RESEARCH ARTICLE OPEN ACCESS

Study of Steel Moment Resisting Frame with Reduced Beam Section

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ABSTRACT
A research became necessary after the collapse of steel structure during the 1994 Northridge and 1995 Kobe earthquakes. Reduced beam section emerged as one of the best solution. Guidelines about the cut, that is to be introduced in the flange of the beam section, are obtained from FEMA 350. Here, a G+15 storey steel building is modeled using RBS as a component in one building and regular beam section as a component of the other in STAAD PRO V8i. Time history analysis is carried out in this paper. Displacement, storey drift, time period and base shear of both the buildings are compared as the result. Base shear shows no change but considerable change in displacement and storey drift is observed.
Keywords: *FEMA 350, MR frames, Reduced beam section, STAAD pro v8i, Time history analysis*

I. INTRODUCTION

During the 1994 Northridge earthquake, the bolted web-welded flange moment connections in steel moment-resisting frames suffered unexpected brittle failures in and near the heat-affected zones [1]. A lot of damage of lives and property was observed during this earthquake. Many industrial steel buildings were severely damaged during this havoc. Many modifications have been proposed for post Northridge earthquake new construction and retrofit of steel moment frames [1].

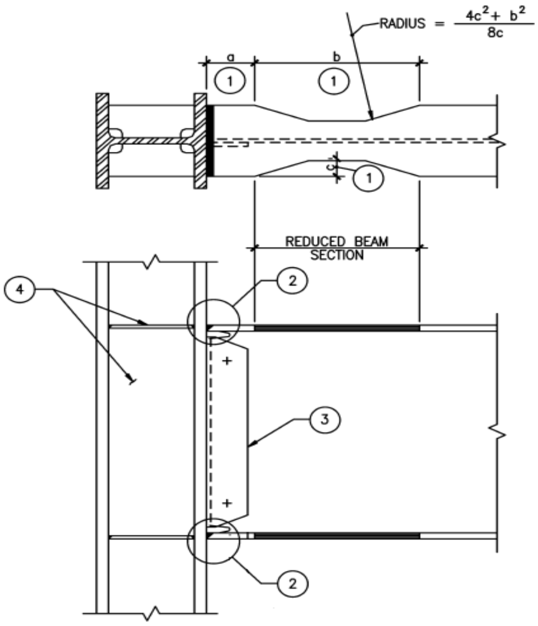
Many of the recommendations in FEMA 350 have since found their way into the Seismic Provisions for Structural Steel Buildings published by the American Institute of Steel Construction (AISC) [2]. The most commonly observed damage occurred in or near the welded joint of the bottom flange of a girder to the supporting flange of column. All of the connections approved in for Steel Moment Resisting Frames combined improvements in welding along with detailing that induce the beam plastic hinge to form a short distance away from the beam-to-column interface. The type of detailing that shifts the plastic hinges away from the connection region generally falls into two main categories, reinforcement detailing and reduced beam section detailing.

The Reduced Beam Section (RBS) is one of the ways to weaken the beam framing to the column. However the typology and technology of the RBS beam to column connection, as well as its behavior under cyclic loading conditions were investigated in USA after the Northridge earthquake. Reduced beam section connections provide similar benefits to reinforced connections, but are more efficient and economical because they do not require the extra field welding and material associated with reinforced connections. RBS connections also have a number of advantages in design practice. Compared to reinforced connections, their use leads to reduced demands for continuity plates, panel zone reinforcement, and strong column–weak-beam requirements [2].

Here, a G+15 storey steel building is analyzed with and without RBS. Time history dynamic analysis is carried out and thus base shear, displacement, storey drift and time period are compared of both the buildings, thus giving the advantages of building with RBS over building without rbs.

II. GUIDELINES FOR DESIGN OF REDUCED BEAM SECTION

The guidelines for deciding the dimensions and design of RBS based on the shear and flexure parameters have been given in FEMA-350. The guideline provided in this document is for design of fully restrained Reduced Beam Section (RBS) connection. Figure 3.1 provides typical details for such connections. When connection with RBS is used, the elastic drift calculations should be considered 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

Fig.1: Detailing of reduced beam section

Design steps for the suitable dimension of RBS

Step 1:

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

$a = (0.5 – 0.75)\, b_f$ (1)

$b = (0.65 – 0.85)\, d_b$ (2)

where a and b are as shown in Fig 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:

- Assume $c = 0.20b_f$
- Calculate Z_{RBS}
- Calculate M_f using $C_{pr} = 1.15$
- If $M_f<C_{pr}, R_y, Z_b, F_y$ the design is acceptable. If M_f is greater than the limit, increase c. The value of c should not exceed $0.25b_f$.

where

B_f = Width of Flange

Z_{RBS} = Plastic section modulus of RBS

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

F_y = Yield stress for steel

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.

Step 4:

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

$V_f = 2M_f / (L – d_c) + V_g$ (3)

where

Vg = 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.

Step 7:

Check continuity plate requirements according to the methods of Section 3.3.3.1 of FEMA 350.

Step 8:

Detail the connection.

