

Research, Development and Design Rules of Moment Resisting Seismic Frames with Reduced Beam Sections

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Introduction

Structural design for large seismic events must explicitly consider the effects of response beyond the elastic range. The moment resisting seismic frame is designed to form beam plastic hinges near the face of the column. After the discovery of brittle fractures in steel moment frame welded connections in the 1994 Northridge earthquake in California, the reduced beam section (RBS) moment connection was developed and is now extensively used throughout North America and other parts of the world.

The intent of the RBS is to move the plastic hinge region away from the face of the column (figure 2). This is accomplished by reducing the beam's actual plastic moment by removing part of the beam flange as shown in figure 1. This reduced section creates a weaker location where yielding and plastic hinge formation is expected to occur, and results in a reduced moment that develops at the face of the column. By reducing moment demands at the column face the beam-column connections and the column panel zone requirements are reduced. Further more, the use of RBS may also lead to smaller column sizes.

This article describes the development and research of the RBS connection, presents principles for the design of the RBS in a New Zealand context. Design rules appropriate for the design of the reduced beam section connection have also been appended.



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

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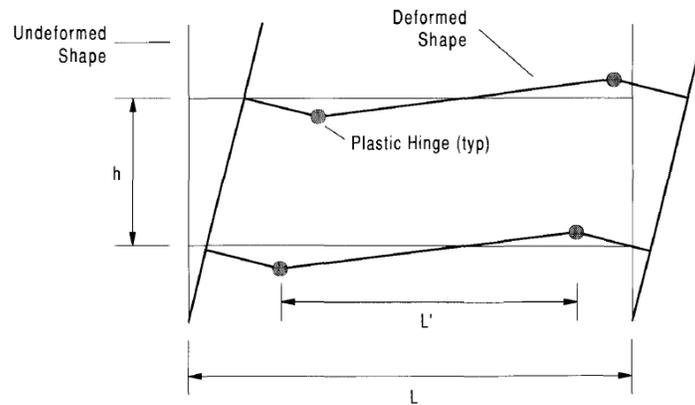


Figure 2: Plastic Hinge Locations

Development of the RBS

The original concept of the RBS was developed in Europe by ArcelorMittal (ARBED) in 1988 and patented in USA in 1992. (ArcelorMittal, unknown date) In the 1994 Northridge (California) earthquake and the 1995 Kobe (Japan) earthquake poor connection behaviour was observed in many moment resisting frames. The RBS concept was widely considered as a smart option to overcome such problems. Following the Northridge earthquake ArcelorMittal gave free use of its patent and the concept was further developed. Following extensive research, design guidance on the RBS has been developed. The RBS has been incorporated into various American specifications (AISC,2006). The Canadian Institute of Steel Construction have included the RBS in their seismic moment connection design guide (CISC,2008).

RBS Research

Introduction to Research

A large number of RBS connections have been tested under a variety of conditions by different investigators at institutions around the world. A listing of relevant research is contained in the bibliography section of (AISC, 2006). These experimental testing programs and analytical studies indicate that the RBS connection provides excellent energy dissipation and significant rotation capacity. A review of available test data indicates that RBS connection assemblies, when designed and constructed according to the limits and procedures outlined in this article, have developed interstorey drift angles of at least 0.04 radians under cyclic loading on a consistent basis. (AISC, 2006)

Tests on RBS connections show that yielding is generally concentrated within the reduced section of the beam and may extend, to a limited extent, to the face of the column (Moore, 2007). Peak specimen strength is usually achieved at an interstorey drift angle of 0.02 radian to 0.03 radian. Specimen strength gradually reduces due to local and lateral torsional buckling of the beam. Ultimate failure generally occurs at an interstorey drift angle of 0.05 to 0.07 radian, typically because of low cycle fatigue fracture initiated by local flange buckling with the RBS (Moore, 2007).

RBS connections have been tested using single-cantilever type specimens (one beam attached to column), and double-sided specimens (specimen consisting of a single column, with beams attached to both flanges). Tests have been conducted primarily on bare-steel specimens, although some testing is also reported on specimens with composite slabs. Tests with composite slabs have shown that the presence of the slab provides a beneficial effect by helping to maintain the stability of the beam at larger interstorey drift angles. (AISC, 2006)

RBS Shapes Tested

The shape, size and location of the RBS all can significantly affect the connection performance. Various shapes have been tested and used in new construction. Successful tests have been conducted on a constant cut, a tapered cut and a radius cut.

