Design Considerations and Benefits 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 and design rules of this type of connection are discussed in a previous Steel Advisor article EQK1002. This article highlights some of the design considerations and benefits of using reduced beam sections with moment resisting seismic frames.

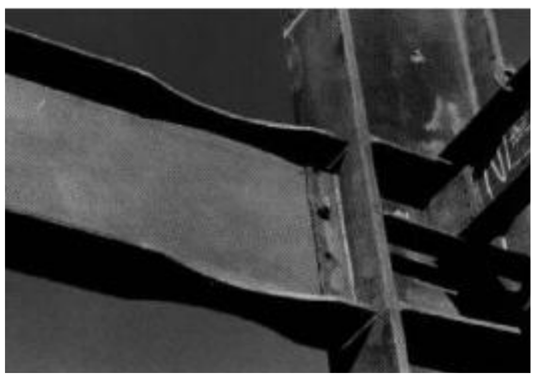

Figure 1: Reduced Beam Section Moment Connection (Englehardt et al, 1996)

Design Considerations

Moment Frame Beam Span Ratio

Moments resisting seismic frames with reduced beam sections are designed to limit the maximum moment that can be developed at the column face to less than the design moment capacity of the full beam section. The maximum column face design action that can be developed during an earthquake event is an amplification of the overstrength moment capacity of the reduced beam section, refer to figure 2. As the moment frame span decreases the seismic moment gradient increases thereby increasing the capacity design derived column face moments. Thus it becomes increasingly difficult to ensure that moment at the face of the column is less than the design moment capacity of the full beam section for low span to height ratios.

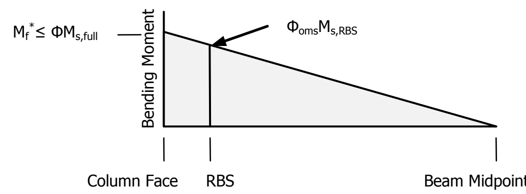

Figure 2: Column Face Bending Moment

Beam Overstrength Factors

Moment resisting frames with reduced beam sections are specifically designed to be used for seismic categories 1 (fully ductile) and 2 (limited ductile). Overstrength factors for beams are found in table 12.2.8(1) of NZS3404 (SNZ,2007) and this is shown in table 1 below. Overstrength factors for Categories 1 and 2 members vary from 1.15 to 1.35. For category 1 members, grade 300, and not from A/NZ the overstrength factor is 1.35. For an overstrength factor of 1.35, even with the maximum flange width reduction, the capacity designed moment at the column face will typically be greater than the design moment capacity of the full beam section. Therefore this beam grade must not be used for category 1 seismic moment frames with RBS. For overstrength of 1.25 and for short bay spans there will be a number of beam sections that will also have a capacity derived moments at the column face greater than the design moment capacity of the full beam section. Category 2 members with steel grade 300 from A/NZ have the lowest overstrength factor of 1.15. Beams designed to this overstrength and with maximum reduced beam sections will all likely to have a capacity derived moment at the column face less than the design moment capacity of the full beam section.

Designers need to ensure the grade of steel supplied for there project matches their design assumptions.

Where the beams support a concrete floor slab and this slab is cast against the steel columns, the presence of the slab increases the flexural overstrength of the beams. If the concrete is isolated from direct bearing on the column the slab participation in increasing the overstrength of the beam is eliminated.

Table 1: Beam Overstrength Factors

	Category 1 Members		Category 2 Members			
Steel Grade	300	300 from A/NZ	300	300 from A/NZ	350	350 from Aust
Overstrength (Φ_{oms})	1.35	1.25	1.25	1.15	1.3	1.25

Reduced Beam Section Minimum Moment Capacity

The maximum reduction of the beam flange width for the reduced beam section is limited to 50%. The reduced moment section capacity based on the maximum flange reduction is 60 - 70% of the full beam width moment section capacity. When comparing I sections of the same depth, the I section with thicker flanges will have lower reduced moment section capacity percentage. Refer to table 2.