III. ANALYSIS AND DESIGN

3.1 Building details

The building details are as follows:

Bottom storey height = 4.2m

Typical storey height = 3.5m

Bay dimensions - 4m x 7m

Properties of steel used:

Modulus of elasticity = 2 × 105 MPa

Poisson's ratio = 0.3

Density = 7833.41 kg/m^3

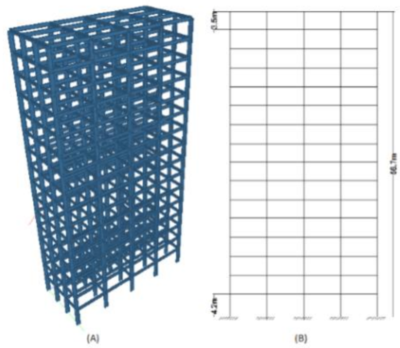


Fig 2. (A) Rendered View (B) Elevation

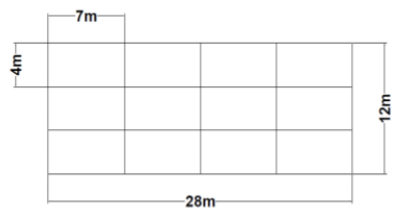


Fig 3. Plan of the building

3.2 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 do 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 1.

a = $0.6b_f = 0.6 \times 210 = 126$mm
b = $0.75d_b = 0.75 \times 600 = 450$mm
c = $0.2b_f = 0.2 \times 210 = 42$mm

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

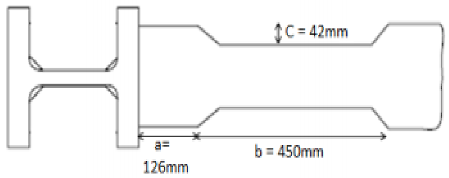


Fig 4.RBS detail

Table 1.Properties of Steel Sections

Properties	Column (mm)	Beam (mm)	RBS (mm)
Depth	738.87	600	600
Flange width	354.58	210	126
Flange thickness	83.06	20.8	20.8
Web thickness	46	12	12

3.3 Load cases and combination

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

A. 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.

B. 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.

C. 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 5

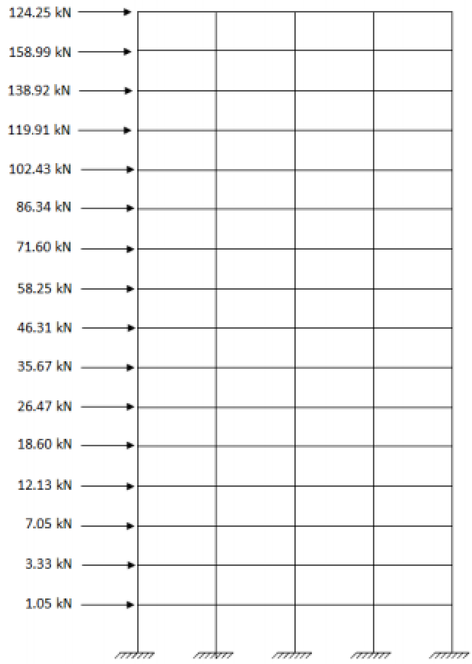


Fig 5.Forces on Each Storey (EQX)

D. 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 2.

Table 2.Wind Intensities

Intensities (kN/m2)	Height (m)
0	0
0.8	10
0.9	15
0.98	20
1.093	30
1.234	50
1.383	100

3.4 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 6.

The Peak Ground Acceleration of this excitation is 0.32g.

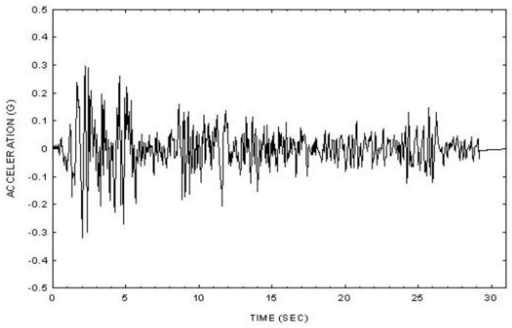


Fig 6.EL Centro NS Excitation

As shown in Figure 6, the EL CENTRO 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.

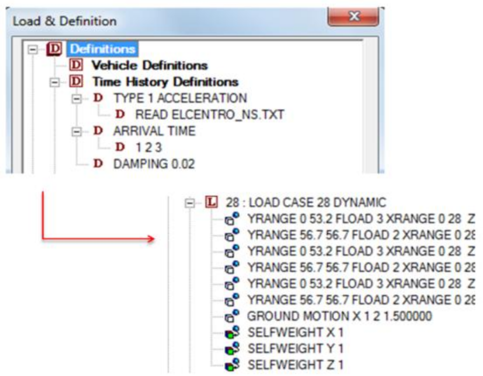


Fig 7.Time History Definition in STAAD Pro V8i

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 numbers 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 show similar amount of displacement.

