BEHAVIOR OF STEEL MOMENT RESISTING FRAME WITH REDUCED BEAM SECTION

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(Computer Aided Structural Analysis And Design)

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Declaration

This is to certify that

i) The thesis comprises of my original work towards the Degree of Master of Technology in Computer Aided Structural Analysis and Design at Nirma University and has not been submitted elsewhere for a degree.

ii) Due acknowledgement has been made in the text to all other material used.

Sahil G. Gondalia

Certificate

This is to certify that the Major Project Report entitled **"BEHAVIOR OF STEEL MOMENT RESISTING FRAME WITH REDUCED BEAM SECTION"** submitted by **Mr. Sahil G. Gondalia** (Roll No: 13MCLC06) towards the partial fulfillment of the requirements for the Degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis And Design) of Nirma University is the record of work carried out by him under our supervision and guidance. The work submitted has in our opinion reached a level required for being accepted for examination. The results embodied in this major project work to the best of our knowledge have not been submitted to any other University or Institution for award of any degree or diploma.

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Abstract

Extensive research became a necessity after the fall back of Moment-Resisting Frames during 1994 Northridge earthquake and 1995 Kobe earthquake. Reduced Beam Connections used in Moment Resisting Frames emerged as one of the most economical connections for the better ductility enhancement. The performance of Reduced Beam Sections under cyclic loading has been found advantageous by undertaking several experiments. The global parameters are also affected by incorporating Reduced Beam Sections in to the structure.

Guidelines for Design of Reduced Beam Section have been suggested in several codes and documents namely, FEMA 350, EC 8 and AISC 358. Here, Design of Reduced Beam Section suggested by several guidelines, has been studied. EC8 and AISC 358 have derived the guidelines from FEMA 350 itself. For a particular steel section, the design has been done and compared.

A G+15 storey steel building with and without RBS are modelled in STAAD.Pro V8i. The verification of results from STAAD.Pro V8i has also been done considering a portal frame. The effect on Time Period, Base Shear, Displacement and Storey Drift has been compared. The Time Period of the G+15 storey building has increased by the incorporation of RBS due to the reduction in the cross-section of the beam members. The Base Shear shows no much change as the reduction in weight because of use of RBS is very less. The increase in the Displacement and Storey Drift has been observed at each storey.

The FEM analysis of Beam-Column Assemblies with and without RBS has been done using ANSYS Workbench 14.5 to see the variation in deflection of beam and in the stresses at several critical locations. The parameters - Total deformation and Equivalent Von-Mises Stresses - are compared. The deflection of cantilever beam

increases with the use of RBS but the benefit is observed in terms of reduction of stresses at several critical locations such as Beam End and Panel Zone.

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I sincerely wish to express my deep gratitude for the enormous amount of help, guidance, constant encouragement and moral support rendered by my guide, **Prof. Utsav K. Koshti** and **Prof. (Dr.) Urmil V. Dave**. I am also thankful to **Prof. (Dr.) Paresh V. Patel**, Head of Civil Engineering Department, Nirma University, Ahmedabad and **Dr. K. Kotecha**, Director, Institute of technology, Nirma university for their kind support in all respect during my study.

I thank my Parents, who are burnt out like a candle and showing me a light of knowledge.

Sahil G. Gondalia
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Abbreviations, Notations and Nomenclature

S_h .... Hinge Location Distance

d_c .... Depth of Column

b_f .... Width of Flange

t_f .... Thickness of Flange

F_y .... Yield stress for steel

d_b .... Depth of Beam

Z_{RBS} .... Plastic Section Modulus of Reduced Beam Section

M_f .... Expected Moment at the Face of Column

C_{pr} .... Factor to account for peak connection strength

R_y .... Ratio of Expected to specified minimum yield stress

Z_b .... Section Modulus of Beam

V_f .... Shear at face of column

f_u .... Ultimate stress in Steel

γ_{ov} .... Material over-strength ratio

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Chapter 1

Introduction

1.1 General

The Northridge Earthquake in 1994 and the Kobe Earthquake in 1995 showed the extensive need of research in the field of performance of Steel Moment-Resisting Frame structures during Earthquakes. The widespread damage to welded steel moment resisting frame systems was one of the major overall lessons of the Northridge and Kobe earthquake. The most commonly observed damage occurred in or near the welded joint of the bottom flange of a girder to the supporting flange of column. The brittle nature of the fractures detected in numerous welded steel beam to column connections, showed lack in design approaches and codal provisions adopted based on "ductile" structural response of steel structure.

"Structural Engineers Association of California", "Applied Technology Council" and "California Universities for Research in Earthquake Engineering", SAC, are jointly doing research on projects to address this issue.

After this research, two key concepts have been developed to provide highly ductile response and reliable performance: strengthening the connection and/or weakening the beam framing to the column. The ***Reduced Beam Section*** (RBS) is one of the ways to weaken the beam framing to the column.

1.2 Significance of Reduced Beam Section

Reduced beam section (RBS) or dog-bone connection as shown in Figure 1.1 is a type of connection in welded steel moment frames in which portions of the bottom beam flange or both top and bottom flanges are cut near the beam-to-column connection thereby reducing the flexural strength of the beam at the RBS region and thus force a plastic hinge to form in a region away from the connection. The presence of this reduced section in the beam also tends to decrease the force demand on the beam flange welds and so mitigate the distress that may cause fracture in the connection.

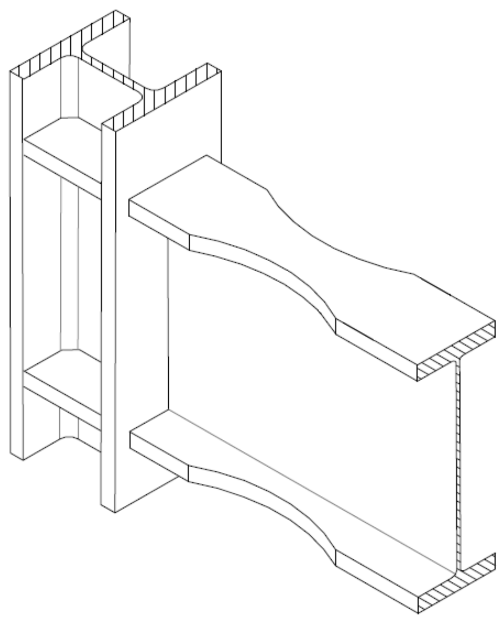


Figure 1.1: Reduced Beam Section

RBS can be bottom flange cut only, or both top and bottom flange cuts. Bottom flange RBS is used if it is difficult or impossible to cut the top flange of an existing beam, for e.g., if the beam is attached to a concrete floor slab.

Various shaped cut-outs are possible in flanges (constant cut, tapered cut or radius cut) to reduce the cross sectional area as shown in the Figure 1.2. The constant cut offers the advantage of ease of fabrication. The tapered cut has the advantage of matching the beam's flexural strength to the flexural demand on the beam under a gravity load. The radius cut is relatively easy to fabricate and because the change in geometry of the cross-section is rather gradual, it also has the advantage of minimizing stress concentration.

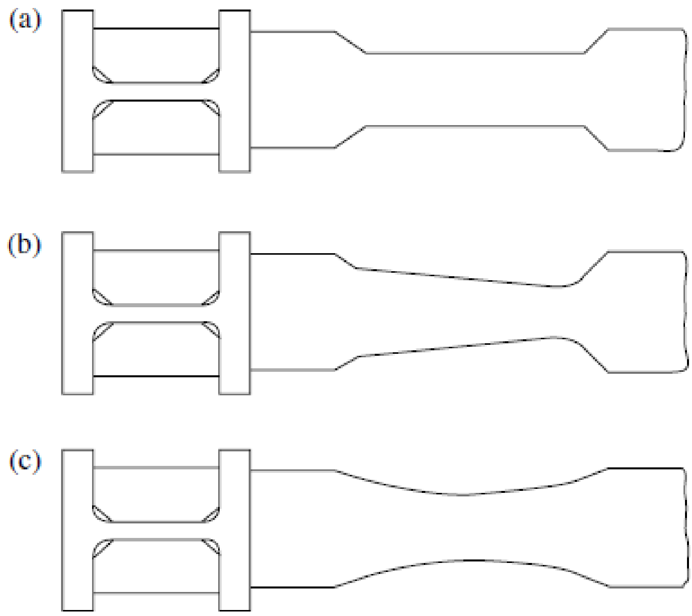


Figure 1.2: RBS Cut Geometries (a) Constant Cut (b) Tapered Cut (c) Radius Cut

Recommendations for the design and detailing of the RBS member are prescribed in FEMA 350 [7], regarding the location and reduction rate of RBS, based on the local performance of tested beam to column assemblies. Euro code 8 - Part 3 [8] and ANSI/AISC 358 [9], also presents design of such type of connections.

1.3 Objective of Study

The objective of the study is as follows:

- To study the design guidelines for Reduced Beam Section given in various standards.

- To study in detail the effect of Reduced Beam Section over conventional sections in Moment-Resisting Frames.

- To study the stress distribution at Reduced Beam Section, beam end and Panel Zone.

- To understand the local behavior of Reduced Beam Section connection.

1.4 Scope of Work

To achieve the above objectives the scope of work is decided as follows:

- Design and detail a Reduced Beam Section as per the guidelines given in relevant codes and documents.

- Modeling and Analysis of Steel MR Frame building with and without Reduced Beam Section in a design software.

- Perform Time History Analysis and study various parameters of both buildings such as Time Period, Base Shear, Displacement and Story Drift.

- Perform FE analysis of Beam-Column Assembly for both buildings and study local behavior in terms of displacement and stresses at several critical locations.

1.5 Organization of Report

Chapter 2 includes the literature review divided into two categories. Category 1 is Dynamic Analysis based Literatures wherein different buildings are described with Dynamic Analysis performed on them and results are discussed. Category 2 is FEM Analysis based Literatures. Here, the FEM analysis on Beam-Column Assemblies have been discussed along with experimental verification.

Chapter 3 includes the guidelines for design of Reduced Beam Section given by different codes, namely, FEMA 350, EC 8 - Part 3 and ANSI/AISC 58. The guidelines are discussed step-wise along with the formulae.

Chapter 4 includes the modeling and design of G+15 storey steel building with and without RBS in Staad.Pro V8i. Non-linear Dynamic (Time History) Analysis is performed and results are compared.

Chapter 5 includes the FEM Analysis of Beam-Column Assemblies, extracted from G+15 storey steel buildings with and without RBS, in ANSYS Workbench 14.5 to study the local effects of RBS.

Chapter 6 discusses the summary and conclusions. It also includes the Future Scope of Work.

Chapter 2

Literature Review

2.1 General

A good range of research has been done in past on various aspects related to the effects and behavior of the MR Frames and Beam-Column assembly with RBS. Some of the relevant papers giving the idea of the benefits of using RBS on various parameters are presented below.

2.2 Dynamic Analysis

Shen [1]investigated the seismic performance of Steel MR Frames with RBS and addressed the design issues related to it. Non-linear static and time history analysis of eight frames with different no. of stories and different RBS configurations were conducted. It was found that Flange Reduction rate and eccentricity decide the strength demand. The strength and stiffness (drift) requirements are, in most cases, were satisfied.

El-Tawil [2] investigated the behavior of RBS frames by subjecting a 4-, 8- and 16-story steel MR frame to a suite of earthquake records. The analysis was done using a computer program. Pushover analysis and suites of transient analysis were conducted on the three frames. Results confirm that RBS frames are capable of economically providing good seismic performance in regions of high seismic risk. Several specific conclusions were also derived.

Kildashti [3] have taken reduced beam sections as a positive approach to mitigate the huge amount of residual drifts which are greatly amplified by P-Δ effects. A 16-story MR Frame is analyzed and the results are processed to assess the effects of RBS detailing on Drift profile, Maximum drift and Residual drift. Results show that RBS diminishes both P-delta effect and residual drifts, simultaneously. It also showed that RBS lower the involvement of lower stories which is the main cause of residual drifts.

2.3 FEM Analysis

Panchumis [4] experimented RBS connection using European HE sections for cyclic loading which were designed using provisions given in EC 8-part 3. FEM model was also prepared which fairly matched the experimental results. This showed that the provisions stated in EC8-part 3 are needed to be adjusted to make it applicable to the European sections.

