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Nonlinear Response of Reduced Beam Section Moment Connections with Box Columns

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Abstract: Before the Northridge earthquake, the flexural frames with welded flange-bolted web connections were used in numerous steel structures as a ductile and appropriate earthquake resisting system. During the 1994 Northridge and the 1995 Kobe earthquakes, which occurred one year later, many moment frames were deteriorated in the beam to column joint region. After earthquake investigations showed that concentration of stress at the location of beam to column connection was the main reason for failure, and subsequently, some methods were offered to mitigate the problem. One of the proposed solutions was a reduction of beam section near the column, which shifts the stress concentration far from critical region. In this paper the nonlinear response of moment connection with a reduction in beam section, and with the box column sections has been studied, and the stress distribution near the column have been compared for both welded and Reduced Beam Section (RBS) moment connections. Studying the results, showed that the reduced beam section connections exhibit a desirable response in comparing with the common connections. RBS force the beams plastification away from the column face and avoids stress concentrations and large deformation in the column. Furthermore, in this research some recommendations on the appropriate geometric parameter such as the connecting gap from the column edge, the length of the reduced section of beam flange, and the depth of reduced section of beam flange are provided.

Keywords: Nonlinear Analysis; Seismic Behavior; Steel Moment Frame; Plastic Hinge; Cyclic Response; Northridge Earthquake, Kobe Earthquake

1. INTRODUCTION

Investigation on damages imposed on buildings after the Northridge and Kobe earthquakes showed that most of the damage in steel moment resisting frames initiated from the bottom of the lower flange of beams [1-3] while mostly the upper flange was in good conditions because of the composite behavior between steel beam and concrete slab. The concentration of stress in the lower flange led to the failure of the connection in the event of an earthquake. Several solutions to fix the problem were suggested in literature.

Reduced Beam Section (RBS) connections are among the solutions that have been shown to exhibit satisfactory levels of ductility in numerous experimental tests [4-7]. In RBS connection, a portion of the flange in a beam is removed at a particular radius. The radius depends on the geometrical shape of the section. The moment capacity of reduced beam section should be greater than the required moment under combination of seismic and gravity loads. By doing this, in case of an earthquake, the plastic hinge in the beam is forced away from the beam-column intersection. As results, a higher ductility and seismic performance can be achieved [8-13].

Although an extensive body of knowledge exists on the performance of the RBS connection systems, the recommendation for the geometry of the connections design and verifying code recommendations for of seismic parameters such as deflection amplification factor, response modification factor, ductility, and over strength factor need more research [14-17]. This lack would become a major significance especially for the RBS connection system that is linked to box columns, since most of the previous studies were conducted on the connection systems linked to I-column sections.

Since the framed structures are usually designed in a way that some of their components enter into the nonlinear region during severe earthquake excitations to attenuate the imposed earthquake energy, it is of importance to evaluate and study the structural nonlinearity through numerical modeling. Different nonlinear analysis methods such as pushover, incremental dynamic [18-25], and cyclic analysis have been utilized to investigate the seismic behavior of structures in global scale e.g. an entire building frame or local scale e.g. structural connections. In this study, the nonlinear response of mentioned moment connections has been studied by using finite element method analysis. For this purpose, a numerical model of the proposed connection was developed by using an application that has been used previously by other researchers in this field [7]. In this model, it is assumed that beam-column connections are welded. The response of RBS connection has been compared with common connection systems, and then the design parameters for this connection system has been investigated.

2. MODELING

2.1 Materials and Loading

All steel sections and elements were made up of ST37 grade steel in accordance with DIN 17100:1966-09 [26], and the stress-strain curve of steel was considered as a bilinear curve resembling an actual stress-strain curve of steel. The cyclic loading pattern has been used to control the displacement up to 15 centimeters with gradual increasing in 2.5-centimeter steps [7].

