Reduced Beam Section Connection (SMRF)
Based on FEMA 350 (July 2000)
Serial #: 12345

Sample Calculation

**Beam Properties of:** W 16x57
- **d:** 16.4 in.
- **b:** 7.12 in.
- **w:** 57 lb.
- **Sx:** 92.2 in³
- **Ix:** 758 in⁴
- **Zx:** 105 in³
- **Rx:** 6.72 in.
- **Tf:** 0.715 in.
- **Sy:** 12.1 in³
- **Ly:** 43.1 in⁴
- **Zy:** 18.9 in³
- **Ry:** 1.6 in.

**Column Properties of:** W 14x53
- **d:** 13.9 in.
- **b:** 8.06 in.
- **w:** 53 lb.
- **Sx:** 77.8 in³
- **Ix:** 541 in⁴
- **Zx:** 87.1 in³
- **Rx:** 5.89 in.
- **Tf:** 0.66 in.
- **Sy:** 14.3 in³
- **Ly:** 57.7 in⁴
- **Zy:** 22 in³
- **Ry:** 1.92 in.

**Steel Properties:**
- **Steel Grade:** A992
- **Fy:** 50 ksi
- **Fu:** 65 ksi
- **Cpr:** 1.15
- **Ry:** 1.1

**Frame Dimensions:**
- **Beam Length (Column C/C):** 20.00 ft.
- **Avg. Floor Height:** 12.00 ft.

**RBS Geometry of the Beam:**
- **a:** Beam Flange x 0.60 = 4.25 in. From 4.272 in.
- **b:** Beam Depth x 0.75 = 12.25 in. From 12.300 in.
- **c:** Beam Flange x 0.20 = 1.50 in. From 1.424 in.
- **Cutout Radius:** 13.255 in.
- **X (Col. Face to RBS Dimension):** 10.38 in.
- **L' (RBS-RBS Dimension):** 17.11 ft.
- **RBS Section Modulus:** 59.22 in³
- **RBS Plastic Modulus:** 71.36 in³

**Beam and column parameters**
- **Beam depth less than 36 inches:** 16.4 in. OK
- **Beam weight less than 300 pounds:** 57 pounds OK
- **Beam's span to depth ratio greater than 7:** 13.79 OK
- **Beam's flange less than 1-3/4 inches thick:** 0.715 OK
- **Mom. capacity of BM's flange less than 0.7xMplastic:** 0.76xMplastic OK
- **Flange reduction less than 50% of flange width:** 57.9% remaining OK
- **Column's size W12x or W14x:** W 14x53 OK
- **Column width less than beam width:** 8.06 in. vs. 7.12 in. OK

**Code Checks**

**Calculated Values**
- **Vg:** 25.71 kip
  - Shear at the column face from factored gravity loads (Occurs at the Right side)
- **Vf:** 71.87 kip
  - Shear at the column face
- **Vp:** 68.02 kip
  - Shear at the RBS (Occurs at the Right side)
- **Mf:** 434.9 ft-kip
  - Probable plastic moment at the face of the column
- **Mc:** 474.3 ft-kip
  - Probable plastic moment at the center of the column
- **Mpr:** 376.1 ft-kip
  - Probable peak plastic hinge moment at RBS
- **ds:** 7.58%
  - Frame's drift increase factor due to RBS
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<table>
<thead>
<tr>
<th><strong>Mf</strong></th>
<th>434.9 ft-kip</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_f$</td>
<td>481.25 ft-kip</td>
</tr>
<tr>
<td>Ratio</td>
<td>0.904 OK</td>
</tr>
</tbody>
</table>

#### Doubler Plates

<table>
<thead>
<tr>
<th>Srbs</th>
<th>59.22 in³</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Cy$</td>
<td>0.72</td>
</tr>
<tr>
<td>$Ry$</td>
<td>1.1</td>
</tr>
<tr>
<td>$t$</td>
<td>0.561 in</td>
</tr>
</tbody>
</table>

FAIL - Doubler plates required

#### Continuity Plates

<table>
<thead>
<tr>
<th>Tcf1</th>
<th>1.211 in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tcf2</td>
<td>1.187 in</td>
</tr>
<tr>
<td>Tcf</td>
<td>1.187 in</td>
</tr>
</tbody>
</table>

FAIL - Continuity plates required

#### Beam Flange

<table>
<thead>
<tr>
<th>$bf/2tf$</th>
<th>3.18</th>
</tr>
</thead>
<tbody>
<tr>
<td>$52/sqrt(Fy)$</td>
<td>7.35</td>
</tr>
<tr>
<td>Ratio</td>
<td>0.43 OK</td>
</tr>
</tbody>
</table>

#### Beam Web

<table>
<thead>
<tr>
<th>$hc/tw$</th>
<th>34.81</th>
</tr>
</thead>
<tbody>
<tr>
<td>$418/sq(fy)$</td>
<td>59.11</td>
</tr>
<tr>
<td>Ratio</td>
<td>0.59 OK</td>
</tr>
</tbody>
</table>

#### Shear capacity of the beam

<table>
<thead>
<tr>
<th>$V_f$</th>
<th>71.87 kip</th>
</tr>
</thead>
<tbody>
<tr>
<td>Allow. Shear</td>
<td>190.40 kip</td>
</tr>
<tr>
<td>Unity Check</td>
<td>0.38 OK</td>
</tr>
</tbody>
</table>

#### Moment capacity of the beam

| Allow. Moment | 393.75 ft-kip |

Actual moment to be less than this amount. Check with your frame analysis software.

#### Moment capacity of the beam at RBS

| Allow. Moment | 267.60 ft-kip |

Actual moment at RBS to be less than this amount. Check with your frame analysis software.
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Sample Calculation

Gravity Loads at Beam

Distributed Loads

<table>
<thead>
<tr>
<th>Dead Load</th>
<th>Live Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.500 kip/ft</td>
<td>1.000 kip/ft</td>
</tr>
<tr>
<td>0.000 kip/ft</td>
<td>0.000 kip/ft</td>
</tr>
<tr>
<td>0.000 kip/ft</td>
<td>0.000 kip/ft</td>
</tr>
</tbody>
</table>

Point Loads

<table>
<thead>
<tr>
<th>Dead Load</th>
<th>Live Load</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.200 kip</td>
<td>2.300 kip</td>
<td>13.00 ft</td>
</tr>
<tr>
<td>1.500 kip</td>
<td>2.100 kip</td>
<td>16.00 ft</td>
</tr>
<tr>
<td>0.000 kip</td>
<td>0.000 kip</td>
<td>0.00 ft</td>
</tr>
</tbody>
</table>

Notes and Assumptions

1- Flexural demand on the girder due to gravity loads is less than about 30% of the girder’s capacity.
2- Strong Column - Weak Beam action is not checked.
3- For bracing and other requirements see FEMA 350.