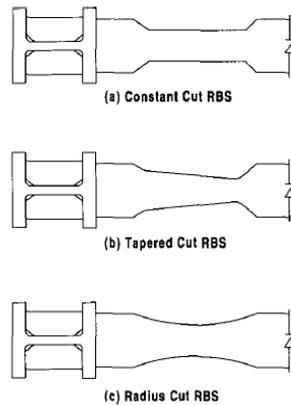


Figure 3: RBS Cut Profiles (Moore et al, 1999)

The tapered cut is intended to allow the section modulus of the beam to match the seismic moment gradient in the reduced beam section, thereby promoting more uniform yielding within the reduced beam section. This is intended to create a more reliable, uniform hinging location. However, stress concentrations at the re-entrant corners of the flange cut may lead to fracture at these locations. After significant plastic rotation, both the constant cut and tapered cut RBS connections, have experienced fractures within the RBS in some laboratory tests. These fractures have occurred at changes in section within the RBS, for example at the minimum section of the tapered RBS. These changes of cross section introduce stress concentrations that can lead to fracture within the highly stresses reduced section of the beam.

The radius cut RBS appears to minimize stress concentrations, thereby reducing the chances of a premature fracture occurring within the reduced section. Furthermore, test results indicate that inelastic deformations distribute over the length of the reduced section. The radius cut is also relatively simple to fabricate and is the most economical. The majority of test reported in the literature used radius-cut RBS sections. The radius cut has been adopted by AISC and CISC as the only RBS profile to be used.

Dimensions of Radius Cut Tested

Dimensions of the RBS cuts for the test specimens reported in the literature vary over a fairly small range. The distance from the face of the column to the start of the RBS cut (dimension "a") ranged from 50 to 75 percent of the beam flange width. The length of the cuts (dimension "b") has varied from approximately 75 to 85 percent of the beam depth. The amount of flange width removed at the minimum section of the RBS has varied from about 38 to 55 percent.

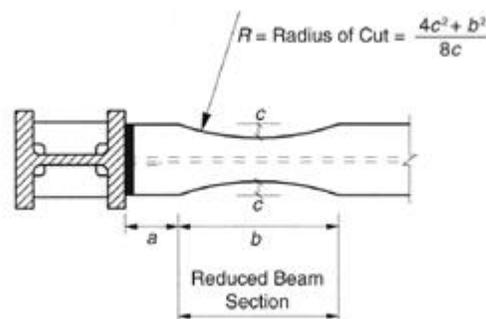


Figure 4: Reduced Beam Section Geometry – Radius Cut (AISC, 2006)

Sizes of Beams and Columns Used in Test Assemblies

A wide range of beam sizes have been tested with the radius-cut RBS. (AISC, 2006) The smallest beam size reported in the literature was a 530UB82. The heaviest beam reported is a W36x300 (447 kg/m, d=932mm, b_f=424mm, t_f=43mm, t_w=24mm).

Most of the tested RBS connections assemblies have used beams spans of approximately 7.6m.

The majority of RBS specimens were constructed with W14 columns (~350mm depth). However, a number of tests have also been conducted using deeper columns, including W18 (~460mm depth), W27 (~690mm depth) and W36 (~920mm depth). Investigation showed that good performance can be achieved with deep columns when a composite slab is present or when adequate lateral bracing is provided for the beam and/or column in the absence of a slab.

Most of the tests of RBS connections have been performed with the beam flange welded to the column flange (ie strong-axis connections). The limited amount of weak axis testing has shown acceptable performance.

Column panel zone strength provided on RBS test specimens has varied over a wide range. This includes specimens with very strong panel zones (no yielding in panel zone), specimens with very weak panel zones (essentially all yielding in panel zone and no yielding in beam), and specimens where yielding has been shared between the panel zone and the beam. Good performance has been achieved for all levels of panel zone strength, including panel zones that are substantially weaker than permitted. However, there are concerns that very weak panel zones may promote fracture in the vicinity of the beam-flange welds due to "kinking" of the column flanges at the boundaries of the panel zone.