Practical Minimum Beam Span

Table 2 presents information for the designer to consider when assessing the suitability of a chosen beam in a moment frame with RBS. The moment section capacity of the reduced beam (column 3) is calculated using the maximum reduction permissible for the beam flange. As the moment frame span decreases the seismic moment gradient increases. Thus the amplification of the overstrength RBS moment increases as the moment frame span decreases. Thus it becomes increasingly difficult to ensure that the moment at the face of the column is less than the design moment capacity of the full beam section. Columns 4 and 5 of table 2 presents the minimum practical clear span length of the beam based on the maximum beam flange reduction permissible. An overstrength factor of 1.25 has been used for category 1 and an overstrength factor of 1.15 has been used for category 2 beams. No allowance has been made for gravity loads. The addition of gravity loads will result in increasing beam clear span minimum limits. The last column in the table shows the maximum reduced section moment capacity for a clear span of 6.5m to ensure that the moment at the face of the column is less than the design moment section capacity of the beam.

Table 2: Beam selection

Section	ϕM_{sx}	$\phi M_{sx,RBS}$ min*	L'$_b$ - practical min		L'$_b$ - 6.5m
			Cat 1	Cat 2	Cat 2 $\phi M_{sx,RBS}$ max
	kNm	kNm	m	m	kNm
610UB101	783	516	7.2	3.9	554
610UB113	829	536	6.1	3.6	587
610UB125	927	592	5.6	3.5	656
700WB115	1023	658	6.6	4.0	715
700WB130	1212	753	5.2	3.5	846
700WB150	1353	814	5.0	3.6	943
700WB173	1610	943	5.0	3.6	1117
800WB122	1229	809	9.0	4.9	848
800WB146	1547	968	6.1	4.0	1063
800WB168	1724	1044	5.7	4.1	1183
800WB192	2031	1197	5.7	4.1	1386
900WB175	2025	1312	8.9	5.2	1369
900WB218	2510	1534	6.4	4.6	1680
900WB257	3074	1821	6.4	4.6	2038
900WB282	3427	1989	6.5	4.6	2269

* Based on 50% flange reduction

Benefits of Using Reduced Beam Sections with Moment Resisting Seismic Frames

Enables Limited Decoupling of Strength and Stiffness

Moments resisting seismic frame beam sections are normally sized to meet drift requirements under gravity and earthquake loadings, often resulting in more lateral resistance than is needed for code specified loadings. The use of the RBS allows the designer to better optimise the structural characteristics of the frame.

The RBS reduces very slightly the stiffness of the structure (between 4% and 9%) because sections are only reduced over very short lengths of the beams. So for a moment resisting frame with reduced beam sections normally does not require any change in the section sizes of the structural elements in order to compensate for this minor stiffness reduction.

While the RBS reduces the ultimate strength of the structure, this may not change the beam members sized for drift governed designs.

Enables the Use of Pre Engineered Connections

For moment resisting seismic frames with reduced beam sections yielding and plastic hinge formation is expected to occur primarily within the reduced beam section zone. The moment that can be developed at the column face is limited and designed to be less than the design moment capacity of the full beam section. Therefore this enables the connection between the beam and column to be designed as elastic. Without having to design the connection for overstrength actions standard pre-engineered connections such as MEP 100/50 Elastic and WM 100/50 Elastic (SCNZ Steel Connect tables (Hyland et al, 2007)) may be used. The use of such pre engineered connections should lead to greater design efficiency.

For the standard seismic moment frame without reduced beam sections the connection of the beam to column must be designed for the overstrength beam moment capacity ($\Phi_{oms}M_s$). The overstrength factor is dependent on steel grade and member seismic category. Overstrength factors for beams from table 12.2.8(1) of NZS3404 (SNZ, 1997) are given in Table 1. For a reduced beam section moment frame the beam to column connection is designed for the design beam section moment capacity (ΦM_s).

