Design Example of Moment Resisting Seismic Frames with Reduced Beam Sections

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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 there by 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.

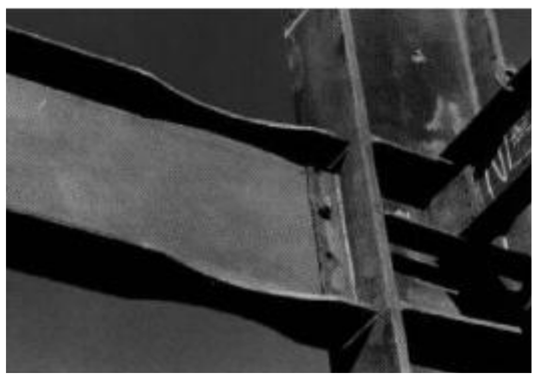


Figure 1: Reduced Beam Section Moment Connection (Englehardt et al, 1996)

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.

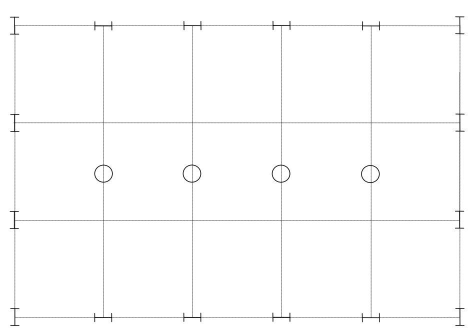


Figure 2: Design Example Plan

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 (μ_{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.

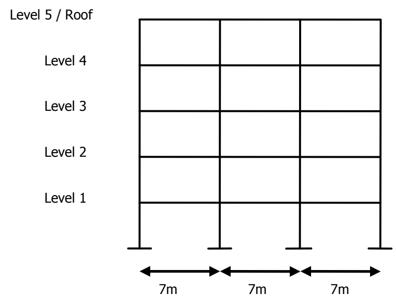


Figure 3: Design Example Elevation

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

Table 1: MRF Beam & Column Sizes

Level	Beam	Column	M_{RBS}^{*} at Centre[1] of RBS kNm	$0.7\ \Phi M_s$ kNm
L5	530UB82	610UB101	78	391
L4	530UB82	610UB101	221	391
L3	610UB101	610UB125	370	547
L2	610UB101	610UB125	477	547
L1	610UB101	610UB125	541	547

Note: 1) The centre of the RBS is at the minimum distance from the column face.

At the preliminary design stage the strength check is made at the centre of the reduced beam section and not at the column face. The minimum derived capacity design derived column face moments is kept at a minimum when the centre of the RBS from the face of the column is at the minimum. The capacity design procedure is based on displacing the seismic-resisting system laterally so that the yielding regions form in the RBS zone of all beams to form a yielding mechanism. In practice this state is typically reached only once in a severe earthquake and for medium rise or taller structures it may not be reached at all. Typically, what happens in taller structures is that bands of inelastic demand migrate up and down the seismic-resisting system affecting a subset of the total number of levels at a time. When determining the earthquake design moments on the beams the design actions from several levels can be summed and averaged over the levels under consideration, without reducing the strength of the overall mechanism. Therefore the average moment at the RBS over the bottom two storeys is 509 kNm.

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
$d =$ 602mm, b_f = 228mm, t_f = 14.8mm, t_w = 10.6mm, $S_x = 2900 \times 10^3\ mm^3$,

Column 610UB125
$d_c =$ 612mm, b_{fc} = 229mm, t_{fc} = 19.6mm, t_{wc} = 11.9mm,

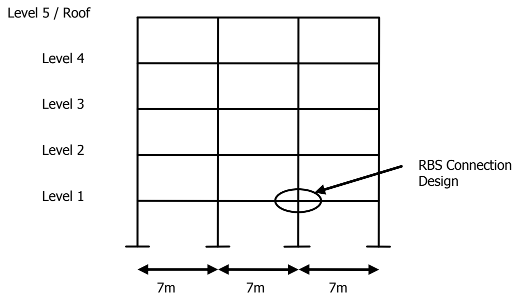


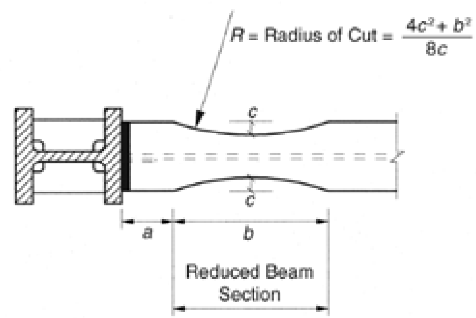
Figure 4: Location of RBS Connection Design

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:

$0.5b_f \le a \le 0.75b_f$
$0.65d \le b \le 0.85d$
$0.1b_f \le c \le 0.25b_f$

$114 \text{ mm} \le a \le 171 \text{ mm}$
$391 \text{ mm} \le b \le 512 \text{ mm}$
$23 \text{ mm} \le c \le 57 \text{ mm}$



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 mm
b =400 mm
c =55 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:

$$S_{RBS} = S - 2ct_f(d - t_f)$$
$$S_{RBS} = 2900 - 2 \times 55 \times 14.8 \times (602 - 14.8)$$
$$S_{RBS} = 1944 \times 10^3 \text{ mm}^3$$

Step 3: Determine design moment capacity at the reduced beam section
The design moment capacity at the centre off the reduced beam section is calculated as:

$$\phi M_{sx,RBS} = \phi f_y Z_{e,RBS}$$
$$\phi M_{sx,RBS} = 0.9 \times 300 \times 1944$$
$$\phi M_{sx,RBS} = 525 \text{ kNm}$$
$$\phi M_{sx,RBS} \ge M^*_{RBS} \therefore \text{OK!}$$

The reduced beam section design moment capacity is greater than the design moment.

