Design Example of Moment Resisting Seismic Frames with Reduced Beam Sections

Author: Kevin Cowie
Affiliation: Steel Construction New Zealand Inc.
Date: 9th July 2010
Ref.: EQK1004

Key Words
Reduced Beam Section, RBS, Moment Connection, Moment Resisting Frame, Seismic Frame,

Introduction
The design of moment resisting seismic frames can by optimised with the use of reduced beam sections. In a reduced beam section (RBS) moment connection (figure 1), portions of the beam flanges are selectively trimmed in the region adjacent to the beam to column connection. Yielding and hinge formation are intended to occur primarily within the reduced section of the beam and thereby limit the design actions and the inelastic deformation demands developed at the face of the column.

The development, research, design rules, design consideration and benefits of the RBS are covered in previous Steel Advisor articles EQK1002 and EQK1003. This article illustrates the application of the RBS design rules by way of a design example.

Design Example Building Description
The design example is for a 5 level building. See figure 2 and 3. The building has a rectangular floor area of 35 metres by 21 metres. There is a 3 bay moment resisting frame on each perimeter wall. The interstorey heights are 3.5 metres.

Disclaimer: SCNZ and the author(s) of this document make no warrantee, guarantee or representation in connection with this document and shall not be held liable or responsible for any loss or damage resulting from the use of this document.
Frame Design
The moment resisting frame has been designed to the earthquake loadings standard NZS1170.5. (SNZ, 2004) The moment resisting frame has been designed as a Category 2 (limited ductile) frame for a structural ductility factor ($\mu_{\text{design}}$) of 3.

To account for composite beam action and the reduction in frame stiffness due to the reduced beam section the beam gross section properties were modified in accordance with the provisions of NZS 3404 appendix N (SNZ, 2007) and the RBS design rules.

The following beam and column sizes satisfy the strength and drift limitations.

<table>
<thead>
<tr>
<th>Level</th>
<th>Beam</th>
<th>Column</th>
<th>$M_{\text{RBS}}$ at Centre of RBS (kNm)</th>
<th>$0.7 \phi M_{p}$ (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L5</td>
<td>530UB82</td>
<td>610UB101</td>
<td>78</td>
<td>391</td>
</tr>
<tr>
<td>L4</td>
<td>530UB82</td>
<td>610UB101</td>
<td>221</td>
<td>391</td>
</tr>
<tr>
<td>L3</td>
<td>610UB101</td>
<td>610UB101</td>
<td>370</td>
<td>547</td>
</tr>
<tr>
<td>L2</td>
<td>610UB101</td>
<td>610UB125</td>
<td>477</td>
<td>547</td>
</tr>
<tr>
<td>L1</td>
<td>610UB101</td>
<td>610UB125</td>
<td>541</td>
<td>547</td>
</tr>
</tbody>
</table>

**Note:** The centre of the RBS is at the minimum distance from the column face.

Reduced Beam Section Connection Design

Introduction
This example considers the design of the RBS at the first storey level as shown figure 4. At the first storey level the section properties for the beams and columns are as follows:

Beam 610UB101
\[
d = 602\text{mm}, b_{y} = 228\text{mm}, t_{f} = 14.8\text{mm}, t_{w} = 10.6\text{mm}, S_{x} = 2900 \times 10^{3} \text{ mm}^{3},
\]

Column 610UB125
\[
d_{c} = 612\text{mm}, b_{c} = 229\text{mm}, t_{c} = 19.6\text{mm}, t_{wc} = 11.9\text{mm},
\]
The dead and live gravity actions are modelled as uniform area loads. The equivalent line loads on the beam is 20 kN/m.

**Step 1: Choose plastic hinge configuration and location**

Limitations for the dimensions of the radius cut for the reduced beam sections are as follows:

- \(0.5b_f \leq a \leq 0.75b_f\)
- \(0.65d \leq b \leq 0.85d\)
- \(0.1b_f \leq c \leq 0.25b_f\)

- \(114 \text{ mm} \leq a \leq 171 \text{ mm}\)
- \(391 \text{ mm} \leq b \leq 512 \text{ mm}\)
- \(23 \text{ mm} \leq c \leq 57 \text{ mm}\)

The minimum amplification of the overstrength moment of the reduced beam section occurs when the centre of RBS is kept at the minimum distance from the face of the column. Therefore select dimensions for ‘a’ and ‘b’ near the minimum limits. To ensure that the capacity designed moment at the column face is less than the design moment of the full beam section the dimension ‘c’ should be selected near the maximum limit.