Kulkarni [5] performed experiment on specimens of Beam-Column assembly of Indian profiles with and without RBS. An FEM model was also created and results were compared with those obtained from the experimental study. It was found that the cyclic performance of RBS moment connection was superior to that without RBS. A reduction in material and labor cost is possible due to elimination of continuity/doubler plates for RBS moment connection.

Sofias [6] conducted experimental study of RBS moment connection with extended endplate with radius cut subjected to cyclic loading. Such connection is widely used in Europe. FEM model was also created and results were in good correlation with the experimental results. The main goal was to protect the connection and its components (endplate, column flange, bolts, welds) from either plasticization or failure. They remained in elastic area due to plastic hinge formation at the RBS.

ADAN [7] tested and modeled four beam-column moment connections without continuity plates for stepwise increasing cyclic tests published by SAC Steel Project. Results showed that all four specimens exceeded the required inter-story drift requirements for use in SMF systems without continuity plates as per FEMA 350. Elimination of continuity plates in RBS moment connections can provide material and labor cost reductions for SMF connections.

2.4 Summary

In this chapter, literatures related to Design of Reduced Beam Section and its applications are reviewed. The factors affected by introduction of RBS at local as well as global level are studied.

Chapter 3

Guidelines for Design of Reduced Beam Section

3.1 General

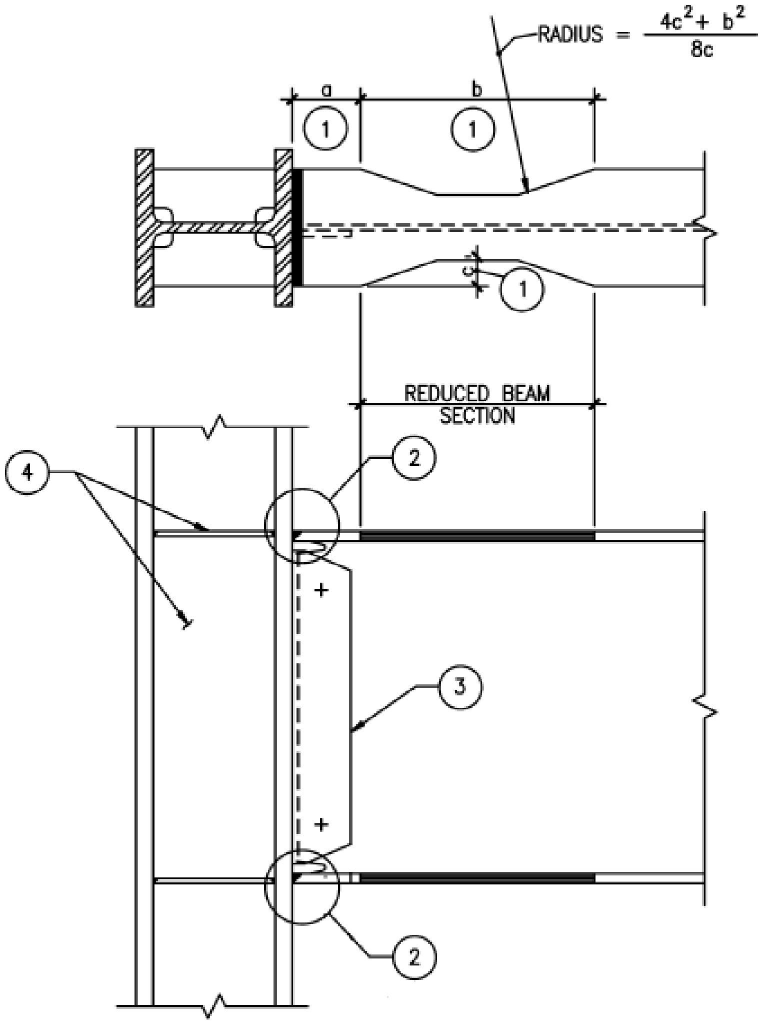
The guidelines for deciding the dimensions and design of RBS based on the shear and flexure parameters have been given in *FEMA-350*. Based on this document, *EC8-Part 3* and *ANSI/AISC 358* also incorporates the steps for design of RBS. These are as shown below.

3.2 FEMA-350

Recommenced Seismic Design Criteria for New Steel Moment-Frame Buildings [9]

The guidelines provided in this document is for design of fully restrained Reduced Beam Section (RBS) Connection. Figure 3.1 provides typical details for such connections. Table 3.1 provides limitations and details of the pre-qualification.

When connection with RBS is used, the elastic drift calculations should consider the effect of the flange reduction. In lieu of specific calculations, a drift increase of 9% may be applied for flange reductions ranging to 50% of the beam flange width, with linear interpolation for lesser values of beam flange reduction.



1) Dimensions of RBS. 2) Rolled or Groove weld. 3) Web Connection. 4) Continuity plates and Web Doubler Plates

Figure 3.1: Detailing of Reduced Beam Section Connection

Table 3.1: Pre-qualification Data for RBS Connections

General	
Applicable systems	OMF, SMF
Hinge Location Distance s_h	d_c+a+b/2
Critical Beam Parameters	
Depth Range	W36 (906 mm) and shallower
Minimum span-to-depth ratio	OMF:5 SMF:7
$b_f/2t_f$	Up to $52/\sqrt{F_y}$
Flange thickness range	1-3/4" (525mm) maximum

The steps suggested by FEMA-350 for the design of RBS are as under:

Step 1: Determine the length and location of the beam flange reduction, based on the following:

$$a \cong (0.5 - 0.75)b_f \tag{3.1}$$

$$b \cong (0.65 - 0.85)d_b \tag{3.2}$$

where a and b are as shown in Figure 3.1, and b_f and d_b are the beam flange width and depth respectively.

Step 2: Determine the depth of the flange reduction, c, according to the following:

a) Assume c = $0.20b_f$

b) Calculate Z_{RBS}.

c) Calculate M_f using $C_{prS} = 1.15$.

d) If $M_f < C_{pr}R_yZ_bF_y$ the design is acceptable. If M_f is greater than the limit, increase c. The value of c should not exceed $0.25b_f$

Step 3: Calculate M_f and M_c based on the final RBS dimensions according to the methods of Section 3.2.7 of FEMA 350[9].

Step 4: Calculate the shear at the column face according to the equation:

$$V_f = 2\frac{M_f}{L - d_c} + V_g \tag{3.3}$$

where, V_g = shear due to factored gravity load.

Step 5: Design shear connection of the beam to the column.

Step 6: Design the panel zone according to the methods of Section 3.3.3.2 of FEMA 350[9].

Step 7: Check continuity plate requirements according to the methods of Section 3.3.3.1 of FEMA 350[9].

Step 8: Detail the connection.

3.3 EC8 - Part 3

Design of Structures for Earthquake Resistance: Strengthening and Repair of Buildings[10]

According to EC8-Part3, RBSs or Dog-Bones behave like a fuse, thus protecting beam-to-column connections against early fracture. The minimum rotations that can be achieved at each Limit State are as below:

Table 3.2: Rotations of RBSs (in radians)

DL	**SD**	**NC**
0.010	0.025	0.040

where,

DL = Damage Limitation

SD = Severe Damage

NC = Near Collapse

To achieve the rotations given in the table, the design of RBS should be carried out through the procedure outlined hereafter.

Step 1: Compute the length and position of the flange reduction by defining 'a' and 'b' as follows:

$$a = 0.60b_f \tag{3.4}$$

$$b = 0.75d_b \tag{3.5}$$

where, b_f and d_b are the beam flange width and depth, respectively.

Step 2: Compare the distance of the plastic hinge formation from the beam edge given by:

$$s = a + \frac{b}{2} \tag{3.6}$$

Step 3: Compute the depth of the flange cut (g); it should be not greater than $0.25b_f$. However, as first trial assume:

$$g = 0.20 \times b_f \tag{3.7}$$

Step 4: Compute the plastic modulus (Z_{RBS}) and hence the plastic moment ($M_{pl,Rd,RBS}$) of the RBS.

$$M_{pl,Rd,RBS} = Z_{RBS} \times f_y \tag{3.8}$$

The plastic modulus of RBS is, $Z_{RBS} = Z_b - 2 \times g \times t_f \times (d - t_f)$
where,Z_b is the plastic modulus of the beam.

Step 5: Compute the plastic shear ($V_{pl,Rd}$) in the section of plastic hinge formation via the free body equilibrium of the part (L') between hinges.

$$V_{pl,Rd} = \frac{2 \times M_{pl,Rd,RBS}}{L'} + \frac{w \times L'}{2} \tag{3.9}$$

where, w is the uniform beam gravity load.

Step 6: Compute the beam plastic moment $M_{pl,Rd,b}$ as follows:

$$M_{pl,Rd,b} = \frac{f_u + f_y}{2 \times f_y} \times Z_b \times \gamma_{ov} \times f_y \tag{3.10}$$

Step 7: Check that the bending moment $M_{cf,Sd}$ is less than $M_{pl,Rd,b}$; otherwise increase the cut-depth c and repeat steps (4) to (6). The length g should be chosen such that the maximum moment at the column flange is about 85% to 100% of the beam expected plastic moment.

Step 8: Check width-to-thickness ratios to prevent local buckling. The flange width should be measures at the ends of the center of 2/3 of the reduced section of the beam unless gravity loads are large enough to shift the hinge point significantly from the center point of the reduced section.

Step 9: Compute the reditus (r) of cuts in both top and bottom flanges over the length b of the beam:

$$r = \frac{b^2 + 4 \times g^2}{8 \times g} \tag{3.11}$$

3.4 ANSI/AISC 358

Pre-qualified Connections for Special and Intermediate Steel Moment Frame for Seismic Applications [11]

Several Pre-qualification limits are defined that the specified element must satisfy. Limits are related to:

a. Beam Limitations

b. Column Limitations

c. Column-Beam Relationship Limitations

d. Beam Flange-to-Column Flange Weld Limitations

e. Beam Web-to-Column Flange Connection Limitations

f. Fabrication of Flange Cuts

Step 1: Choose trial values for the beam sections, column sections and RBS dimensions a, b and c subject to the limits:

$$0.5b_f \leqslant a \leqslant 0.75b_f \tag{3.12}$$

$$0.65d_b \leqslant b \leqslant 0.85d_b \tag{3.13}$$

$$0.1b_f \leqslant c \leqslant 0.25b_f \tag{3.14}$$

Step 2: Compute the plastic section modulus at the center of the reduced beam section:

$$Z_{RBS} = Z_b - 2ct_f(d - t_f) \tag{3.15}$$

Step 3: Compute the probable maximum moment, M_{pr}, at the center of the reduced beam section:

$$M_{pr} = C_{pr}R_yF_yZ_{RBS} \tag{3.16}$$

Step 4: Compute the shear force a the center of the reduced beam sections at each end of the beam.

Step 5: Compute the probable maximum moment at the face of the column. The moment at the face of the column shall be compared from a free-body diagram of the segment of the beam between the center of the reduced beam section and the face of the column.

Based on this free-body diagram, the moment at the face of the column is computed as follows:

$$M_{pr} = M_{pr} + V_{RBS}S_h \tag{3.17}$$

Step 6: Compute M_{pe}, the plastic moment of the beam based on the expected yield stress:

$$M_{pe} = R_y F_y Z_b \tag{3.18}$$

Step 7: Check the flexural strength of the beam at the face of the column:

$$M_f = \phi_d M_{pe} \tag{3.19}$$

If above equation is not satisfied, adjust the values of c, a and b, or adjust the section size, and repeat steps (2) to (7).

Step 8: Determine the required shear strength V_u of beam and beam web-to-column connection from:

$$V_u = \frac{2 \times M_{pr}}{L_h} + V_{gravity} \tag{3.20}$$

Check the design for shear strength of beam according to chapter G of the AISC specifications.

3.5 Design of RBS

It has been observed that as the EC8 and AISC358 are derived from the FEMA350 guidelines, the procedure for design and capacity check remains almost same. This can be observed from the following example.

The design as per the three guidelines narrated above has been done for ISWB 550. The reduction has been taken as 0.20 times width of flange(b_f). Figures 3.2, 3.3 and 3.4 are the screen-shots of the design of RBS done in Excel.