Six radius cut specimens were tested with a composite floor slab. (Moore et al, 1999) For two specimens a 25mm gap was intentionally left between the face of column and slab in an attempt to minimize composite actions. For other no such gap was provided. No detrimental effect of the slab was observed in any of these tests. The investigators noted that the slab enhanced overall energy dissipation by delaying beam instability. All composite specimens had no shear studs placed in the region of the RBS or between the face of the column and the start of the RBS.

Successful tests have also been conducted on RBS connections with built-up box columns. The largest box columns for which test data was available was 610mm by 610mm.

Design of Moment Resisting Seismic Frames with Reduced Beam Section

To achieve adequate ductility, specific design and detailing of all elements resisting earthquake actions is required. Design rules for RBS have been developed based on the research and tests previously mentioned and the recommendations and design procedures from the American Institute of Steel Construction. These design rules are appended.

Appropriate Level of Ductility

In order to provide the level of earthquake resistance required by Part 5 of the Loadings Standard (NZS1170.5, 2004), the structure as a whole and all the elements that resist earthquake forces of deformations must be designed to possess an appropriate level of ductility as well as to satisfy the earthquake loading provisions of the loadings Standard.

Classification of Structural Systems

All steel seismic resisting systems are required to be classified into one of four categories for the purposes of seismic design. These are specified in Clause 12.2.3.1 of NZS 3404 (SNZ, 2007) as follows:

- (Category 1) Fully ductile systems.
 - These are to be capable of sustaining structural displacement ductility demands sufficient to strain yielding regions in the primary seismic resisting members or elements into the strain hardening region under severe earthquake loads or effects
- (Category 2) Systems of limited ductility capacity or subject to limited ductility demand
 - These are to be capable of sustaining structural displacement ductility demands sufficient to form yielding regions in the primary seismic resisting members or elements under severe earthquake loads or effects
- (Category 3) Nominally ductile systems
 - These are to be capable of sustaining structural displacement ductility demands sufficient to yield the flanges of primary seismic-resisting members or elements under the design level ultimate limit state earthquake loads or effects and to resist collapse under a maximum considered earthquake as directed by the Loadings Standard, (NZS1170.5, 2004).
- (Category 4) Elastic systems
 - These are expected to respond with minimal structural displacement ductility demand under the design level ultimate earthquake loads or effects and to resist collapse under a maximum

considered earthquake as directed by the Loadings Standard. Elastic systems are not brittle systems.

The reduced beam section moment connection is applicable for category 1 and 2 moment resisting frames.

The moment-resisting frame provisions of section 12.10.2 NZS3404 shall apply to reduced beam section moment connections, including the requirement for a capacity design procedure to be used.

For the application of capacity design procedures to moment resisting frames with reduced beam sections, the reduced beam section zone shall be the primary element. The remaining portions of beam outside the reduced beam sections and the columns shall be secondary elements.

General Philosophy of Capacity Design

In capacity design of a seismic-resisting system, the principal energy dissipating elements or mechanisms (the primary seismic-resisting elements) shall be chosen and suitably designed and proportioned while all other elements of the seismic-resisting system (the secondary seismic-resisting elements) are provided with sufficient reserve strength to ensure that the chosen energy dissipating mechanisms within the seismic-resisting system are maintained throughout the deformations that may occur.

The members which contain these yielding regions are known as primary members. All other members of the seismic-resisting system are known as secondary members. The maximum (overstrength) design actions which can be generated by the primary members are calculated. The design actions for the secondary members are then specifically determined, based on the overstrength capacities of the primary members, to force a hierarchy of inelastic demand between the two types of members, and to provide sufficient reserve strength to ensure that the chosen energy dissipating mechanisms can be maintained.