Try
- \(a = 120 \text{ mm}\)
- \(b = 400 \text{ mm}\)
- \(c = 55 \text{ mm}\)

**Step 2: Determine plastic section modulus at the reduced beam section**

The plastic section modulus at the centre of the reduced beam section is a function of the initial plastic section modulus of the beam minus the proportion removed and is calculated as follows:

\[
S_{RBS} = S - 2ct_f - \frac{t_f}{t_c}
\]

- \(S_{RBS} = 2900 - 2 \times 55 \times 14.8 \times (02 - 14.8)\)
- \(S_{RBS} = 1944 \times 10^3 \text{ mm}^3\)

**Step 3: Determine design moment capacity at the reduced beam section**

The design moment capacity at the centre of the reduced beam section is calculated as:

\[
\phi M_{x,RBS} = \phi f_y Z_{c,RBS}
\]

- \(\phi M_{x,RBS} = 0.9 \times 300 \times 1944\)
- \(\phi M_{x,RBS} = 525 \text{ kNm}\)
- \(\phi M_{x,RBS} = M_{RBS} \cdot \text{OK!}\)

The reduced beam section design moment capacity is greater than the design moment.
**Step 4: Determine overstrength moment at the reduced beam section**
The column is isolated from the slab. Therefore the slab participation factor equals 1.0. The overstrength moment capacity at the centre of the reduced beam section is calculated as:

\[
\phi_{oms} M_{sx,RBS} = \phi_{oms} f_y Z_{e,RBS}
\]

\[
\phi_{oms} M_{sx,RBS} = 1.15 \times 300 \times 1944
\]

\[
\phi_{oms} M_{sx,RBS} = 671 \text{ kNm}
\]

**Step 5: Determine shear force at centre of reduced beam section**
The shear force is calculated at the centre of the reduced beam sections located near each end of the beam. The shear force is determined by a free body diagram of the portion of the beam between the centres of the reduced beam sections.

\[
V^c_{RBS} = 2\phi_{oms} M_{sx,RBS} + \frac{w^* L'}{2}
\]

\[
L' = L - d_c - 2a - b
\]

\[
L' = 7 - 0.612 - 2 \times 0.12 - 0.4 = 5.75 \text{m}
\]

\[
V^c_{RBS} = \frac{2 \times 671 + 20 \times 5.75}{5.75} \frac{2}{2}
\]

\[
V^c_{RBS} = 291 \text{ kN}
\]

**Step 6: Determine maximum bending moment at the face of the column**
The moment at the face of the column is determined by using a free body diagram between centre of the RBS and the face of the column and is calculated as follows:

\[
M^c_f = \phi_{oms} M_{sx,RBS} + V^c_{RBS} (a + \frac{b}{2}) + \frac{w^*}{2} \left(a + \frac{b}{2}\right)^2
\]
Step 7: Check that $M_f$ does not exceed $\Phi M_{sx}$

Check the moment at the face of the column is less than the design moment capacity of the beam.

$$M_f = 765 \text{kNm}$$

$$\phi M_s = 783 \text{kNm}$$

Is $M_f \leq \phi M_s$? Yes!

Step 8: Determine the required shear strength

The shear at the face of the column is determined by using a free body diagram between centre of the RBS and the face of the column and is calculated as follows:

$$V_f^C = \frac{2\phi_{\text{om}} M_{0x,\text{RBS}}}{L} + V_{Q,G,\text{j}}$$

$$V_f^C = \frac{2 \times 765}{5.75} + \frac{20 \times 7}{2} = 303 \text{kN}$$

In a plastic hinge zone $\Phi V_w$ is reduced to 0.8 $\Phi V_w$ (cl 12.10.0.1 NZS 3404). The design shear capacity of the beam is:

$$\Phi V_w = 0.8 \times 0.6 \times 320 \times 602 \times 10.6 / 1000 = 880 \text{kN}$$

Step 9: Beam to Column Connection

No inelastic is anticipated at the beam-to-column connection. Therefore standard pre-engineered connections can be used. Standard SCNZ Connections are selected. These could be WM Elastic or MEP 100/50 Elastic.

Step 10: Continuity Stiffeners Requirements

Select continuity stiffeners to have equivalent area to beam flanges.

$$A_s \geq \frac{f_{yb}}{t_{yd} t_{db}} \left( \frac{f_{yb}}{f_{ys}} \right)$$

$$A_s \geq \frac{374 - 16.9 \times 14.8}{250} \left( \frac{300}{250} \right) = 3705 \text{ mm}^2$$

Choose:

100 x 20 flats, area = 4000 mm$^2$

Step 11: Check Column Panel Zone

The concrete will be isolated from the column. Therefore there will be no slab participation.