FEMA Guidelines : 350								
Column depth			Section	ISWB 550		Reduced Dimensions		
Dc	1.35	m	D	0.55	m	a	0.15	m
Beam Length			bf	0.25	m	b	0.4125	m
Span	7	m	tf	0.0176	m	c	0.05	m
			tw	0.0105	m	Zrbs	0.0021	m^3
Mpr	333.85	kNm	Cpr	1.15				
Vp	179.15	kN	Ry	1.3				
Mf = Mpr + Vp*X			Zrbs	0.002101				
=	397.672	kNm	Fy	250000	kN/m^2			
			Cpr*Ry*Zrbs*Fy =		785.328	kNm		
					OK			
Final Mf and Mc			Shear at column face					
Mf	397.672	kNm	Vg	41.1825	kN			
Mc	518.598	kNm	Vf = 2*(Mf/(L-dc) + Vg					
			=	154.8031	kN			
			Design connection for this shear force					

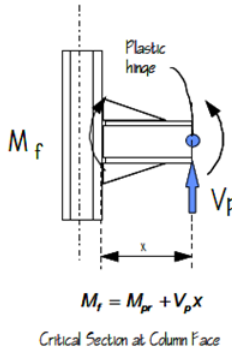


Figure 3.2: Design as per FEMA 350

AISC 568-05								
Column depth			Section	ISWB 550		Reduced Dimensions		
Dc	1.35	m	D	0.55	m	a	0.15	m
Beam Length			bf	0.25	m	b	0.4125	m
Span	7	m	tf	0.0176	m	c	0.05	m
UDL gravit	14.04	kN/m	tw	0.0105	m			
Zb	0.00304	m^3						
Zrbs	0.0021	m^3						
Cpr	1.15			Mpr= Cpr*Ry*Zrbs*Fy =		785.328	kNm	
Ry	1.3			Vp	179.15	kN		
Zrbs	0.0021			Mf = Mpr + Vp*X =		849.15	kNm	
Fy	250000	kN/m^2		Mpe	987.426369	kNm		
					OK			
Vf = 2*Mpr/L' + Vg =	298.946	kN						
Design connection for this shear force								

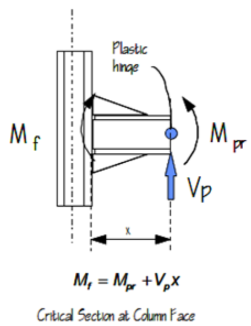


Figure 3.3: Design as per AISC 358

EURO CODE 8 - PART 3								
Column			Section	ISWB 550		Reduced Dimensions		
Depth	1.35	m	D	0.55	m	a	0.15	m
Beam			bf	0.25	m	b	0.4125	m
Span	7	m	tf	0.0176	m	g	0.05	m
UDL gravity	14.04	kN/m	tw	0.0105	m			
Zb	0.00304	m^3						
Zrbs	0.0021	m^3						
M pl. RBS	525.303	kNm						
V pl. RBS	204.774	kN						
M pl. B	1378.6	kNm						
Mcf,Sd	598.253	kNm						
	OK							

Figure 3.4: Design as per EC8-P3

The check in all the three guidelines is between the capacity of beam end and the expected moment at the beam end considering the RBS. As seen, the assumed dimensions come out to be sufficient as per all the three guidelines. Compared to FEMA 350 and AISC 358, EC 8 gives higher beam end capacity due to the incorporation of Material Over-strength Factor(γ_{ov}).

3.6 Summary

In this chapter, the procedure of deciding the dimensions and checking the capacity of RBS as per FEMA 350, EC8-Part3 and AISC 358 has been shown. Also, as an example, the dimensions and capacity has been checked for one I section. It suggests that the decided dimensions are satisfactory for strength check.

Chapter 4

Analysis and Design of G+15 storey building

4.1 General

To understand the change in global parameters of the building due to introduction of RBS, G+15 storey buildings, one regular and the other with RBS, are modeled in STAAD.Pro V8i. The parameters like Time period, Base Shear, Deflection and Storey Drift are compared for both the buildings. Dynamic Analysis of the buildings give the above results with more accuracy. Here, Time History Analysis Method has been used. Later, the building is designed using Limit State Method.

4.2 G+15 storey building

4.2.1 Building Details

The building details are as follows :

Bottom Storey Height = 4.2m

Typical Storey Height = 3.5m

Bay Dimensions - 4m x 7m

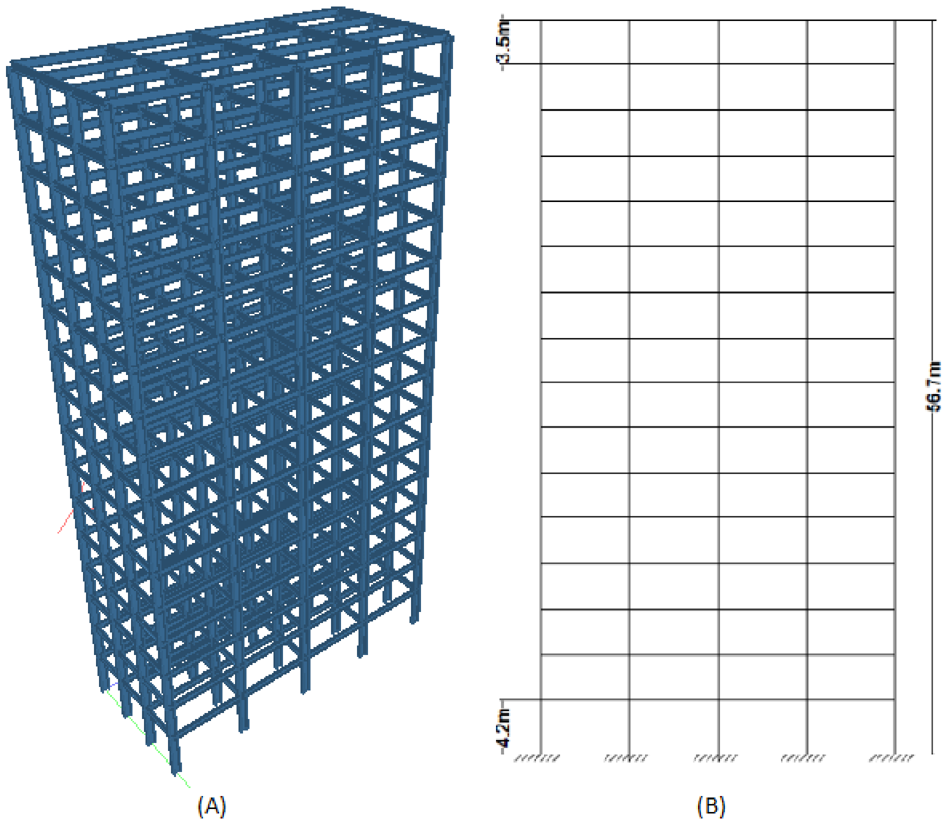


Figure 4.1: (A) Rendered View (B) Elevation of Building

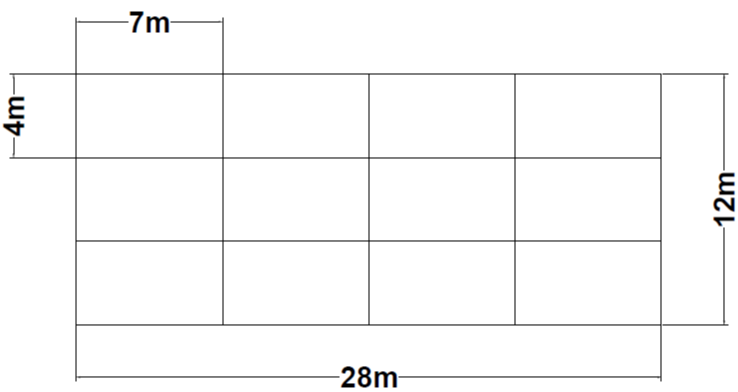


Figure 4.2: Plan of Building

Properties of Steel used:

Modulus of Elasticity $= 2 \times 10^5 MPa$

Poisson's Ratio $= 0.3$

Density $= 7833.41$ kg/m^3

4.2.2 Section Properties

The beam sections in both the buildings are ISMB 600 sections. The columns are built-up sections as the pre-defined Indian sections does not meet the requirements. The RBS has been formed from ISMB 600 section itself as per FEMA 350 guidelines. The calculations of dimensions of RBS are as shown below and the details of all these sections are as in the Table 4.1:

$a = 0.6b_f = 0.6 \times 210 = 126\text{mm}$

$b = 0.75d_b = 0.75 \times 600 = 450\text{mm}$

$c = 0.2b_f = 0.2 \times 210 = 42\text{mm}$

Here, c is the depth of cut of flange width. So, the width of flange of RBS will be $210 - 2 \times 42 = 126\text{mm}$

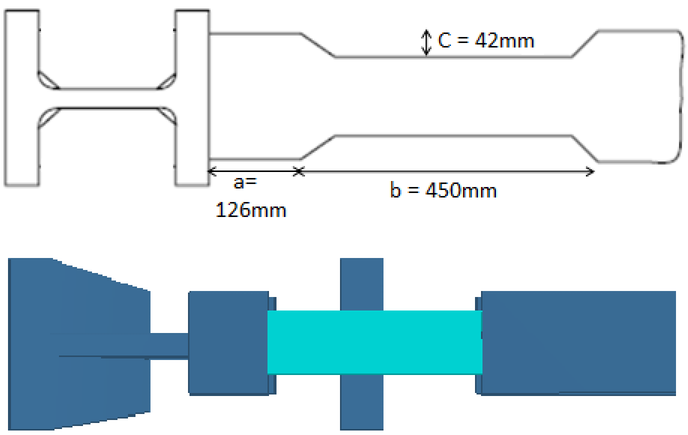


Figure 4.3: RBS Details

Table 4.1: Properties of Steel Sections

Properties	Column	Beam	RBS
Depth	738.87mm	600mm	600mm
Flange Width	354.58mm	210mm	126mm
Flange Thickness	83.06mm	20.8mm	20.8mm
Web thickness	46mm	12mm	12mm

The RBS is modeled as a separate element by creating nodes on regular beam at a distance 'a' from the connection. To check whether the analysis in STAAD.Pro is giving correct results, validation has been done manually by Matrix (Stiffness) Method. For validation, a hypothetical case is taken considering a portal frame subjected to UDL on beam. The manual and software results significantly matches. The detailed calculations are shown in Appendix A.

4.2.3 Load Cases and Combinations

The loads applied to the building are discussed in detail below:

Dead Load:-

On typical floors, the dead load applied is 3 kN/m^2 and that for the top storey is 2 kN/m^2. The dead load includes the super-imposed Dead Load. The self-weight of all the structural elements is also included in the Dead Load.

Live Load:-

On typical floors, the live load applied is 2 kN/m^2 and that for the top storey is 1.5 kN/m^2.

Earthquake Load:-

The seismic parameters are set considering IS 1893:2002 as:

Zone V i.e. Z = 0.36

Response Reduction Factor R = 5

Importance Factor I = 1

Damping Ratio DM = 0.02

The Earthquake force has been applied in both X and Z direction of the building. The seismic weight of the building includes full Dead load and 25% of Live load. The forces on each storey for both the buildings are as shown in the Figure 4.4.