Capacity design therefore provides an assurance that the inelastic behaviour of a structure will be predictable and that the failure mechanisms will be capable of providing significant energy dissipation. Fundamental to the capacity design philosophy is the need to be able to estimate the maximum design actions that secondary members must be able to resist during the inelastic response. This requires evaluation of the maximum strength that can be developed in the yielding regions as a result of the inelastic demand, taking into consideration:

- the actual size and strength of the primary members, as designed
- the variation in material strength, recognizing that the actual strength will be somewhat greater than the minimum specified strength
- the increase in strength that accompanies strain hardening in yielding regions
- the contribution of the concrete slab to the primary member strength

These factors are taken into account by multiplying the nominal capacity of a primary member by the overstrength factor Φ_{omsr} or Φ_{omss} where slab participation can occur. The value of the overstrength factor depends on the material variation, strain hardening and any effect of composite action in increasing the strength from the primary members. The provisions are given in NZS 3404 Clause 12.2.8 (SNZ, 2007). Structural steel members of a given grade exhibit varying mechanical properties, eg yield stress and tensile strength, with the mean and standard deviation determined by the grade, method of manufacture and age of the steel making facility. The material variation factor, Φ_{omr} , given in NZS 3404 is defined as the ratio of the (97.5 percentile strength / 2.5 percentile strength) and the values have been determined from statistical studies of principally Australian and New Zealand steels.

Reduced Flange Section Geometry

Limits are set on the dimensions of the radius cut based on the tested RBS specimen assemblies. The same limits as used by AISC have been adopted.

Dimension 'a' must be large enough to permit stress in RBS to be spread uniformly across the flange width. (Figure 4) Dimension 'b' must be large enough to avoid excessive inelastic strains within the RBS. The dimensions 'a' and 'b' should be kept as small as possible in order to minimize the increase of moment between the plastic hinge located in the RBS and the face of the column. An upper limit to 'c' is specified to avoid excessive loss of strength and stiffness.

Beam Limits

To achieve dependable ductile performance and prevent premature local buckling beams must be rolled or built-up I-shaped members conforming to the material requirements of NZS 3404.1 and the section geometry requirements of clause 12.5 for the appropriate member ductility category.

When determining the flange slenderness ratio for the purpose of classifying member categories, the value of b_f shall not be taken as less than the flange width at the ends of the centre two-thirds of the reduced section (figure 2 in the appendix) provided that the gravity design actions do not shift the location of the plastic hinge a significant distance from the centre of the reduced beam section. This provision was judged reasonable by AISC since many of the RBS tests conducted as a part of the FEMA/SAC program used a W30×99 beam, which does not quite satisfy the flange width-to-thickness ratio at the uncut section. Nevertheless, the tests were successful.

The heaviest beam tested is used to set the limits on beam depth, beam mass and flange thickness. This limit is set due to the absence of data for larger size beams and not for any poor performance observed and so may be relaxed following further testing.

The beam depth and the beam span to depth ratio are significant in the inelastic behaviour of beam to column connections. For the same induced curvature, deep beams will experience greater strains than shallower beams. Similarly, beams with shorter span to depth ratio will have a sharper moment gradient across the beam span, resulting in reduced length of the beam participating in plastic hinging and increased strains under inelastic rotation demands. Most of the tested RBS connections assemblies have used beams spans of approximately 7.6m. Given the degree to which most specimens significantly exceeded the minimum interstorey drift requirements in America, it is considered reasonable to adopt similar minimum clear span to depth ratios as used by AISC for various moment resisting seismic categories.

Lateral restraint of primary beams shall be in accordance with section 12.6 NZS3404. The testing of the RBS connections showed the importance of adequate lateral performance for good performance.

The yielding region consists of the portion of beam between the face of the column and the end of the reduced beam section cut farthest from the face of the column. Tests indicate that yielding in some instances extended towards the face of the column from the RBS. In the yielding region there are specific requirements for fabrication, attachments and quality control.

Column Limits

Columns acting as a secondary seismic resisting member shall be rolled or built-up I-shaped members conforming to the requirements of Sections 12.5 (section geometry) and NZS 3404.1 (materials)

The size of I shaped section and box section columns have been limited to what has been tested. The behaviour of RBS connections with cruciform columns (figure 3 in appendix) is expected to be similar to that of an I section column and thus the same limits are applied to these dimensions. The box section column may be used for two way moment frames. Although not specifically tested, it is expected that performance will be the same as single plane connections, since the RBS does not rely on panel zone yielding for good performance, and the column is expected to remain essentially elastic for the case of orthogonal connections. For a boxed I section column where single plane moment connections are made to the flange the column depth limit is set at the same limit as for an I section. Where the boxed I section column is used as part of a 2 way moment frame then the dimensions (depth and width) of the boxed I section column is set to the same limits as for a box section. Most of the tests of RBS connections have been performed with the beam flange welded to the column flange (ie strong-axis connections). The limited amount of weak axis testing has shown acceptable performance. However in the American code the RBS moment connection is limited to the strong axis only.