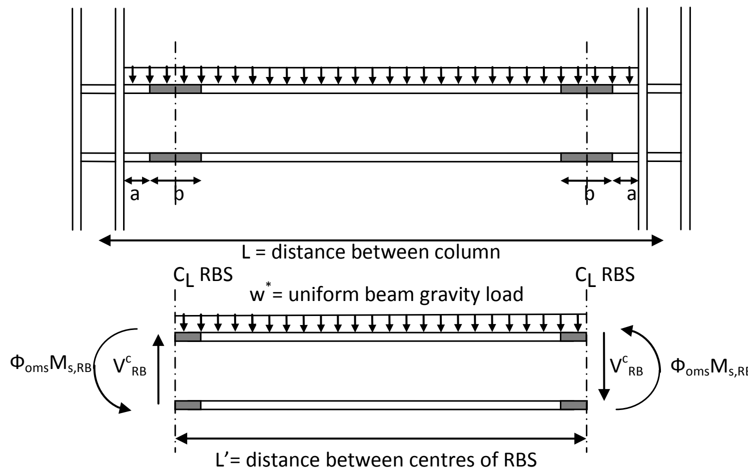
Step 4: Determine overstrength moment at the reduced beam section
The column is isolated from the slab. Therefore the slab participation factor equals 1.0. The overstrength moment capacity at the centre off the reduced beam section is calculated as:

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

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

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

Step 5: Determine shear force at centre of reduce 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.



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

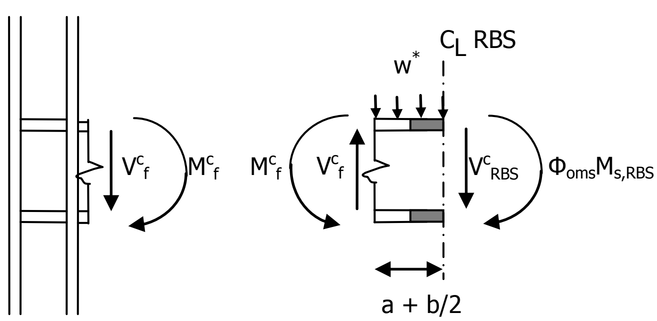
$$L' = L - d_c - 2a - b$$

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

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

$$V^c_{RBS} = 291\text{kN}$$

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:



$$M^c_f = \phi_{oms}M_{sx,RBS} + V^c_{RBS}(a + \frac{b}{2}) + \frac{w^*}{2}(a + \frac{b}{2})^2$$

$$M_f^c = 671 + 291(0.12 + \frac{0.4}{2}) + \frac{20}{2}(0.12 + \frac{0.4}{2})^2$$

$$M_f^c = 671 + 93 + 1 = 765\,\text{kNm}$$

Step 7: Check that M^c_f does not exceed ΦMsx

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

$$M_f^c = 765\,\text{kNm}$$

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

Is $M_f^* \le \phi M_s$? 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:

$$V_f^c = \frac{2\phi_{oms} M_{sx,RBS}}{L'} + V_{G\&Q_u}$$

$$V_f^c = \frac{2 \times 671}{5.75} + \frac{20 \times 7}{2}$$

$$V_f^c = 303\,\text{kN}$$

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

$$\phi V_v = 0.8\phi 0.6 f_y A_w$$

$$\phi V_v = 0.8 \times \phi \times 0.6 \times 320 \times 602 \times 10.6 / 1000 = 880\,\text{kN}$$

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.

$$A_s \ge \left(A_{fb} - t_{wc} t_{fb}\right)\left(\frac{f_{yb}}{f_{ys}}\right)$$

$$A_s \ge \left(3374 - 19.6 \times 14.8\right)\left(\frac{300}{250}\right) = 3705\ \text{mm}^2$$

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:

$$V_p^* = \frac{\phi M_{sx,L}}{(d - t_f)_L} + \frac{\phi M_{sx,R}}{(d - t_f)_R} - V_{Col}$$

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

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

$$V_{Col} = \frac{783 + 783}{(3.5 - 0.602)} = 540\,\text{kN}$$

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

Check panel zone shear capacity without doubler plates

$$\phi V_c = 0.6\phi f_{yp}^* d_c (t_{wc} + t_p)\eta \left[1 + \frac{3 b_c t_{fc}^2}{d_b d_c (t_{wc} + t_p)}\right]$$

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

$$\phi V_c = 1180 \times [1.06] = 1251\text{kN}$$

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

Try 10mm thick doubler plate on one side

$$\phi V_c = 0.6\phi f_{yp}^* d_c (t_{wc} + t_p)\eta \left[1 + \frac{3 b_c t_{fc}^2}{d_b d_c (t_{wc} + t_p)}\right]$$

$$f_{yp}^* = \frac{t_{wc} f_{yc} + t_p f_{yp}}{t_{wc} + t_p}$$

$$f_{yp}^* = \frac{11.9 \times 300 + 10 \times 260}{11.9 + 10} = 282\text{MPa}$$

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

$$\phi V_c = 2039 \times [1.03] = 2100\text{kN}$$

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

$$\phi^\circ = \frac{\sum \left|M_{beam}^c\right|}{\sum \left|M_{beam,E}\right|}$$

Where:

$\sum \left|M_{beam}^c\right|$ = the sum of the absolute values of the capacity design derived moments at the column centreline for the beams framing into the joint.

$\sum \left|M_{beam,E}\right|$ = 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 ϕ° 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

$$N_{inn,col} = N_{col,Q\&Qu} + \sum_i^n V_f^c \le N_{col,GQuEmax}$$

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.

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