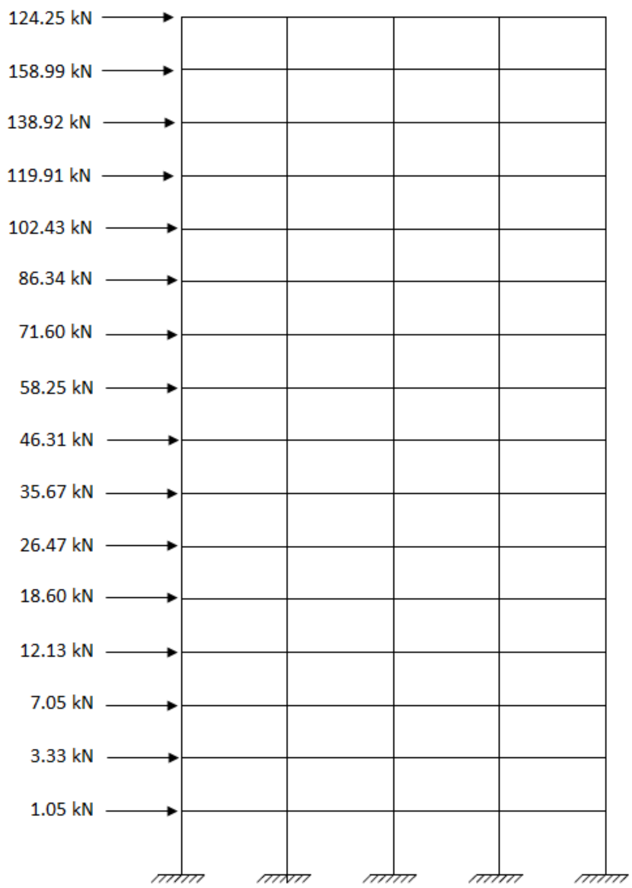


Figure 4.4: Forces on each storey(EQ_X)

Wind Load:-

The wind load has been calculated as per IS 875:1987 Part 3. The load is applied in both X and Z directions of the building. The intensities used as an input in STAAD.Pro are given in Table 4.2:

Table 4.2: Wind Intensities

Intensity (kN/m^2)	**Height (m)**
0	0
0.8	10
0.9	15
0.98	20
1.093	30
1.234	50
1.383	100

Load Combinations:-

The Load Combinations have been taken as per Limit State Method from Cl. 5.3.3 IS 800:2007[8] for the Load Cases Dead Load, Live Load, Earthquake Load and Wind Load. The combinations are as below:

1) 1.5DL + 1.5LL

2) 1.5DL ± 1.5EQX

3) 1.5DL ± 1.5EQZ

4) 1.2DL + 1.2LL ± 1.2EQX

5) 1.2DL + 1.2LL ± 1.2EQZ

6) 1.5DL ± 1.5WLX

7) 1.5DL ± 1.5WLZ

8) 1.2DL + 1.2LL ± 1.2WLX

9) 1.2DL + 1.2LL ± 1.2WLZ

4.2.4 Time History Analysis

Time History Analysis provides structural response of building subjected to dynamic loading which may vary according to the specified time function. Dynamic equilibrium equations are solved either by modal method or direct-integration method. STAAD.Pro V8i solves this by Modal analysis.

Here, EL Centro NS time history has been applied to both the structures and the response has been studied in terms of Time Period, Base Shear, Deflection, Storey Drift and Acceleration. The EL Centro NS time-acceleration graph is as shown in the Figure 4.5:

The Peak Ground Acceleration of this excitation is 0.32g.

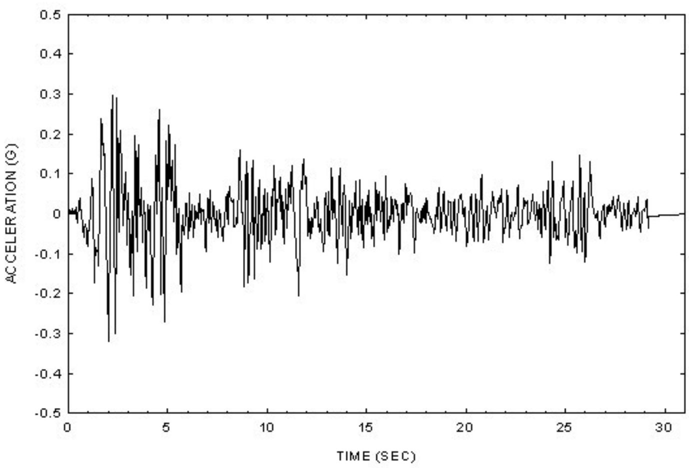


Figure 4.5: EL Centro NS Excitation

Time History Analysis can be carried out by two methods: Modal Analysis method and Direct Integration Method. In STAAD.Pro V8i, the Modal Analysis approach is adopted. This is reflected in the definition of Time History Command. As shown in Figure 4.6, the ELCENTRO_NS.txt is read by the software in terms of acceleration. The damping is defined as 0.02 (for steel). Now, in load cases, a dynamic load case is required to be defined where in the Dead Load is defined in all three directions. This is because STAAD.Pro performs Modal Analysis.

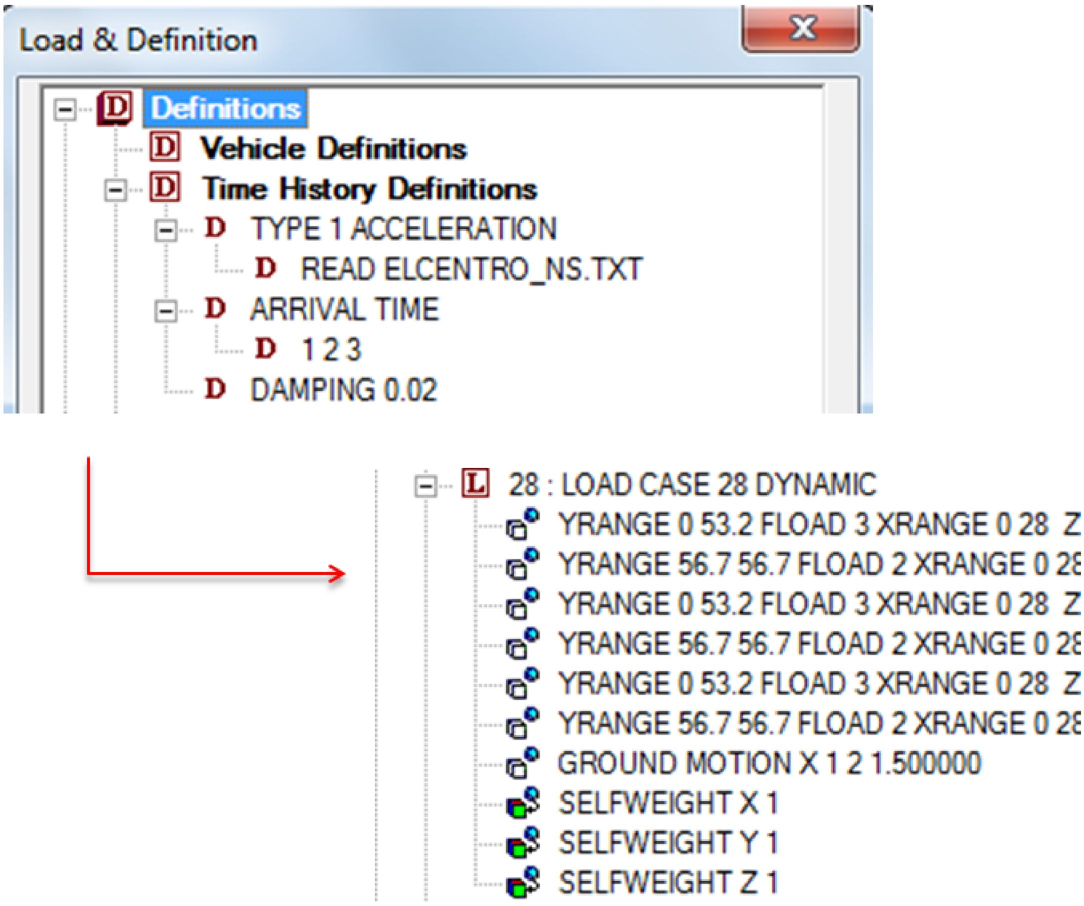


Figure 4.6: Time History Definition in STAAD.Pro V8i

After the time history analysis is performed in the software, the time step used in the analysis is obtained from the output file. Here, the time step used is 0.00139sec and total number of time steps used in the solution are 40852. No. of mode shapes considered are 45.

The output obtained is in the form of graphs for displacement and acceleration with respect to time. For each node on each storey that graph can be obtained. Due to diaphragm action, all the nodes on a particular storey shows similar amount of displacement. So, as shown in the Figure 4.5 and 4.6, the highlighted node at each storey is chosen and the quantities are noted down.

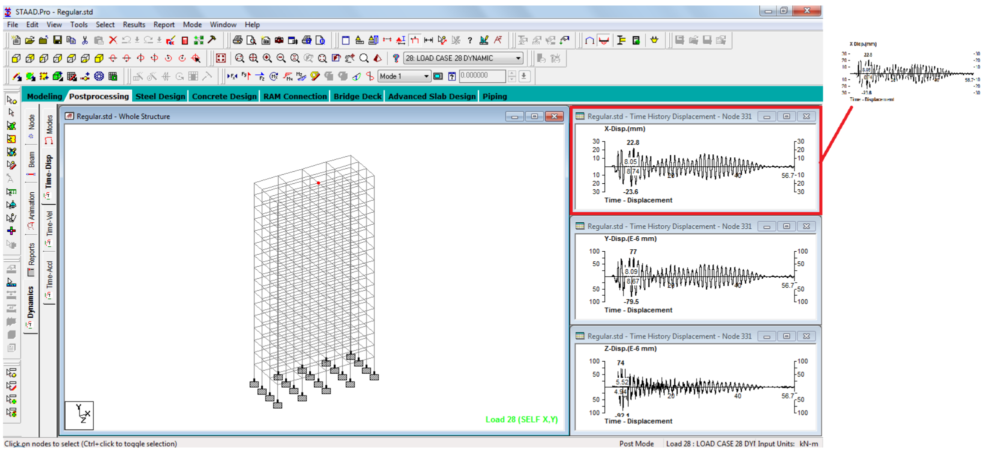


Figure 4.7: Time-Displacement

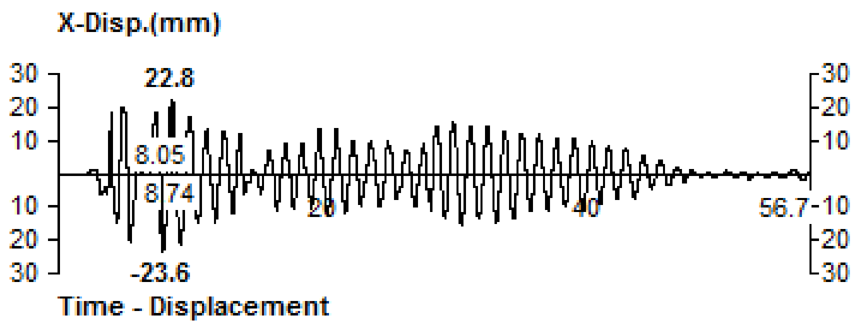


Figure 4.8: Time-Displacement Plot

Similarly, for the same nodes the acceleration is also noted down and graphs are obtained as shown in Figure ?? and ??. Acceleration at a storey gives the idea about whether the humans can perceive the motion of the building or not. Research work suggests human perception levels relative to the acceleration that a human can feel or bear.

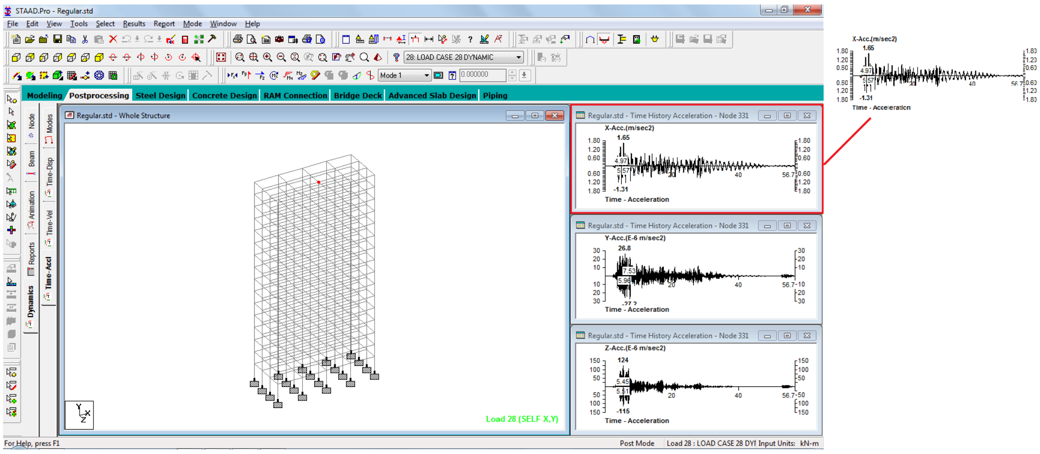


Figure 4.9: Time-Acceleration

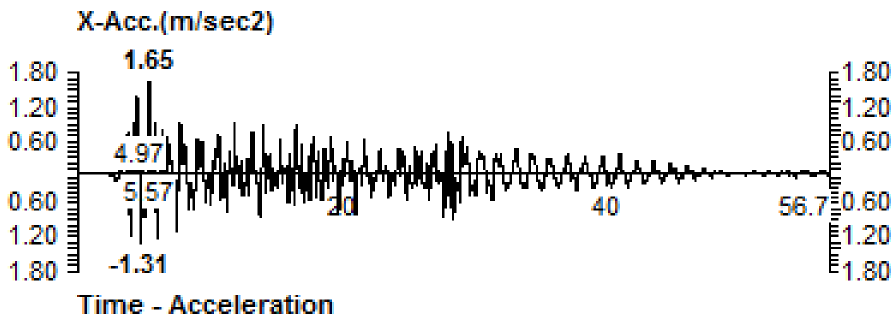


Figure 4.10: Time-Acceleration Plot

4.2.5 Design of Building

The design of building has been done using Limit State Method suggested by IS 800:2007[8]. The beam, column and RBS are steel sections and no concrete elements are used. As the Indian section with highest capacity does not satisfy the requirement as a column element, built-up sections are used as stated in Section 4.2.2. With the above stated elements, the building is safe for the load combinations incorporated based on limit states (Refer Section 4.2.3).