Lateral restraint of columns shall conform to Section 12.6 of NZS3404. The testing of the RBS connections showed the importance of adequate lateral performance for good performance

Calculation of Elastic Drift

The addition of the RBS cutouts will reduce the stiffness of a steel moment frame. This reduction in stiffness is generally quite small. Studies have shown that over a wide range of frame heights and configurations, the average reduction in frame stiffness for a 50 percent flange reduction was on the order of 6 to 7 percent. (Moore et al, 1999) For a 40 percent flange reduction, the reduction was on the order of 4 to 5 percent. Based on these studies a simple approach to determining the increase in elastic drift can be done. It is recommended for flange reductions of up to 50% of the beam width, the effective elastic drifts may be calculated by multiplying elastic drifts based on the gross beam section by 1.1. Linear interpolation may be used for lesser values of beam flange width reduction. Alternatively, a refined structural model, including the reduced stiffness at each connection due to the RBS, can be developed to check the stiffness of the frame. (Mortensen et al, 2008) propose a method for determining spring constants, which can be easily implemented into commercial structural analysis software, to simulate the effects of the RBS cuts of the stiffness of the moment frame.

Column Face Design Action Effects

The location and size of the RBS will determine the capacity design derived column face moment. The overall goal in sizing the RBS cut is to limit the maximum beam moment that can develop at the face of the column to less than the design moment capacity of beam (ΦM_s). The RBS seismic moment diagram is presented in Figure 5. The seismic design bending moment (line 1) for the selected ductility level (μ_{design}) increases linearly from the midpoint of the beam. The design moment capacity (line 2) of the beam is the smallest at the reduced beam section. The RBS cut is sized so that the design moment capacity of the reduced beam section ($\Phi M_{s,RBS}$) is greater than the design bending moment (line 1)

For the maximum seismic event the actual bending moment will be greater than the design bending moment leading to yielding of the RBS. This inelastic demand will result in the section attaining its overstrength moment capacity. This overstrength moment capacity (line 3) is dependent on the statistical variation in yield stress and strength increase due to strain hardening. The bending moment (line 4) will increase until yielding and strain hardening of the reduced beam section occurs. The overstrength bending moment on the face of the column (M_F^c) is the amplification of the overstrength bending moment at the reduced beam section ($\Phi_{oms} M_{s,RBS}$). The overstrength bending moment at the face of column must be less than design moment capacity.

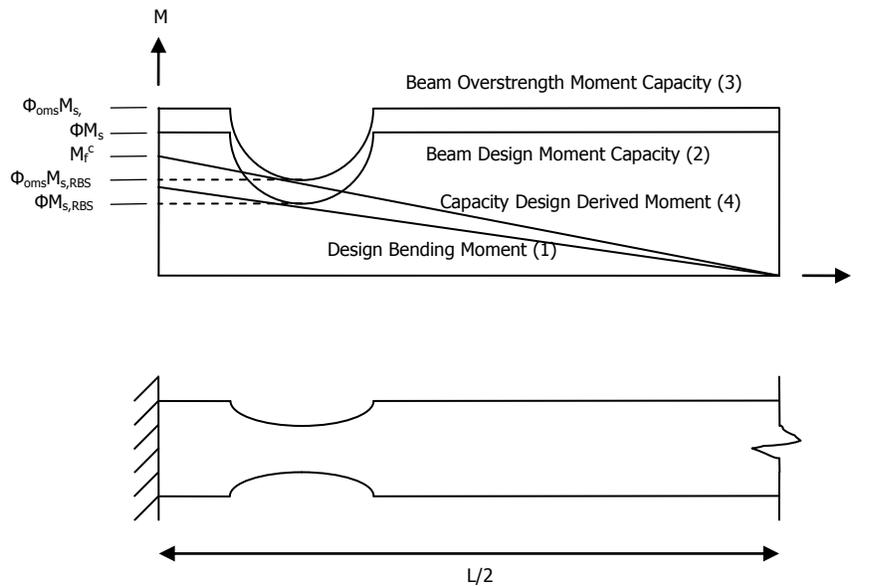


Figure 5: RBS Seismic Moment Diagram (Top), RBS Beam Flange Geometry (Bottom)

By using free body diagrams, equations can be developed for determining design actions at the column face under gravity loadings in addition to the seismic actions described above.