Reduces Column Connection Requirements
The panel zone of a beam-column joint is the rectangular segment of the column web surrounded by the column flanges (left and right vertical boundaries) and the continuity stiffeners (top and bottom horizontal boundaries). The panel zone is simultaneously subjected to axial forces, shears, and moments from the columns and beams, as shown in figure 3.

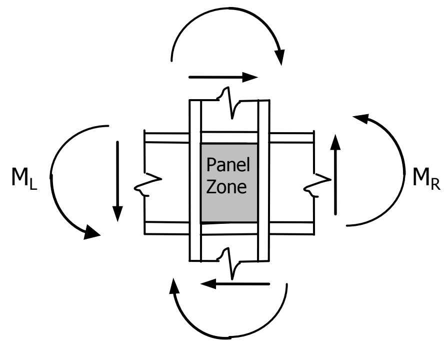

Figure 3: Panel Zone

Resolving equilibrium on the free body diagram of figure 3, the horizontal shear acting in the panel zone can be calculated as:

$$V_p^* = \frac{M_L}{(d_b - t_{fb})_L} + \frac{M_R}{(d_b - t_{fb})_R} - V_{Col}$$

A similar equation is presented in NZS 3404 cl 12.9.5.2 (SNZ, 2007). Therefore the horizontal shear acting on the panel zone is dependent on the magnitude of moment that can be generated by the incoming beams at the column face. For the standard moment frame without reduced beam sections the moment generated at the face of the column will exceed the design beam section moment capacity as the beam will undergo inelastic action and strain hardening to form the plastic hinge. Increased strain hardening will occur for category 1 members as compared to category 2 members. This is reflected in the equation in the Steel Structures Standard for determining the moment generated. The equation is as follows:

$$M_L = M_R = C_2 M_s$$

Where
C_2 = 1.15 for category 1 primary members framing into the connection
C_2 = 1.1 for category 2 primary members framing into the connection
C_2 = 1.0 for category 3 primary members framing into the connection

With the use of a reduced beam section the maximum moment (overstrength moment) that can be generated at the column face is limited to be less than the design beam section moment capacity. Using the design beam section moment capacity as the maximum moment that can be generated at the column face then the equation for the horizontal shear becomes:

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

For interior columns in moment frames the panel zone horizontal shear capacity will typically be exceeded. In some instances exterior columns in moment frames will also have their panel zone horizontal shear capacity exceeded. When this occurs strengthening of the panel zone with the use of doubler plates is required. With the use of the reduced beam section the reduction in horizontal shear is significant and will result in reduce doubler plate thicknesses or may also eliminate the need for doubler plates in some instances. The fabrication and welding of doubler plates is an expensive operation.

The continuity stiffeners are required to transfer the actions from the incoming beam into the web of the column and are essential for good seismic performance. The Steel Structures Standard NZS 3404 cl 12.9.5.3.1(c) (SNZ, 2007) presents two equations for determining the area of continuity stiffeners. There is an equation for the continuity stiffener opposite the compression flange and an equation for when the continuity stiffener is opposite the tension flange. Under seismic actions the loading is reversing. The governing equation is typically the equation for the continuity stiffener opposite the tension flange. Using this equation the total area of the continuity stiffeners is to be at least the equivalent area of the beam flange adjusted for the difference in yield stress. The equation is:

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

This equation assumes the full development of the overstrength moment action generated by full beam section. With the use of a reduced beam section the maximum moment (overstrength moment) that can be generated at the column face is limited to be less the design beam section moment capacity. Therefore the equation could be modified in accordance with note (2)(i) of cl 12.9.5.3.1 NZS3404 (SNZ,2007) to become:

$$A_s \geq \left(0.9A_{fb} - t_{wc}t_{fb}\right)\left(\frac{f_{yb}}{f_{ys}}\right)$$

Welds of continuity stiffeners to column webs and column flange are to be designed to develop the design capacity of the continuity stiffener. The weld of the continuity stiffener remote from an incoming beam may be welded with a nominal fillet weld.