The design of Reduced Beam Section has been checked manually for a top storey beam. The design has been checked by FEMA 350 guidelines as shown below.

Selected Dimensions 'a', 'b', 'c' are 0.15m, 0.4125m and 0.42m respectively. Section modulus of RBS is,

$$Z_{RBS} = Z_b - 2(c)(t_f)(d_b - t_f) = 0.00245m^3 \tag{4.1}$$

Moment at center of RBS,

$$M_{pr} = C_{pr} R_y Z_{RBS} F_y = 916.96kNm \tag{4.2}$$

Shear force at center of RBS $V_p = 9.12$kN

Location of plastic hinge,

$$S_h = a + b/2 = 0.3562m \tag{4.3}$$

Expected moment at the face of column,

$$M_f = M_{pr} + V_p \times S_h = 920.2kNm \tag{4.4}$$

Moment capacity of Beam end,

$$M_e = Z_b f_y R_y = 1127.75kNm \tag{4.5}$$

Here, $M_f < M_e$. i.e. SAFE.

The calculations suggest that the assumed dimensions can be safely adopted for a shear force at the center of RBS of 580kN.

4.3 Results

By incorporation of RBS, the time period of the building changes. The program calculated time period of both the buildings have been found. Also, the base shear is obtained. The time period and base shear of the buildings are as below:

Table 4.3: Time Period and Base Shear of buildings

Type of Building	**Time Period**	**Base Shear**
Regular	1.86sec	1163.14 kN
RBS	2.32sec	1160.62 kN

The displacement of each storey of both the buildings are noted and graph of no. of storeys versus Displacement and Storey Drift has been plotted.

Table 4.4: Displacement and Storey Drifts

Storey	***Displacement*(mm)**		***Drift*(mm)**	
	Regular	***RBS***	***Regular***	***RBS***
16	231.3198	284.2349	2.79585	4.58127
15	228.524	279.6537	3.61989	6.1803
14	224.9041	273.4734	5.01291	8.12268
13	219.8912	265.3507	6.89643	10.03563
12	212.9947	255.3151	9.2214	12.00744
11	203.7733	243.3076	11.79162	14.18526
10	191.9817	229.1224	14.46975	16.53966
9	177.512	212.5827	17.11845	18.96273
8	160.3935	193.62	19.49247	21.27789
7	140.901	172.3421	21.582	23.42628
6	119.319	148.9158	23.20065	25.42752
5	96.11838	123.4883	24.1326	27.11484
4	71.98578	96.37344	23.77944	28.19394
3	48.20634	68.1795	20.9934	27.8604
2	27.21294	40.3191	16.65738	24.60348
1	10.55556	15.71562	10.55556	15.71562
0	0	0	0	0

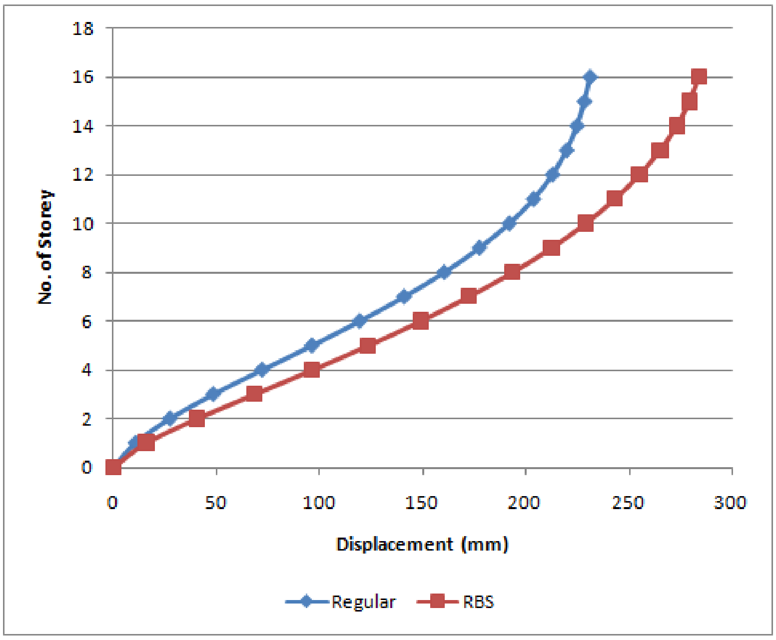


Figure 4.11: Displacement Plot

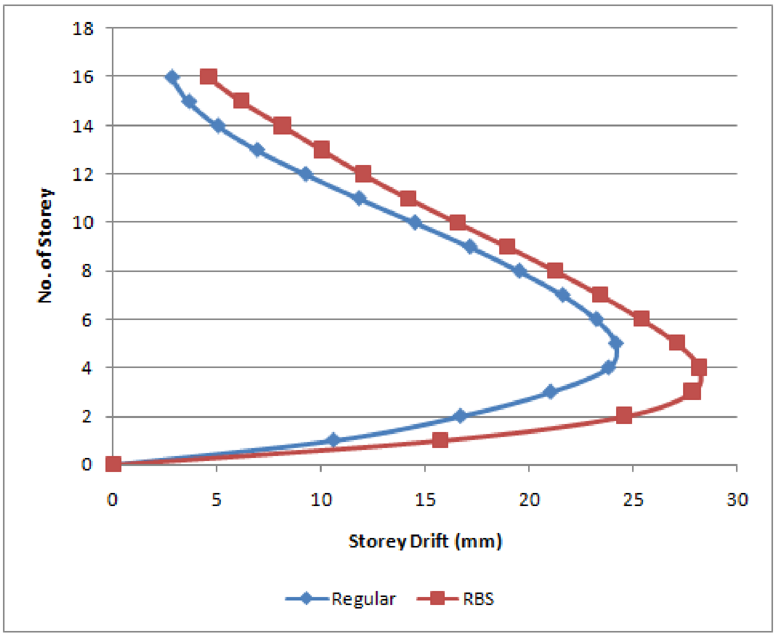


Figure 4.12: Storey Drift Plot

RBS being a cut in the regular beam element, reduces the weight of the building. The total reduction of weight is calculated here to see the cost benefit of using RBS. Table 4.5 summarizes the weights of elements in both the buildings.

Table 4.5: Material Weight

Element	**Length**	**Weight**
Regular building		
Column	1134 m	7418.67 kN
Beam	2752 m	3302.18 kN
	Total	10720.85 kN
Building with RBS		
Column	1134 m	7418.67 kN
Beam	2352.3m	2822.56 kN
RBS	399.71m	366.7 kN
	Total	10607.93 kN

So,

the difference in weight = 112.92 kN = 11.51 ton.

Considering cost of steel as Rs.45/kg,

Cost Benefit = Rs.45000 $\times$ 11.51 ton = Rs.5.18 lacs.

4.4 Summary

In this chapter, the parameters Time Period, Deflection and Storey Drift has been compared for G+15 storey building with and without RBS. The results show that there has been an increase in the Time Period of building with RBS by 25% over the building with conventional beams. Also, the deflection of the top storey of the building with RBS increases by 23% over the regular building. The storey drifts also shows an increase with incorporation of RBS.

Chapter 5

FEM analysis using ANSYS

5.1 General

The change in plastic moment distribution by introduction of Reduced Beam Section can be studied by performing Pushover Analysis of the buildings with and without RBS. To perform that, it is necessary to know the exact location of formation of plastic hinge in the beam as well as in column.

5.2 Beam-Column Assemblies Extracted from G+15 storey Buildings

The Finite Element Method can give the exact stress contours in the element due to the applied load. This helps in determining the location where the plastic hinge may form. So here, a Beam-Column Assembly is extracted from the first storey of G+15 storey building mentioned earlier. The notations of the extracted Assembly is as shown in Figure 5.1:

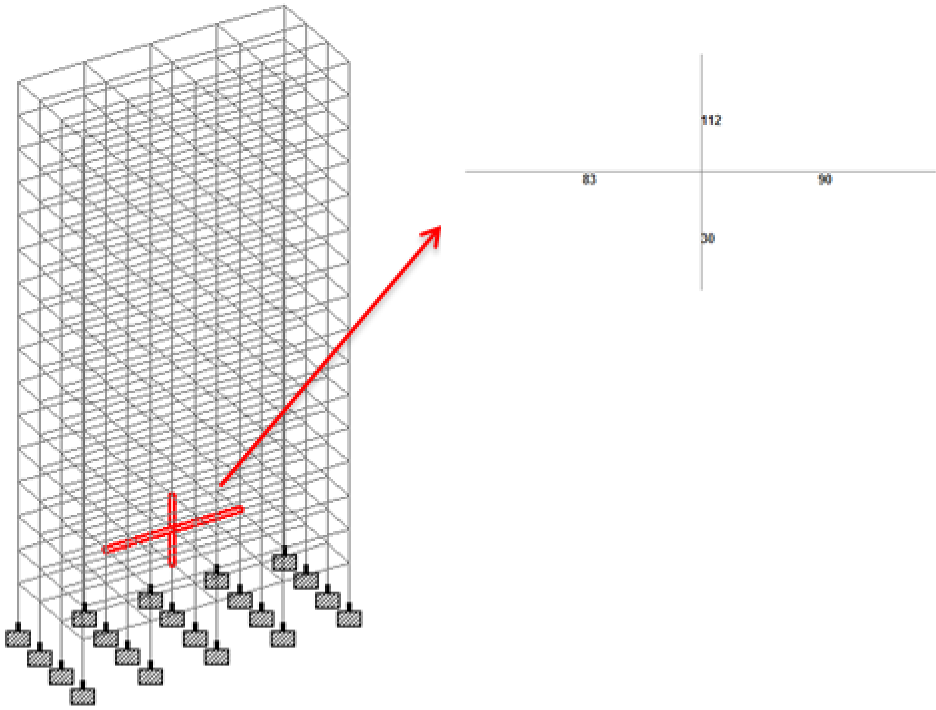


Figure 5.1: Extracted Beam-Column Assembly

Column height is 3.5m and total beam length is 7m. Half of the assembly is modeled in ANSYS as shown in the next section.

5.3 Analysis Procedure in ANSYS Workbench

The geometry can be developed in different ways. One of the way is by using cross-sections available in the software and extruding them as per requirement. Several body operations are also required to be done. Other alternate is to import the geometry from AUTOCAD.

The Figure 5.3 is beam-column assembly for regular building imported from AUTOCAD and few body operations are used to rotate the assembly as per the global axes.