Fabrication and Detailing Requirements

The yielding region consists of the portion of beam between the face of the column and the end of the reduced beam section cut farthest from the face of the column. Large inelastic strains are expected to occur in this region, any discontinuities in the material in the steel beam in the hinge zones could become fracture initiation points.

Any discontinuities created by fabrication or erection procedures must be repaired. Attachments for support of perimeter edge angles, facades, ductwork, piping or other construction are prohibited between the column face and 150mm beyond the extreme end of the reduced beam section. Shear studs and mechanical deck fasteners to the beam flange are also prohibited between the column face and 150mm beyond the extreme end of the reduced beam section.

The RBS cut should be made by thermal cutting. The finished cut must have a maximum surface roughness of $12\mu\text{m}$ in accordance with Table 14.3.3 NZS3404 (SNZ, 2007), avoiding nicks, gouges and other discontinuities. All corners must be rounded to minimize notch effects, and cut edges must be ground in the direction of the flange length (cl 14.3.3.5 NZS3404). Thermal cutting tolerances shall be plus or minus 6mm from the theoretical

cut line. The beam effective flange width at any section shall have a tolerance of plus or minus 10mm. Gouges and notches that occur in the thermally cut RBS surface may be repaired by grinding if not more than 6mm deep. If a sharp notch exists, the area shall be inspected by the Magnetic Particle testing (MT) method after grinding to ensure that the entire depth of notch has been removed. Imperfections outside the above limits shall be repaired by welding in accordance with AS/NZS 1554.1

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Appendix

Reduced Beam Section (RBS) Moment Connection Design Rules

1 General

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 reduced beam section moment connection is applicable for category 1 and 2 moment resisting frames.

2 Design Procedure

- i) The moment-resisting frame provisions of section 12.10.2 NZS3404 shall apply to reduced beam section moment connections subject to the additional requirements of the Reduced Beam Section (RBS) Moment Connection Rules.
- ii) For the application of capacity design procedures to moment resisting frames with reduced beam sections, the reduced beam section zone shall be the primary element. The remaining portions of beam outside the reduced beam sections shall be secondary elements.

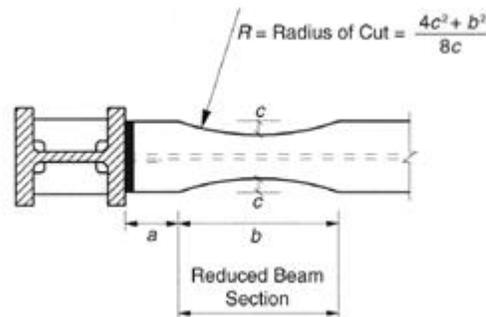


Figure 1: Reduced Beam Section Connection

3 Limitations

3.1 Reduced flange section geometry

Trimming of beam flanges (figure 1) is subject to the following geometrical limits:

$$0.5b_f \leq a \leq 0.75b_f$$

$$0.65d \leq b \leq 0.85d$$

$$0.1b_f \leq c \leq 0.25b_f$$

where:

b_f = width of beam flange, (mm)

d = depth of beam, (mm).

a = distance from face of column to start of RBS cut, (mm).

b = length of RBS cut, (mm).

c = depth of cut at centre of the reduced beam section, (mm)

3.2 Beams

Beams shall satisfy the following limitations:

- (1) Beams shall be rolled or built-up I-shaped members conforming to the material requirements of NZS 3404.1 and the section geometry requirements of clause 12.5 NZS3404 for the appropriate member ductility category.

When determining the flange slenderness ratio for the purpose of classifying member categories, the value of b_f shall not be taken as less than the flange width at the ends of the centre two-thirds of the reduced section provided that the gravity design actions do not shift the location of the plastic hinge a significant distance from the centre of the reduced beam section (figure 2).

