n = (171 \text{ kips})/(333 \text{ kips}) = 0.51 \]

\[ P \chi \phi P_n + 8M_u/(9\phi M_u) = 0.51 + 8(57 \text{ k-ft})/(9(290 \text{ k-ft})) = 0.68 \leq 1.0, \text{ therefore OK} \]

The shear strength is determined as:

\[ \phi V_a = \phi(0.6)F_{cr} t_w d_c = 1.0(0.6)(50 \text{ ksi})(0.35 \text{ in.})(20.7 \text{ in.})(1.0) = 217 \geq 13 \text{ kips}, \text{ therefore OK} \]

Because the governing forces for the beam at level 2 were considered in the design, the W21x44 may conservatively be used at all levels.

**Step 5 – Connection Design**

The required strength of the brace-to-gusset connection is the adjusted fuse strength, which can be calculated based on the amplified brace deformation, \( \Delta_{bl} \), associated with the design story drift determined in accordance with ASCE 7. Alternatively, the adjusted fuse strength can conservatively be taken as \( P_{pr,max} \). Both approaches were illustrated in Step 2 – Column Design. The maximum required axial force in the beam delivered through the end connection and the required shear in the beam were determined in Step 4 – Beam Design. With the axial forces in the brace and beam and the shear force in the beam identified, the detailing parameters for the connection associated with the given bay configuration can be determined using the Re-Fuse Braced Frame Connection Design Aid of Appendix D. Alternatively, the connections can be designed for these forces using conventional connection design approaches.

**Step 6 –Fuse Deformation Capacity**

The service level uniform load to the floor beams, \( w_a \), is assumed to be 1.0 klf under the earthquake load combinations. The deflection at middle of the beam, \( \Delta_{b,mid} \), is determined as:

\[ \Delta_{b,mid} = 5w_a L^4/(384EI_x) = 5(1.0 \text{ klf})(17.5 \text{ ft})^4(1728)/(384(29,000 \text{ ksi})(843 \text{ in.}^4)) = 0.09 \text{ in.} \]
The axial deformation of the brace assemblies, $\Delta_{br,beam}$, due to the deflection of the beam once the assembly enters into the inelastic range is determined as:

$$
\Delta_{br,beam} = L_{br} - \sqrt{(H - \Delta_{b,mid})^2 + L^2} = 234.79 \text{ in.} - \sqrt{(210 \text{ in.} - 0.09 \text{ in.})^2 + (105 \text{ in.})^2} = 0.08 \text{ in.}
$$

As calculated in Step 3, the approximate deformation of the brace assembly at the design story drift, $\Delta_{bm}$, is 1.08 in. The maximum deformation demand of an individual fuse, $\Delta_{fuse}$, is determined as:

$$
\Delta_{fuse} = (\Delta_{bm} + \Delta_{br,beam})/n_{fuse} = (1.08 \text{ in.} + 0.09 \text{ in.})/2 = 0.59 \text{ in.} \leq \Delta_{max,fuse} = 0.75 \text{ in.}, \text{ therefore OK}
$$

Because $P_{pr,max}$ was used for the design of columns and connections the designs need not be re-checked for the adjusted brace assembly strength associated with the maximum fuse deformation.

**Step 7 – Story Drift Limit**

As indicated in Step 3 – Column Design, typically the calculated stiffness of all the braces would be utilized within an analysis model that includes P-$\Delta$ effects to determine elastic story drifts, which would then be amplified in accordance with ASCE 7 to determine design story drifts. Recognizing the contribution of axial stiffness of the beams and columns to the inelastic drift of the frame is minimal, and that P-$\Delta$ effects are also relatively small for braced frame systems, these can be neglected for the illustrative purposes of this example. In doing so $\Delta_{bm}$ was approximated in Step 3 as the elastic deformation of the brace assembly multiplied by the deflection amplification factor, $C_d$, assuming the importance factor, $I_e$, is taken as 1.0. The story drift between level 3 and level 2, $\Delta_2$, may then be determined as:

$$
\Delta_2 = \sqrt{(L_{br} + \Delta_{bm})^2 - H^2} - \sqrt{L_{br}^2 - H^2} = \sqrt{(234.78 \text{ in.} + 1.08 \text{ in.})^2 - 210^2} - \sqrt{234.78^2 - 210^2}
$$

$$
\Delta_2 = 2.39 \text{ in.}
$$

Assuming a Risk Category II structure, the allowable story drift in accordance with Table 12.12-1 of ASCE 7 is 2% of the story height. The allowable story drift for each story is determined as:

$$
\Delta_{a2} = 0.02H = 0.02(17.5 \text{ ft})(12 \text{ in./ft}) = 4.2 \text{ in.} \geq \Delta_2 = 2.39 \text{ in.}, \text{ therefore OK}.
$$
Final Frame Design

The final frame design including member sizes, fuse types, maximum forces for brace connection and drag beam connection design, and references to the Braced Frame Connection Design Aid in Appendix D is shown Figure C-2 below.

Figure C-2. Final Frame Design
APPENDIX D – RE-FUSE BRACED FRAME CONNECTION DESIGN AID

This design aid provides pre-designed braced frame and drag beam collector connections for use only with the Re-Fuse Braced Frame System. The connection design information presented has been prepared in accordance with recognized engineering principals. The application of the information provided is at the sole discretion of the Structural Engineer of Record for a given project and is not the responsibility of Novel Structures, LLC or its representatives. Where connection design parameters herein cannot be adhered to, alternate connections shall be requested from the Structural Engineer of Record.

ABBREVIATIONS/SYMBOLS

BM = BEAM
CL = CENTER LINE
CLR = CLEAR
CNTR = CENTER
COL = COLUMN
CONN = CONNECTION
DTL = DETAIL
EA = EACH
EE = EACH END
ES = EACH SIDE
INFO = INFORMATION
OPP = OPPOSITE
PL = PLATE
RECD = REQUIRED
SP = SPACING
TYP = TYPICAL
WF = WIDE FLANGE
WP = WORK POINT
∪ = RE-FUSE CONNECTION
SEE DETAIL 1

EXAMPLE BRACED FRAME CONFIGURATION

SEE CONTRACT DOCUMENTS FOR:
- ACTUAL BRACED FRAME CONFIGURATIONS AND DIMENSIONS SPECIFIC TO PROJECT
- MEMBER SIZES
- FUSE CONFIGURATIONS/TYPES
- MAXIMUM FUSE FORCES FOR CONNECTION SELECTION
- COLLECTOR FORCES AT DRAG BEAMS
- CONNECTION REQUIREMENT OTHER THAN THOSE INDICATED HEREIN
<table>
<thead>
<tr>
<th>FUSE TYPE</th>
<th>MAX FUSE FORCE, $P_{ps max}$ (KIPS)</th>
<th>MIN BOLT ROWS, NRC, FOR BRACE ANGLE = 0</th>
</tr>
</thead>
<tbody>
<tr>
<td>RF-30</td>
<td>55</td>
<td>2</td>
</tr>
<tr>
<td>RF-60</td>
<td>110</td>
<td>3</td>
</tr>
<tr>
<td>RF-90</td>
<td>164</td>
<td>4</td>
</tr>
<tr>
<td>RF-120</td>
<td>219</td>
<td>5</td>
</tr>
<tr>
<td>RF-150</td>
<td>274</td>
<td>7</td>
</tr>
<tr>
<td>RF-180</td>
<td>329</td>
<td>8</td>
</tr>
<tr>
<td>RF-210</td>
<td>384</td>
<td>9</td>
</tr>
</tbody>
</table>

* See Beam Drag Connection Parameters Schedule for beam connection info

---

**BM-CCN CONN - BRACE ABOVE AND BELOW BM**

**TYP BM-CCN**

BRACED FRAME BM-CCN CONNECTION AT WF COL WEE
### Gusset-to-Base Plate Connection Parameters (LRFD)