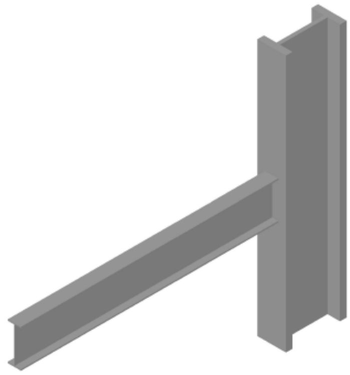


Figure 5.2: Beam-Column Assembly without RBS (Autocad)

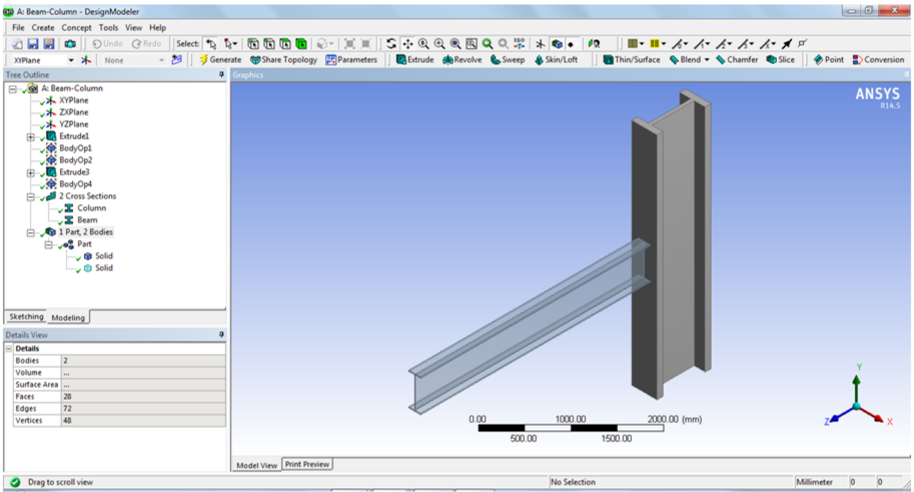


Figure 5.3: Beam-Column Assembly without RBS

Similarly, Beam-Column Assembly with RBS is also imported from AUTOCAD and modeled in ANSYS.

The Connections in the building are designed as fully rigid connections. So, here in Ansys, the connection between the beam and the column is assigned as Bonded connection. It represents fully restrained behavior.

The Meshing in FEM software is an important parameter to be taken care of. The size of the meshing decides the accuracy of the solution. Finer the mesh, more accurate will be the solution. Though, more time is required to get the solution. The Figure 5.4 shows the meshing parameters applied to the model here.

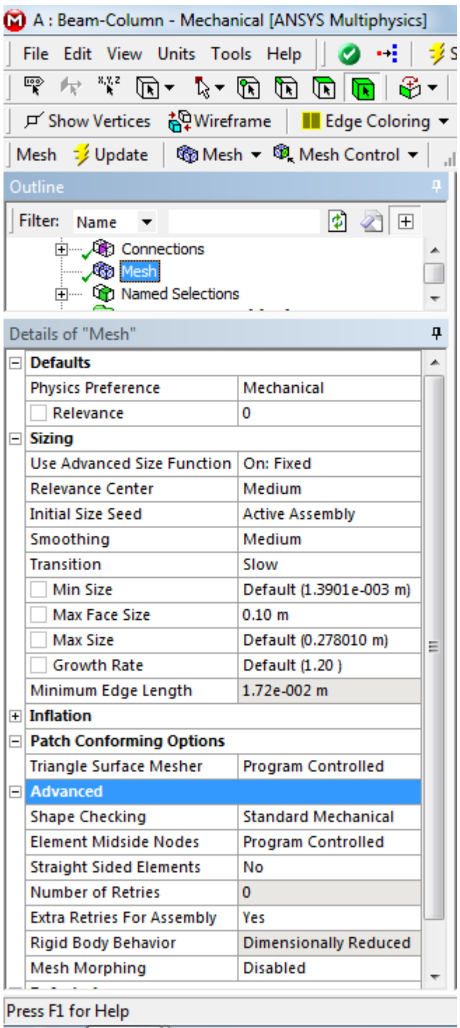


Figure 5.4: Meshing parameters

The meshing, when applied, must be checked for the nodal connectivity between the meshes of both the elements. The node-to-node connectivity ensures the proper distribution of load/stresses from one member to another. For this, the Beam-Column assembly needs to be defined as whole one 'body' with two 'parts'. This will get the proper connectivity of nodes between different elements modeled. As shown in Figure 5.5, the meshing of beam and column has proper node-to-node connection.

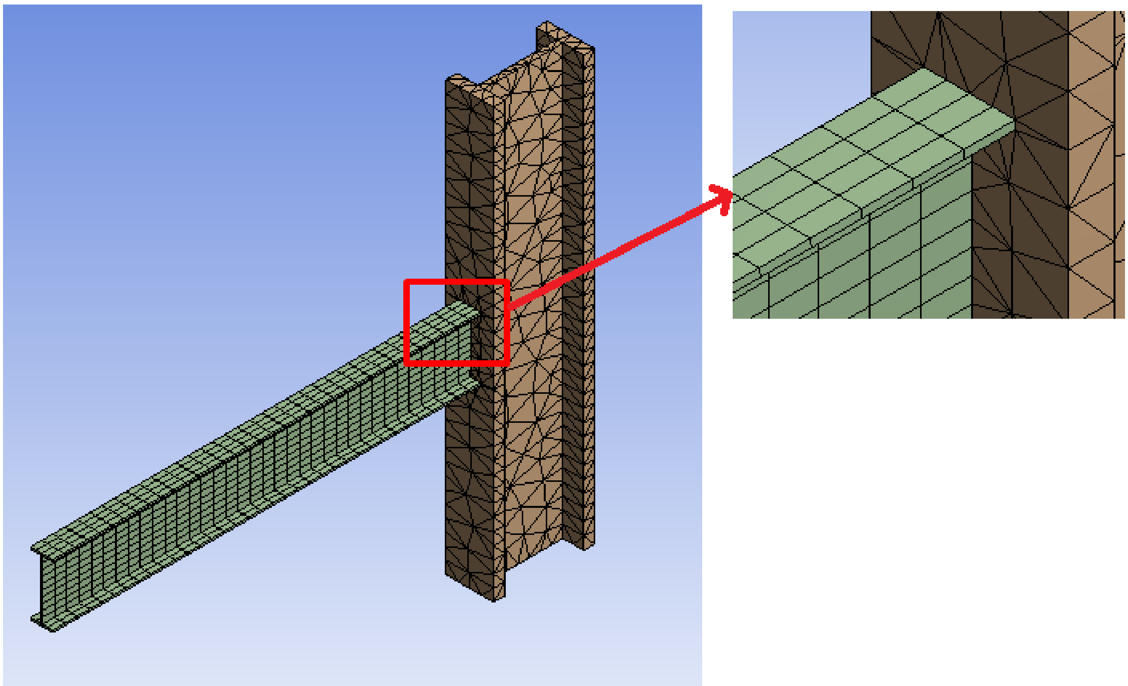


Figure 5.5: Nodal Connectivity

The column ends are assigned fixed support as shown in Figure 5.6. Then, a point load of 10kN is applied at the tip of the cantilever beam as shown in Figure 5.7.

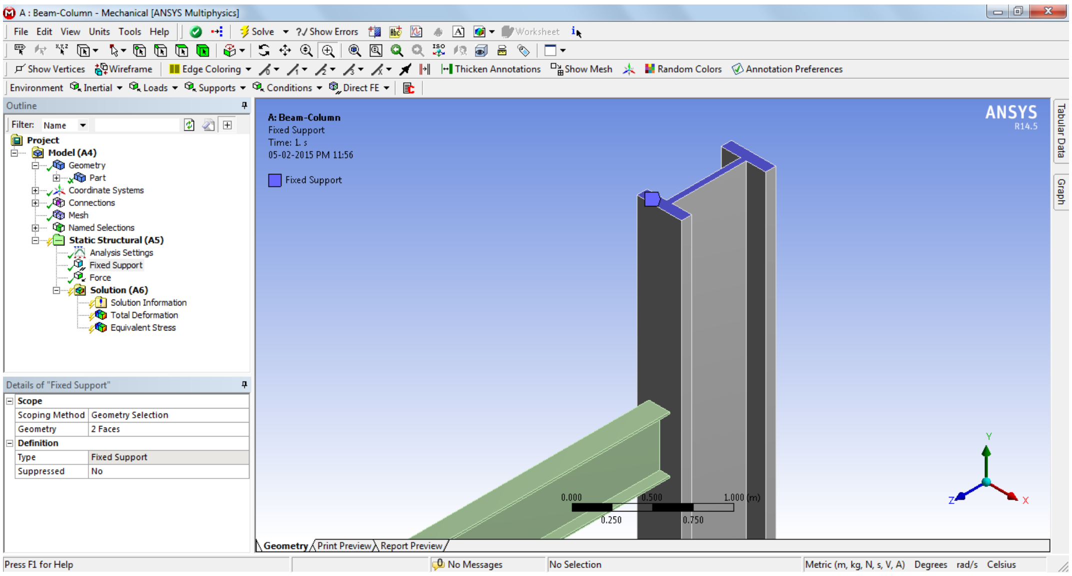


Figure 5.6: Column ends assigned Fixed support

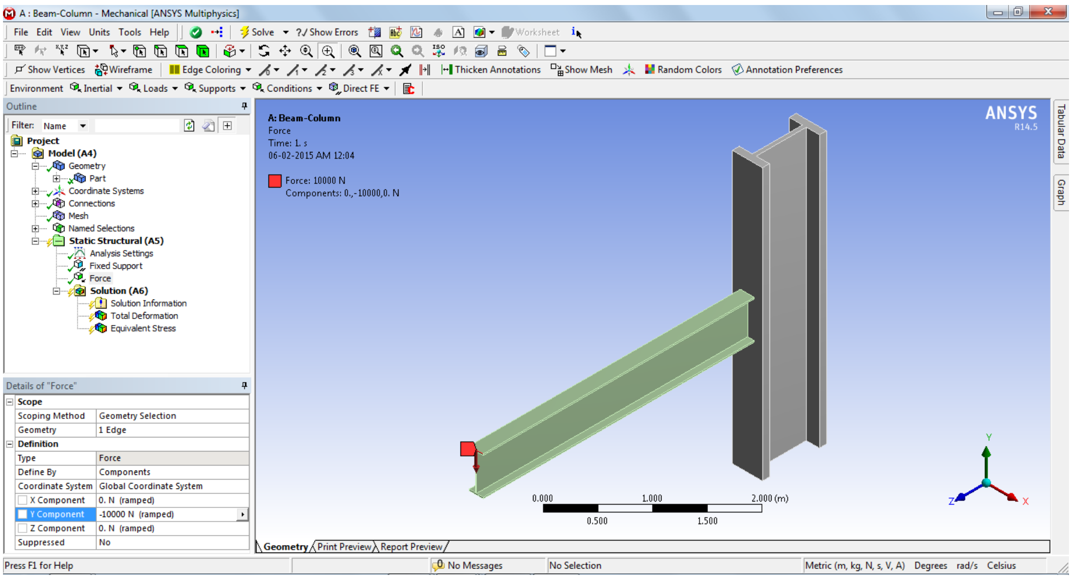


Figure 5.7: Applied Force

After solving, the solution is generated. The elements are automatically assigned to it. As shown below, SOLID186 is assigned to both the elements. The type of element decides the degree of freedom of that element.

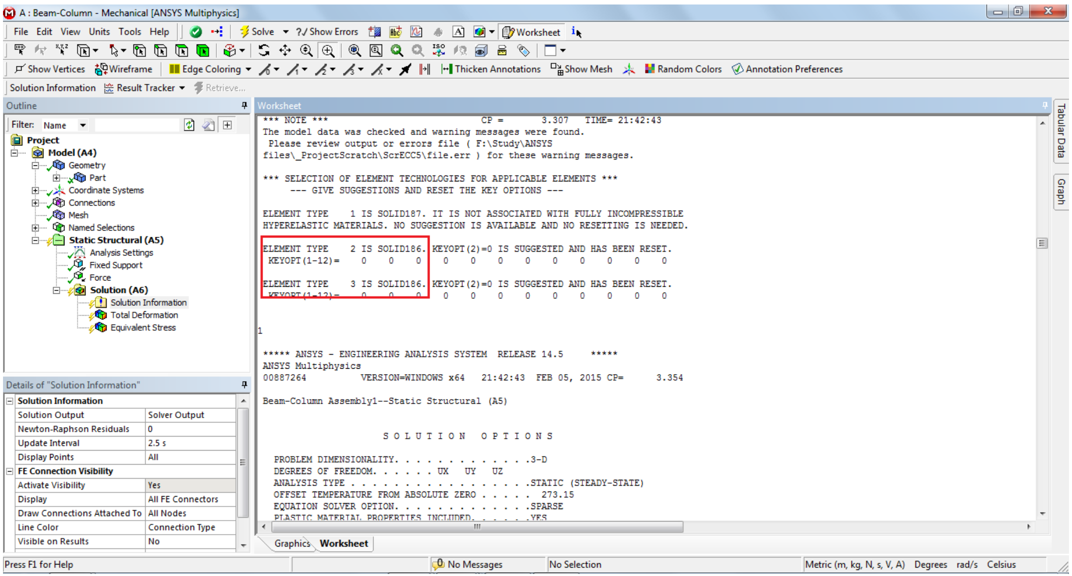


Figure 5.8: Solution Information

SOLID 186 is a 3-D 20-node solid element that has quadratic displacement behavior with each node having three translational degrees of freedom.