2.2 Specification of Specimens

As shown in Figure 1, IPE30 sections were selected as beam sections and the columns are chosen from box sections with dimensions of 30 x 30 centimeters with a wall thickness of 2 centimeters. For the design of RBS connection, three significant geometric parameters of connecting gap from the column edge (a), the length of the reduced section of beam flange (b), and the depth of reduced section (c) as shown in Figure 1 are considered. For each parameter, a group of specimens has been considered and different values are considered for each parameter as shown in the Table 1 to 3. The parameters are defined concerning the section properties so they can be potentially extended to other connections with different dimensions. Parameters "a" and "c" are defined as a portion of flange width (b_f) and parameter b is defined as a portion of section depth (d). The specimen indicated by MA1 was selected for comparing RBS connection with the connections that were common before 1994 with no reduction in section.

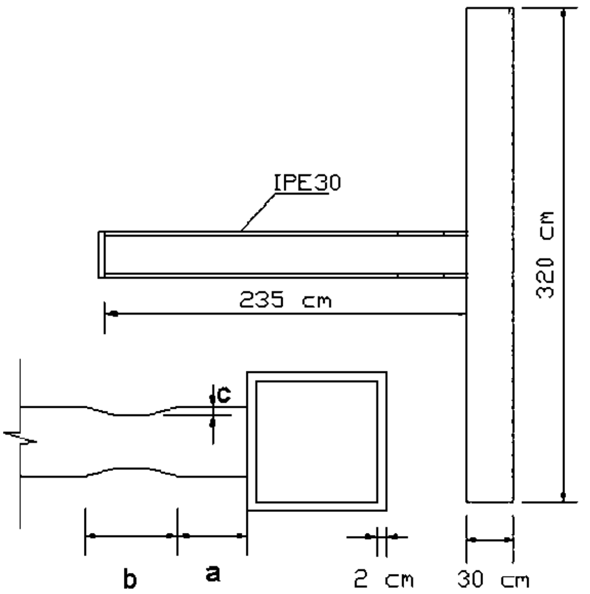


Figure 1: Geometry of RBS connection used in the analytical

Table 1: Connection specifications to determine optimized "a" parameter.

Specimen	Column	Beam	a	b	c
MA1	30×30×2×2	IPE30	-	-	-
MA2	30×30×2×2	IPE30	0.4 b_f	0.75 d	0.2 b_f
MA3	30×30×2×2	IPE30	0.5 b_f	0.75 d	0.2 b_f
MA4	30×30×2×2	IPE30	0.6 b_f	0.75 d	0.2 b_f
MA5	30×30×2×2	IPE30	0.75 b_f	0.75 d	0.2 b_f
MA6	30×30×2×2	IPE30	0.8 b_f	0.75 d	0.2 b_f
MA7	30×30×2×2	IPE30	0.9 b_f	0.75 d	0.2 b_f

Table 2: Connection specifications to determine optimized "b" parameter.

Specimen	Column	Beam	a	b	c
MB1	30×30×2×2	IPE30	0.6 b_f	0.55 d	0.2 b_f
MB2	30×30×2×2	IPE30	0.6 b_f	0.65 d	0.2 b_f
MB3	30×30×2×2	IPE30	0.6 b_f	0.75 d	0.2 b_f
MB4	30×30×2×2	IPE30	0.6 b_f	0.85 d	0.2 b_f
MB5	30×30×2×2	IPE30	0.6 b_f	1.0 d	0.2 b_f
MB6	30×30×2×2	IPE30	0.6 b_f	1.1 d	0.2 b_f
MB7	30×30×2×2	IPE30	0.6 b_f	1.2 d	0.2 b_f

Table 3: Connection specifications to determine optimized "c" parameter.