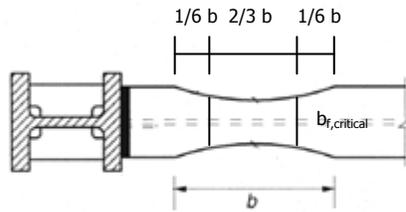


Figure 2: Critical section location for flange slenderness check

- (2) $d \leq 920\text{mm}$ (rolled and built-up shapes).
- (3) Beam mass $\leq 445\text{ kg/m}$.
- (4) $t_f \leq 45\text{ mm}$.
- (5) a) $\frac{L_b}{d} \geq 7$ (category 1 MRF)
 b) $\frac{L_b}{d} \geq 5$ (category 2 MRF)
 where:
 L_b = clear span of beam between column faces
- (6) Lateral restraint of primary beams shall be in accordance with section 12.6 NZS 3404.
- (7) The yielding region consists of the portion of beam between the face of the column and the end of the reduced beam section cut farthest from the face of the column.

3.3 Columns

- (1) Columns acting as a secondary seismic resisting member shall be rolled or built-up I-shaped members conforming to the requirements of Sections 12.5 NZS3404 (section geometry) and NZS 3404.1 (materials).
- (2) For column dimensional limitations refer figure 3

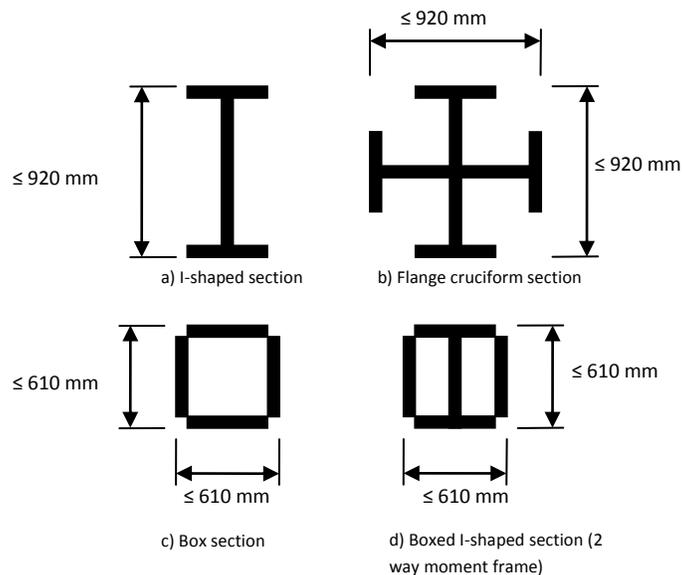


Figure 3 Column geometric limitations

(3) Lateral restraint of columns shall conform to Section 12.6 NZS3404.

4 Calculation of Elastic Drift

- Calculation of elastic drift shall consider the effect of the reduced beam section.
- In lieu of specific calculations, effective elastic drifts may be calculated by multiplying elastic drifts based on gross beam sections by 1.1 for flange reductions up to 50 percent of the beam flange width.
- Linear interpolation may be used for lesser values of beam flange width reduction.

5 Reduced Section Plastic Section Modulus

The plastic section modulus at the centre of the reduced beam section is given by the following equation:

$$S_{RBS} = S - 2ct_f \quad \text{Eqn. 1}$$

- S_{RBS} = plastic section modulus at the centre of the RBS (mm^3)
 S = plastic section modulus, full beam cross-section, (mm^3)
 t_f = thickness, of beam flange, (mm)
 c = depth of cut at centre line of reduced beam section (mm)

6 Column Face Design Action Effects

The column face design capacity derived actions effects are as follows:

- Bending moment

$$M_f^c = \phi_{oms} M_{sx,RBS} + V_{RBS}^* \left(a + \frac{b}{2}\right) + \frac{w^*}{2} \left(a + \frac{b}{2}\right)^2 \leq \phi M_{sx,full} \quad \text{Eqn. 2}$$

where:

$$V_{RBS}^c = \frac{2\phi_{oms} M_{sx,RBS}}{L'} + V_{G\&Q_u,RBS} \quad \text{Eqn. 3}$$

or

$$V_{RBS}^c = \frac{2\phi_{oms} M_{sx,RBS}}{L'} - V_{G\&Q_u,RBS} \quad \text{Eqn. 4}$$