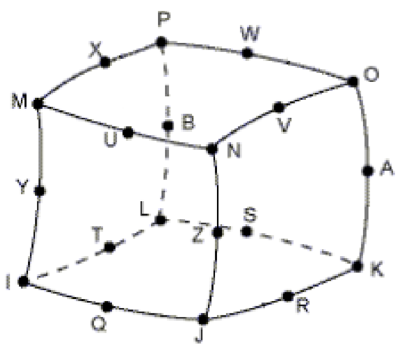


Figure 5.9: Solid186 Element

5.4 Results

5.4.1 Total Deformation

The Deformation is obtained as shown in Figure 5.10 and Figure 5.11 for both the assemblies, with and without RBS. There is an increase in the deformation at the tip of the beam by 12% due to incorporation of RBS.

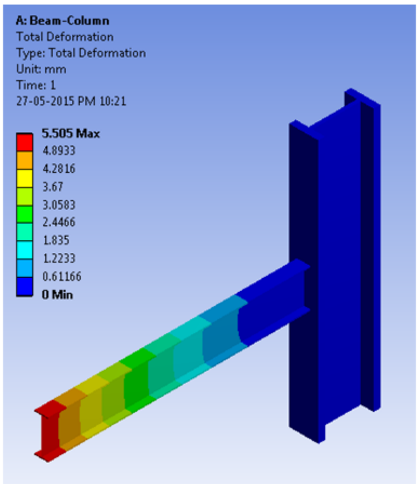


Figure 5.10: Deformation contour for Assembly without RBS

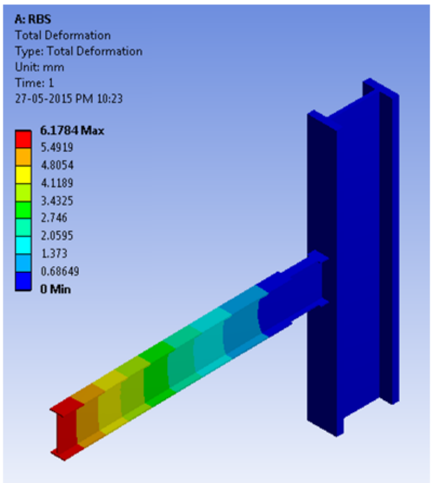


Figure 5.11: Deformation contour for Assembly with RBS

5.4.2 Equivalent Stresses at RBS portion

As seen in the Figures 5.12 and 5.13, there is an increase in the stresses at the RBS portion. The stresses at beam flange increases from 541.01MPa to 819.93MPa.

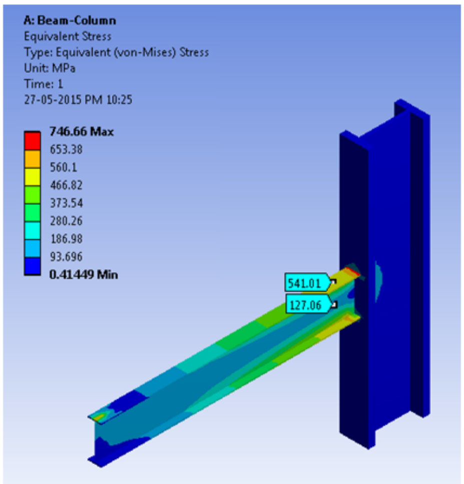


Figure 5.12: Equivalent (Von-Mises) stress contour for Assembly without RBS

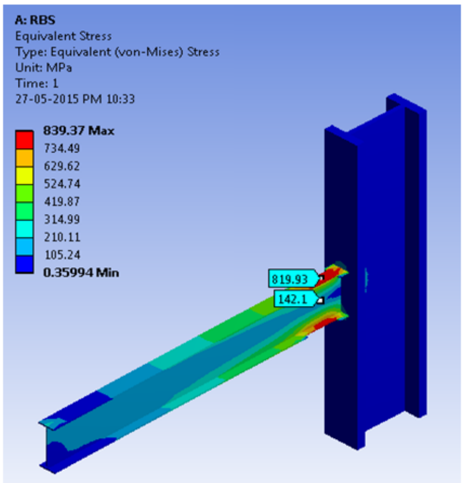


Figure 5.13: Equivalent (Von-Mises) stress contour for Assembly with RBS

5.4.3 Equivalent Stresses at Connection

By introducing RBS, as shown in the Figures 5.14 and 5.15, the stress at the connection between beam-column reduces from 133 MPa to 59.3 MPa. The stresses reduced due to the use of RBS which will help avoid the brittle failure of connection.

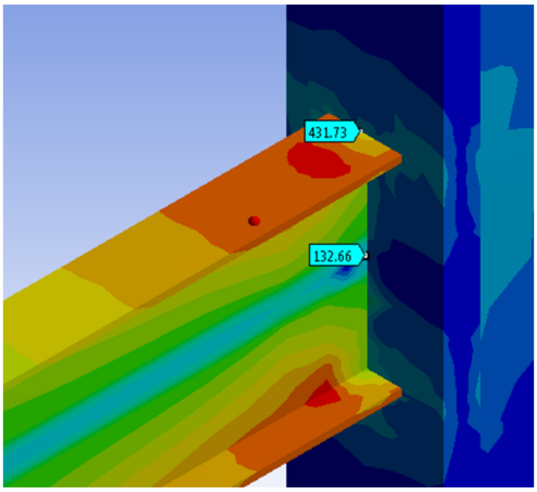


Figure 5.14: Connection of Regular Assembly

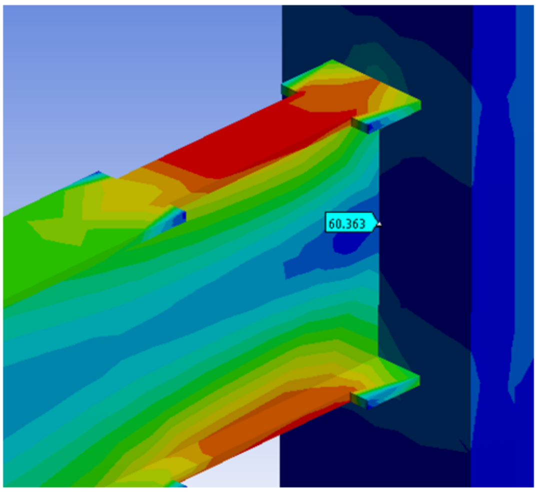


Figure 5.15: Connection of Assembly with RBS

5.4.4 Equivalent Stresses at Panel Zone

The introduction of RBS also shows change in stress distribution at Panel Zone. Lesser stresses are generated when RBS is included compared to the conventional one as can be seen in Figures 5.16 and 5.17

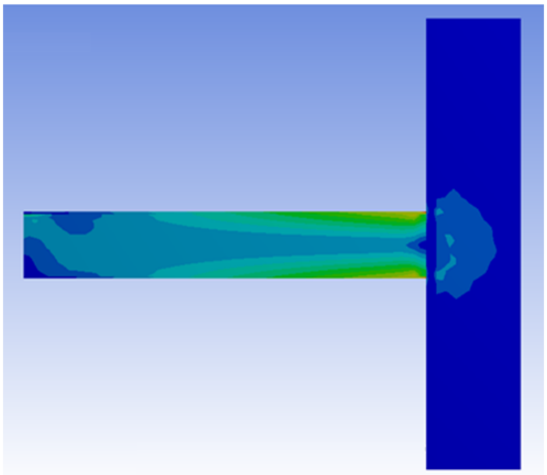


Figure 5.16: Stresses in Panel Zone without RBS

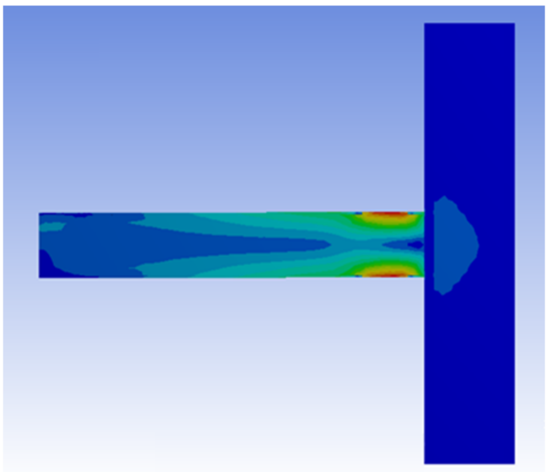


Figure 5.17: Stresses in Panel Zone with RBS

5.5 Summary

In this chapter, the output of FEM analysis is obtained in terms of total deformation and stresses (equivalent Von-Mises) at different locations. The deformation at the tip of the cantilever beam due to application of 10kN point load increases by 12% when RBS is used over the regular beam. Also, when the stresses are compared at locations such as the region of RBS, near connection and Panel Zone, the benefits of using RBS can be observed. The increase in the stresses at RBS is due to the reduction of cross-section. The reduced section attracts more stresses and so the stresses at the connections and the panel zone decreases subsequently.

Chapter 6

Summary and Conclusion

6.1 Summary

In the present study, dynamic analysis of G+15 storey building and FEM analysis of beam-column assembly has been done. The main objective behind this study is to understand, globally as well as locally, the effect of use of Reduced Beam Sections in buildings with Moment Resisting Frames.

Two buildings, with and without Reduced Beam Section, are modeled in STAAD.Pro V8i. The buildings are of 15 storey with steel sections and the load and section details are same for both the buildings. The Reduced Beam Sections are introduced near the beam ends at distance and dimensions specified by FEMA 350 guidelines. The guidelines The building is subjected to highest static earthquake forces. It is then subjected to El Centro NS time history. The results are obtained in form of Time Period, Displacement and Storey Drift. The comparison of these parameters give the effect of RBS over the buildings.

To study the local behavior, the beam-column assembly is subjected to hypothetical point force of 10kN at the tip of the cantilever steel beam connected to a column with fixed connection. The same is done for an assembly with RBS incorporated near the connection. The results are then obtained in the form of Deformation and Stresses at various locations.

6.2 Conclusion

Following are the important conclusions made from the present study:

- The time period of the building increases with the use of RBS. It is because of the change in configuration of beam element. The reduction in cross-section results in increase of the time period of the structure.
- When the deflection of the top storey of the buildings subjected to time history are compared, it is observed that the deflection increases with the use of RBS. Consequently, there is also increase in the Storey Drift.
- Due to the reduction of beam cross section at some locations, there is a decrease in structural steel material. So, there will be a benefit in total cost of material.
- The reduction in the cross section of beam attracts more stresses. The same happens when RBS is incorporated at the beam end near connection. The RBS shows concentration of stresses thereby indicating probable location of plastic hinge formation away from the connection. This prevents the connection from getting over stressed and helps in avoiding hazardous brittle failure.
- The Panel Zone also shows reduction in stress level due to use of RBS. Lesser stresses are transferred to column compared to conventional beams.

6.3 Future Scope of Work

- The frame system of the building can be varied and the variation with use of RBS can be studied.

- The study related to optimum location and number of RBS in a building for a particular geometry can be carried out and carry out the cost analysis.
- The study related to the performance of different type of connections along with RBS can be done by performing FEM analysis.
- The Experimental evaluation of beam-column assemblies with and without RBS subjected to cyclic loading can be carried out to validate the FEM analysis for the same. The hysteresis loop can be generated.
- Also, different type of connections can be incorporated in experimental work and performance of each can be compared.
- The study of Frames with and without RBS can be done experimentally and the various parameters can be compared.

Appendix A

Validation of RBS modeled in STAAD.Pro V8i

As introduced, the basic idea behind using RBS in a moment resisting frame is reducing the stress demand on the beam-column connection. To understand this, two frames are analyzed, one without RBS and one with RBS using Stiffness Matrix Method. The detail of the frames are as below:

- Beam: ISMB 200.
- Column: ISMB 200.
- RBS: ISMB 200 with flange reduced by 50%.
- Load: 10kN/m UDL.