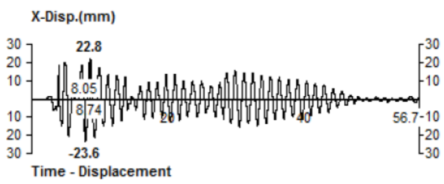


Fig 8.Time – Displacement Plot

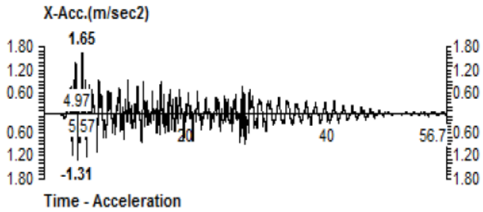


Fig 9.Time – Acceleration Plot

3.5 Results

By incorporation of RBS, the time period of the building changes. The program calculated time period of both the buildings has been found. Also, the base shear is obtained. The time period and base shear of the buildings are as below and the displacement of each storey of both the buildings are noted and graph of no. of storeys vs. displacement and storey drift has been plotted.

Table 3.Time period and base shear of the building

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

Table 4.Displacement and storey drifts

	Displacement mm		Drift (mm)	
Storey	Regular	RBS	Regular	RBS
16	231.319	284.235	2.795	4.581
15	228.524	279.653	3.619	6.180
14	224.904	273.473	5.012	8.123
13	219.891	265.350	6.896	10.035
12	212.994	225.315	9.221	12.007
11	203.773	243.307	11.791	14.185
10	191.981	229.122	14.469	16.539
9	177.512	212.582	17.118	18.962
8	160.393	193.62	19.492	21.278
7	140.901	172.342	21.582	23.426
6	119.319	148.915	23.20	25.427
5	96.118	123.488	24.132	27.114
4	71.985	96.373	23.779	28.193
3	48.206	68.179	20.993	27.86
2	27.212	40.319	16.657	24.603
1	10.556	15.715	10.555	15.715
0	0	0	0	0

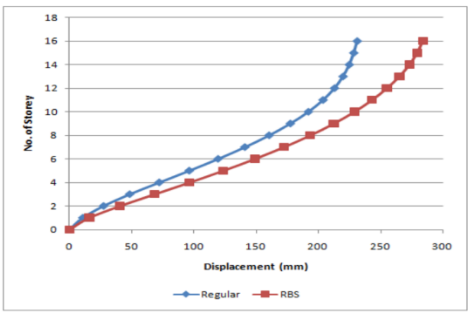


Fig 10.Displacement Plot

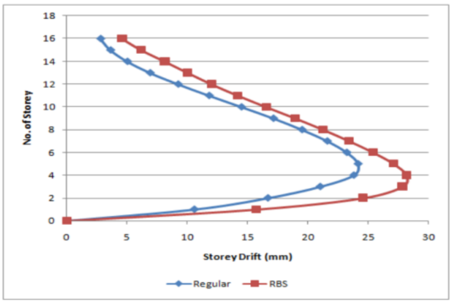


Fig 11.Storey Drift Plot

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 5 summarizes the weights of elements in both the buildings.

Table 5.Material Weight

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

The difference in weight = 112.92 kN = 11.51 ton.
Considering cost of steel as Rs.65/kg
Cost Benefit = Rs.65000 × 11.51 ton = Rs.7.48 lakhs

IV. CONCLUSION

Following are the important conclusions made from the present study:

- 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.
- 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.
- There is also a considerable amount of increase in the storey drift by incorporating RBS.
- The difference between base shear of the building with RBS and without RBS is almost negligible.
- 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.

REFERENCES

Journal Papers:

[1] J.shen, T. Kitjasateanphun, W. Srivanich, “Seismic performance of steel moment frames with reduced beam sections”, Engineering Structures 22, pp 968-983, 2000

[2] Jun Jin, Sherif El-Tawil, “Seismic Performance of Steel Frames with reduced beam section connections”, Journal of Constructional Steel Research 61, pp 452-471, 2005

[3] Swati Ajay Kulkarni, Gaurang Vesmala,“ Study of Steel Moment connection with and without reduced beam section”, Case Studies in Structural Engineering 1, pp 26-31, 2014

[4] Scott M. Adan, Lawrence D. Reaveley,“The Reduced Beam Section Moment Connection without Continuity Plates”,13th World Conference on Earthquake, 2000

[5] Brandon Chi, Chia-Ming Uang, “Cyclic response and design recommendations of reduced beam section moment connections with deep columns”, Journal of Structural Engineering, 0733-9445 ,pp 464-472,2002

[6] Scott L jones, Gary T fry, “Reduced beam section welded steel moment frames”, 13th World Conference on Earthquake, pp 1671, 2000

[7] K.Kildashti, R.Mirghaderi, I.M.Kani ,“The efficiency of reduced beam section connections for reducing residual drifts in MR Frames”, Open Journal of Civil Engineering, 2, pp 68-72, 2012

[8] C.E.Sofias, C.N.Kalfas, D.T.Pachoumis, “Experimental and FEM analysis of reduced beam section moment endplate connections under cyclic loading”, Engineering Structures, 59,pp 320-329,2014

[9] IS 800-2007, General construction in steel-code of practice

[10] FEMA 350, Recommenced seismic design criteria for new steel moment frame buildings