Inner Column

Panel Zone Horizontal Shear actions:

$$V_p = \phi M_{0x,L} \left( \frac{d-t_r}{d} \right)_L + \phi M_{0x,R} \left( \frac{d-t_r}{d} \right)_R - V_{Col}$$

Using approach in NZS3404 Commentary CI C12.9.5.2, if points of contraflexure in the columns above and below the connection occurs at mid-height then

$$V_{Col} = \frac{\phi M_{0x,L} + \phi M_{0x,R}}{L_c}$$

$$V_{Col} = \frac{783 + 783}{3.5 - 0.602} = 540 \text{kN}$$
\[ V_p^* = \frac{783}{(0.602 - 0.0148)_L} + \frac{783}{(0.602 - 0.0148)_R} - 540 = 2127 \text{kN} \]

Check panel zone shear capacity without doubler plates

\[ \phi V_c = 0.6 \phi f_{yp} d_c (w_c + t_p + \frac{3b_t t_c^2}{d_b d_t (w_c + t_p)}) \]

\[ \phi V_c = 0.6 \times 0.9 \times 300 \times 1000 \times 0.612 \times 0.0119 \times 1 \times \left[ 1 + \frac{3 \times 0.229 \times 0.0196^2}{0.602 \times 0.612 \times 0.0119} \right] \text{Assumed } \eta = 1 \]

\[ \phi V_c = 1180 \times \frac{0.06}{1251} = 1251 \text{kN} \]

Are doubler plates required? Yes
If so what doubler plate(s) thickness is required?

Try 10mm thick doubler plate on one side

\[ \phi V_c = 0.6 \phi f_{yp} d_c (w_c + t_p + \frac{3b_t t_c^2}{d_b d_t (w_c + t_p)}) \]

\[ f_{yp} = \frac{t_{wc} f_{yc} + t_p f_{yp}}{t_{wc} + t_p} \]

\[ f_{yp} = \frac{11.9 \times 300 + 10 \times 260}{11.9 + 10} = 282 \text{MPa} \]

\[ \phi V_c = 0.6 \times 0.9 \times 282 \times 1000 \times 0.612 \times (0.0119 + 0.01) \times \left[ 1 + \frac{3 \times 0.229 \times 0.0196^2}{0.602 \times 0.612 \times (0.0119 + 0.01)} \right] \]

\[ \phi V_c = 2039 \times \frac{0.03}{1251} = 2100 \text{kN} \]

1 % difference OK.

**Step 12: Check weak beam – strong column criteria**

Determine capacity designed derived beam bending moment at the column centreline

\[ M_{beam}^c = \phi M_{sx} + V_f^c \frac{d_c}{2} \]

Determine the overstrength factor at each beam-column joint

\[ \phi = \frac{\sum M_{beam}^c}{\sum |M_{beam,E}|} \]

Where:

\[ \sum |M_{beam}| = \text{the sum of the absolute values of the capacity design derived moments at the column centreline for the beams framing into the joint.} \]

\[ \sum |M_{beam,E}| = \text{the sum of the absolute values of the beam moments at the column centreline for load case E.} \]

Column bending moments are scaled up by the overstrength factor \( \phi^* \) calculated at each beam-column joints. The column design bending moment \( M_{col}^* \) is obtained at the beam faces.

The column design axial forces on the inner columns are determined using

\[ N_{n_n,col} = M_{col}^* Q & Qu + \sum_{i=1}^{n} V_i^c \leq N_{col}(Q & U)^{\text{max}} \]

The column is checked for the combination of moment and axial actions.

If an RBS is not used the overstrength factor for the joint will be significantly higher and therefore the column bending moments will have to be scaled up to by a higher amount. In this particular case the column strength will not be adequate and so a larger size column will be required.
References

Cowie, K., Research, Development and Design Rules of Moment Resisting Seismic Frames with Reduced Beam Sections, Steel Advisor EQK1002, Steel Construction New Zealand Inc., Manukau City, 2010

Cowie, K., Design Considerations and Benefits of Moment Resisting Seismic Frames with Reduced Beam Sections, Steel Advisor EQK1003, Steel Construction New Zealand Inc., Manukau City, 2010

SNZ, Structural Design Actions, NZS 1170.5, Standards New Zealand, Wellington, 2004