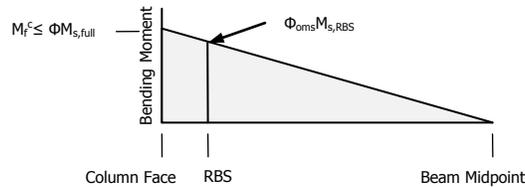


Figure 4 Column face bending moment

- Shear force

$$V_f^c = V_{G\&Q_u} + \frac{2\phi_{oms} M_{sx,RBS}}{L'} \quad \text{Eqn. 5}$$

where:

L' = distance between centres of RBS cuts

M_f^c = capacity design derived column face moment

$\phi M_{sx,full}$ = design section moment capacity, full section

$\phi M_{sx,RBS}$ = design section plastic moment capacity, reduced beam section

$\phi_{oms} M_{sx,RBS}$ = over strength section moment capacity, reduced beam section

V_f^c = design capacity derived shear force at column face

$V_{G\&Q_u}$ = shear force at column face due to the loads $G\&Q_u$

$V_{G\&Q_u,RBS}$ = shear force at centre of reduced beam section due to the loads $G\&Q_u$

V_{RBS}^c = the capacity design derived shear force at the centre of reduced beam section

C6 For a uniformly distributed beam load (figure 5) the values of V_{RBS}^{*} and V_{RBS}^{*} are as follows:

$$V_{RBS}^c = \frac{2\phi_{oms} M_{sx,RBS}}{L'} + \frac{w^* L'}{2} \quad \text{Eqn. 6}$$

$$V_{RBS}^{c'} = \frac{2\phi_{oms} M_{sx,RBS}}{L'} - \frac{w^* L'}{2} \quad \text{Eqn. 7}$$

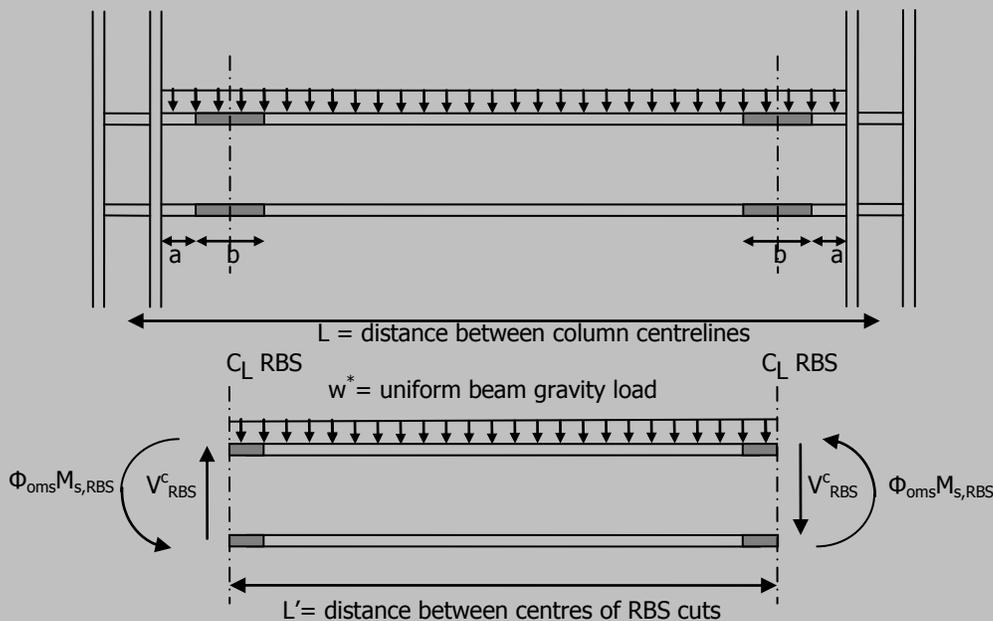


Figure 5: Free body diagrams associated with the calculation of column face design actions

If the gravity design actions on the beam are large, the plastic hinge at one end of the beam may move toward the centre of the beam. If this is the case, the plastic hinge centre line dimension L' should be adjusted accordingly to account for the actual hinge locations. In no case shall the plastic hinge centreline move beyond the centre two-thirds of the reduced beam section unless the beam outside the reduced beam section meets the Code provisions for a primary member.

7 Beam to Column Connections

The beam-to-column connection shall be designed for the full beam design section capacity ΦM_s .

8 Composite Construction

Shear studs and mechanical deck fasteners to the beam flange are prohibited between the column face and 150mm beyond the extreme end of the reduced beam section.