Enables Reduction in Column Sizes

Seismic moment frames for categories 1 and 2 are sized using a capacity design approach. This approach uses a strong column – weak beam philosophy. With the RBS cuts the actions on the column from the beam are reduced. The column must be sized to have adequate strength for the development of the overstrength moment capacity ($\Phi_{oms}M_{s,RBS}$) of the RBS in the attached beams. The maximum moment (overstrength moment) that can be generated at the column face is limited to be less than the design beam section moment capacity. For a typical moment frame the maximum moment at the column face is equal to the development of the overstrength moment capacity of the full beam section. Therefore the RBS enables a reduced column size to be selected to maintain the strong column – weak beam hierarchy required for capacity design.

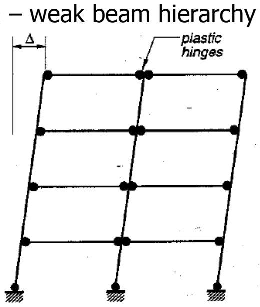

Figure 4: Strong Column - Weak Beam

Enables the Use of Beams that Otherwise Would not meet Seismic Category Geometry Requirements

When determining the flange slenderness for the purposes of classifying member categories, the value of b_f can be taken as the flange width at the ends of centre two thirds of the reduced section. This will mean that there will be some wide flanges of small thickness that will be able to meet the slenderness limits that otherwise would not have for conventional moment frame design.

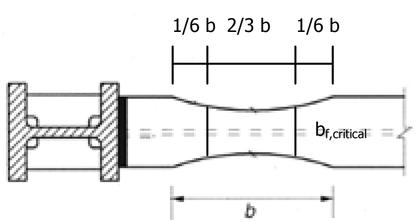

Figure 5: Critical Section Location for Flange Slenderness Check

RBS Applicable when Retrofitting Existing Moment Resisting Seismic Frames

The reduced beam section philosophy can be applied to the retrofitting of existing moment resisting seismic frames. The removal of a portion of the flange will reduce demand on the column. Access is limited or impossible at the upper flange of the beam, due to the presence of a floor slab, so RBS modifications typically occur at the bottom flange of the moment frame only. The design methodology has to be modified accordingly. There has been a great deal of effort and research spent on the use of RBS modifications to existing moment resisting seismic frames. A summary of this work is contained in AISC *Design Guide Series Twelve* (AISC, 1999).

Can be Used for Two-way Moment Frame Collar Detail

The reduced beam section concept is readily applicable to the design of ductile and limited ductile two way moment frame using circular hollow sections. To date this type of connection has been used in low seisimic regions designed as nominal ductile, with limited energy dissipation occurring in the flange plates. Full yielding of the beam section can be achieved, while avoiding inelastic demand on the connection by forcing the hinge zone away from the collar detail.

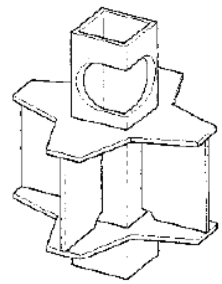

Figure 6: Two-way Moment Frame - External Diapragm

Can be Economically Fabricated

The fabrication of the reduced beam section can easily be done with the use of automated profile cutting machinery. Most of the cost will be in handling the beam, moving to the station and then flipping over to do the other side. The length of RBS cut and flange thickness would not be a significant factor.

It is estimated the additional fabrication cost of a beam shaft with a RBS on each end is as follows:

Additional handling of shaft to RBS cutting machine	\$40
Handling of shaft at workstation	\$40
Profile cutting of RBS	<u>\$40</u>
Total	\$120

A beam 101kg/m and 7m in length has a weight of 707 kg, or 0.7 tonne. Therefore the increase in cost per tonne is \$170.

The profile cutting of the flanges for welded beams such as Steltech beams is also readily achievable. There is no extra handling component. However there will be a small increase in cost due to the time required to set up CNC cutting files and some amount of additional hand welding that will have to be done at the ends. The estimated increase in cost is \$75 per tonne and therefore for 0.7 tonne beam shaft the extra cost is \$53.

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