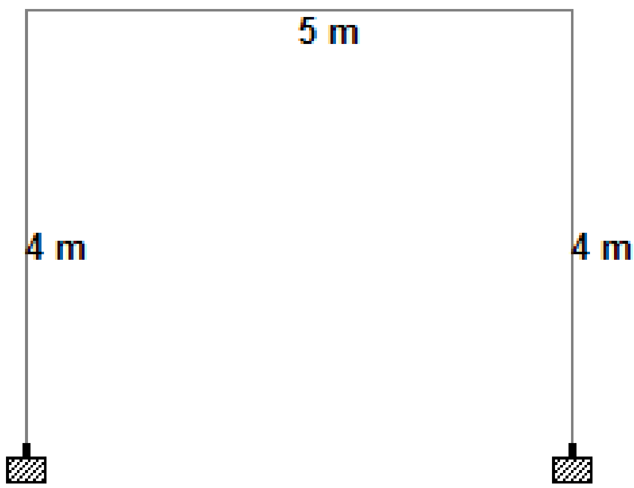


Figure A.1: Frame 1 (w/o RBS)

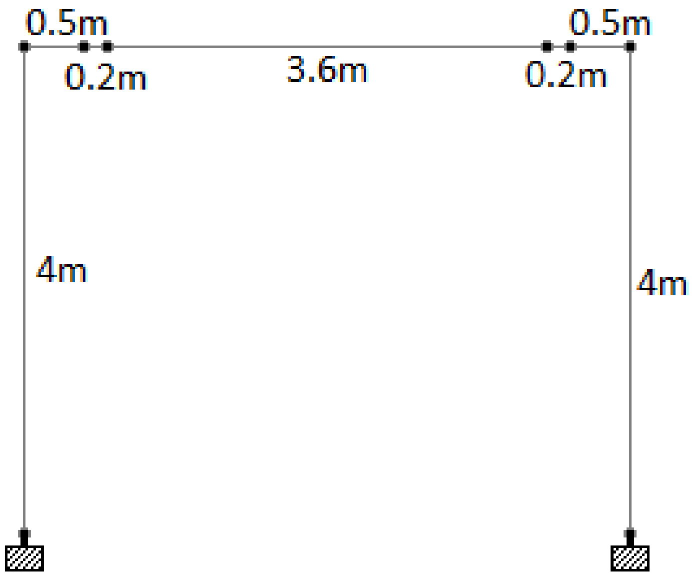


Figure A.2: Frame 2 (with RBS)

For Frame 1,

Stiffness matrix [S] :

$$\begin{bmatrix} 9 & -24 & 4 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ -24 & 936 & 60 & -840 & 84 & 0 & 0 & 0 & 0 & 0 \\ 4 & 60 & 19.2 & -84 & 5.6 & 0 & 0 & 0 & 0 & 0 \\ 0 & -840 & -84 & 840.25 & -83.54 & -0.257 & 0.463 & 0 & 0 & 0 \\ 0 & 84 & 5.6 & -83.54 & 12.31 & -0.463 & 0.55 & 0 & 0 & 0 \\ 0 & 0 & 0 & -0.257 & -0.463 & 840.26 & 83.537 & -840 & 84 & 0 \\ 0 & 0 & 0 & 0.463 & 0.55 & 83.537 & 12.31 & -84 & 5.6 & 0 \\ 0 & 0 & 0 & 0 & 0 & -840 & -84 & 936 & -60 & 24 \\ 0 & 0 & 0 & 0 & 0 & 84 & 5.6 & -60 & 19.2 & 4 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 24 & 4 & 9 \end{bmatrix}$$

$[A_{DL}]$:

$$\begin{bmatrix} -0.21 \\ -3.5 \\ 0.18 \\ -19 \\ -10.77 \\ -19 \\ 10.77 \\ -3.5 \\ -0.18 \\ -0.21 \end{bmatrix}$$

[S,inv] :

$$\begin{bmatrix}
0.6043 & 0.2548 & 0.4191 & 0.3265 & 0.2994 & 0.1114 & -0.193 & 0.0748 & -0.172 & -0.123 \\
0.2548 & 0.1397 & 0.265 & 0.1852 & 0.1908 & 0.0675 & -0.117 & 0.0454 & -0.104 & -0.075 \\
0.4191 & 0.265 & 0.647 & 0.3766 & 0.4712 & 0.1547 & -0.267 & 0.104 & -0.238 & -0.171 \\
0.3265 & 0.1852 & 0.3766 & 0.2539 & 0.2993 & 0.1005 & -0.174 & 0.0675 & -0.155 & -0.111 \\
0.2994 & 0.1908 & 0.4712 & 0.2993 & 0.6158 & 0.1734 & -0.297 & 0.1168 & -0.267 & -0.193 \\
0.1114 & 0.0675 & 0.1547 & 0.1005 & 0.1734 & 0.2532 & -0.298 & 0.1847 & -0.376 & -0.326 \\
-0.193 & -0.117 & -0.267 & -0.174 & -0.297 & -0.298 & 0.6146 & -0.19 & 0.4697 & 0.2982 \\
0.0748 & 0.0454 & 0.104 & 0.0675 & 0.1168 & 0.1847 & -0.19 & 0.1393 & -0.264 & -0.254 \\
-0.172 & -0.104 & -0.238 & -0.155 & -0.267 & -0.376 & 0.4697 & -0.264 & 0.6454 & 0.4177 \\
-0.123 & -0.075 & -0.171 & -0.111 & -0.193 & -0.326 & 0.2982 & -0.254 & 0.4177 & 0.6032
\end{bmatrix}$$

[D] :

$$\begin{bmatrix}
-14.77 \\
-8.736 \\
-19.23 \\
-12.66 \\
-19.78 \\
-12.54 \\
19.707 \\
-8.641 \\
19.087 \\
14.535
\end{bmatrix}$$

$[A_{MD}]$:

$$\begin{bmatrix}
-0.375 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0.5 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0.375 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
24 & -96 & 24 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
8 & -24 & 4 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
-24 & 96 & -24 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
4 & -24 & 8 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 840 & 84 & -840 & 84 & 0 & 0 & 0 & 0 & 0 \\
0 & 84 & 11.2 & -84 & 5.6 & 0 & 0 & 0 & 0 & 0 \\
0 & -840 & -84 & 840 & -84 & 0 & 0 & 0 & 0 & 0 \\
0 & 84 & 5.6 & -84 & 11.2 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0.257 & 0.463 & -0.257 & 0.463 & 0 & 0 & 0 \\
0 & 0 & 0 & 0.463 & 1.11 & -0.463 & 0.55 & 0 & 0 & 0 \\
0 & 0 & 0 & -0.257 & -0.463 & 0.257 & -0.463 & 0 & 0 & 0 \\
0 & 0 & 0 & 0.463 & 0.55 & -0.463 & 1.11 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 840 & 84 & -840 & 84 & 0 \\
0 & 0 & 0 & 0 & 0 & 84 & 11.2 & -84 & 5.6 & 0 \\
0 & 0 & 0 & 0 & 0 & -840 & -84 & 840 & -84 & 0 \\
0 & 0 & 0 & 0 & 0 & 84 & 5.6 & -84 & 11.2 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 96 & 24 & 24 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 24 & 8 & 4 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & -96 & -24 & -24 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 24 & 4 & 8 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0.375 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & -0.375 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0.5
\end{bmatrix}$$

$[A_{ML}]$:

$$\begin{bmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 2.5 \\ 0.21 \\ 2.5 \\ -0.21 \\ 1 \\ 0.03 \\ 1 \\ -0.03 \\ 18 \\ 10.8 \\ 18 \\ -10.8 \\ 1 \\ 0.03 \\ 1 \\ -0.03 \\ 2.5 \\ 0.21 \\ 2.5 \\ -0.21 \\ 0 \\ 0 \\ 0 \\ 0 \end{bmatrix}$$

Now, $[A_M] = [A_M] + [A_{MD}] \times [D] =$

$$\begin{bmatrix} 5.5397 \\ -7.386 \\ -5.54 \\ -14.77 \\ 25.083 \\ 14.773 \\ -20.08 \\ -3.481 \\ 20.083 \\ 3.4811 \\ -18.08 \\ 0.3356 \\ 17.935 \\ -0.374 \\ 18.065 \\ 0.1403 \\ -18.06 \\ -0.14 \\ 20.065 \\ -3.673 \\ -20.06 \\ 3.6726 \\ 25.065 \\ -14.96 \\ 5.4507 \\ 14.535 \\ -5.451 \\ 7.2675 \end{bmatrix}$$

The same frame has been modeled in STAAD.Pro V8i and results are obtained as shown in Figure A.3. The values obtained from Stiffness Matrix Analysis fairly matches with these results.

The frame w/o RBS is also modeled and results are obtained as shown in Figure A.4. The well known fact that the end moments are reduced by incorporating RBS is reflected from the figure A.3 and figure A.4.

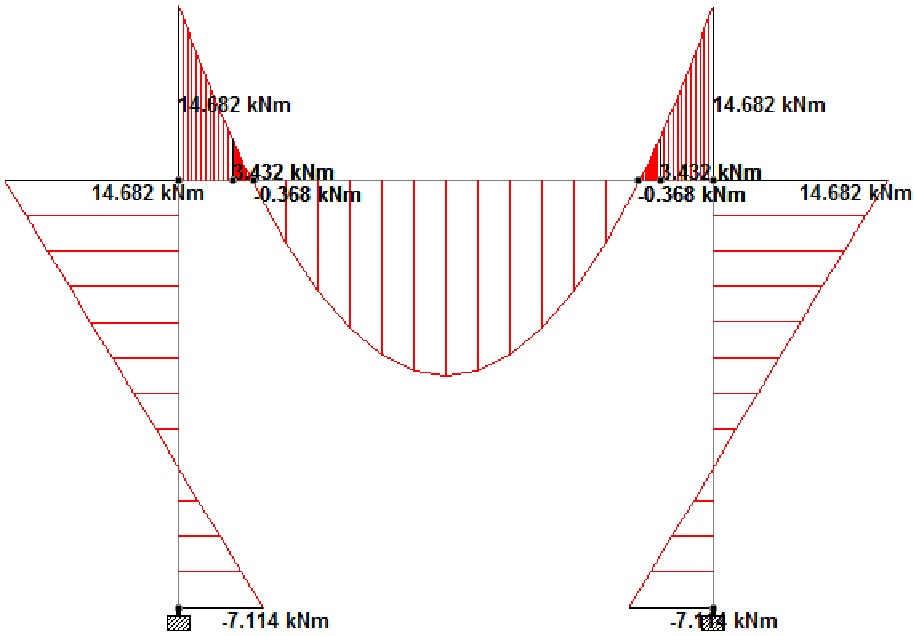


Figure A.3: Frame with RBS - Mz

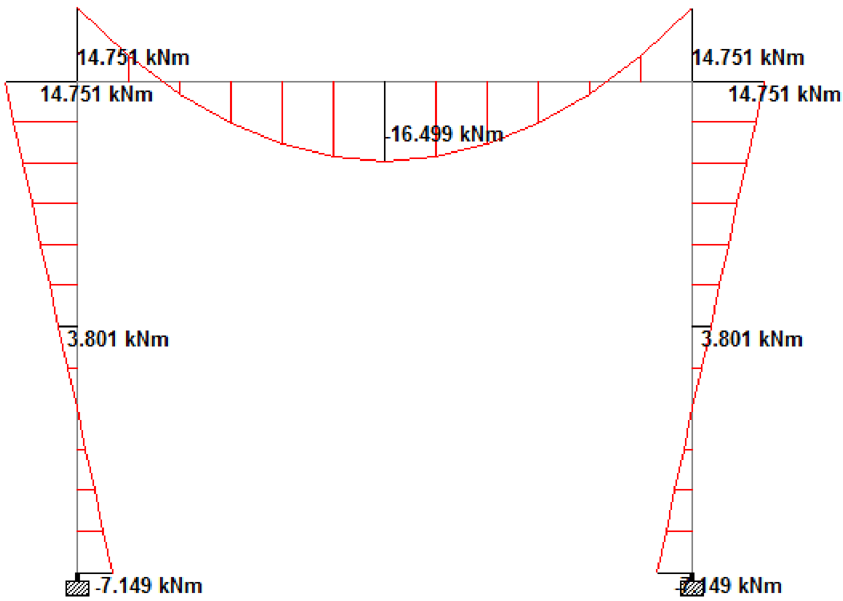


Figure A.4: Frame w/o RBS - Mz

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