<table>
<thead>
<tr>
<th>Fuse Type</th>
<th>Max Fuse Force, ( P_{pr,\text{max}} ) (KIPS)</th>
<th>Min Weld Length (IN.) for Brace Angle = ( \theta )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>20° ≤ ( \theta &lt; 35° )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>LC</td>
</tr>
<tr>
<td>RF-30</td>
<td>55</td>
<td>4</td>
</tr>
<tr>
<td>RF-60</td>
<td>110</td>
<td>8</td>
</tr>
<tr>
<td>RF-90</td>
<td>164</td>
<td>12</td>
</tr>
<tr>
<td>RF-120</td>
<td>219</td>
<td>15</td>
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<tr>
<td>RF-150</td>
<td>274</td>
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<tr>
<td>RF-180</td>
<td>329</td>
<td>23</td>
</tr>
<tr>
<td>RF-210</td>
<td>384</td>
<td>26</td>
</tr>
</tbody>
</table>

---

![Diagram](image_url)

**Braced Frame Col Base Conn at WF Column Web**

---

26
### V-Brace Gusset-to-Beam Connection Parameters (LRFD)

<table>
<thead>
<tr>
<th>Fuse Type</th>
<th>Max Fuse Force, $P_{fr, max}$ (Kips)</th>
<th>Min Weld Length (In.) for Brace Angle = ( \theta )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$20^\circ \leq \theta &lt; 35^\circ$</td>
</tr>
<tr>
<td>RF-30</td>
<td>55</td>
<td>4</td>
</tr>
<tr>
<td>RF-60</td>
<td>110</td>
<td>8</td>
</tr>
<tr>
<td>RF-90</td>
<td>164</td>
<td>12</td>
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<tr>
<td>RF-120</td>
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<td>15</td>
</tr>
<tr>
<td>RF-150</td>
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<td>19</td>
</tr>
<tr>
<td>RF-180</td>
<td>329</td>
<td>23</td>
</tr>
<tr>
<td>RF-210</td>
<td>384</td>
<td>26</td>
</tr>
</tbody>
</table>

![Diagram of Brace Intersection Conn on WF BM]

Mirror conn about horizontal at brace above BM condition. See DTL 1 for re-fuse conn info.
<table>
<thead>
<tr>
<th>BEAM SIZE</th>
<th>BOLT ROWS NRB</th>
<th>MAX BEAM REACTION, $V_u$ (KIPs)</th>
<th>MAX AXIAL FORCE, $A_u$ (KIPs), FOR NCB BOLT COLUMNS</th>
</tr>
</thead>
<tbody>
<tr>
<td>W8x13 &amp; Up</td>
<td>2</td>
<td>17</td>
<td>NCB = 3 20 20 20 NCB = 4 84 84 84 NCB = 5 116 116 116</td>
</tr>
<tr>
<td>W10x15 &amp; Up</td>
<td>2</td>
<td>17</td>
<td></td>
</tr>
<tr>
<td>W12 - W14</td>
<td>3</td>
<td>25</td>
<td></td>
</tr>
<tr>
<td>W16</td>
<td>4</td>
<td>43</td>
<td></td>
</tr>
<tr>
<td>W18</td>
<td>5</td>
<td>64</td>
<td></td>
</tr>
<tr>
<td>W21 - W24</td>
<td>6</td>
<td>88</td>
<td></td>
</tr>
<tr>
<td>W27</td>
<td>7</td>
<td>115</td>
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</tr>
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<td>W30</td>
<td>8</td>
<td>142</td>
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</tr>
<tr>
<td>W33</td>
<td>9</td>
<td>170</td>
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</tr>
<tr>
<td>W36 - W44</td>
<td>10</td>
<td>200</td>
<td></td>
</tr>
</tbody>
</table>

**BEAM DRAG CONNECTION PARAMETERS (LRFD)**

**DRAG CONN AT WF COL WEB**

**BOLTED-BOLTED ANGLE ALTERNATE CONN**

28