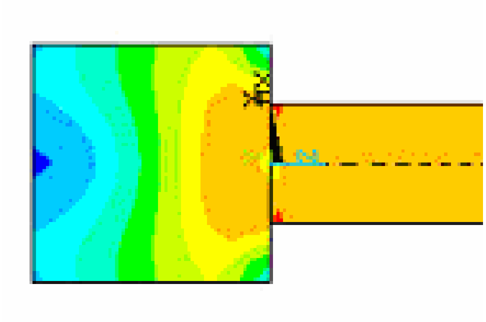
Specimen	Column	Beam	a	b	c
MC1	30×30×2×2	IPE30	0.6 b_f	0.75 d	0.2 b_f
MC2	30×30×2×2	IPE30	0.6 b_f	0.75 d	0.25 b_f
MC3	30×30×2×2	IPE30	0.6 b_f	0.75 d	0.3 b_f

model.

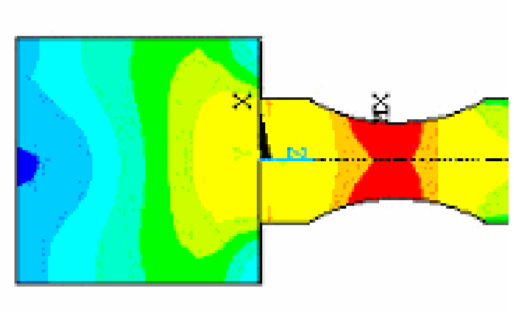
3. RESULTS

3.1 Comparing RBS and Ordinary Connections

Analysis of the MA1 specimen; the connection without any reduction in section; shows that the maximum stress has occurred just beside the column, which is not favorable (see Figure 2a). To make a comparison, specimen indicated by MA4 is selected from the group of RBS connection, which is considered as a RBS connection representative. In MA4 connection, the maximum stress has occurred at the reduced section of the beam, which is desired (see Figure 2b). In addition, despite the reduction of beams flanges, the load-displacement hysteretic curve has a stable state (see Figure 3).



(a)



(b)

Figure 2: (a) MA1 stress contour, (b) MA4 stress contour.

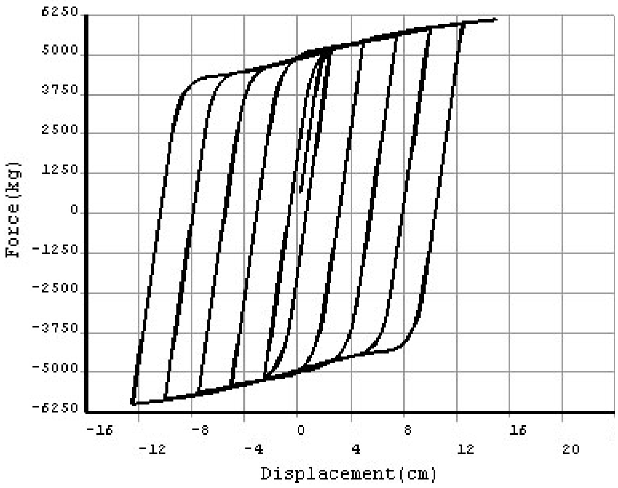
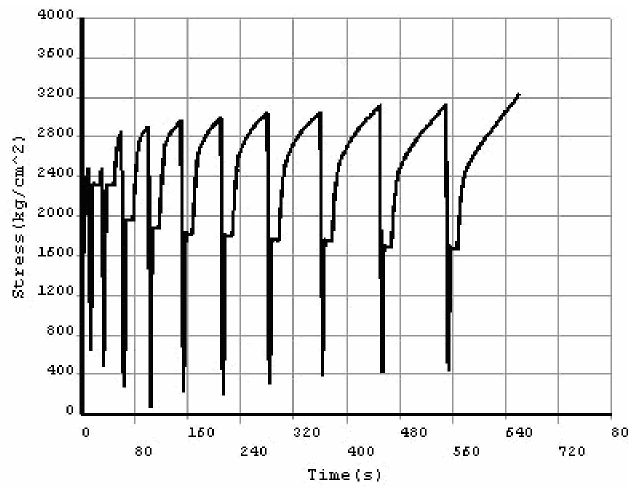
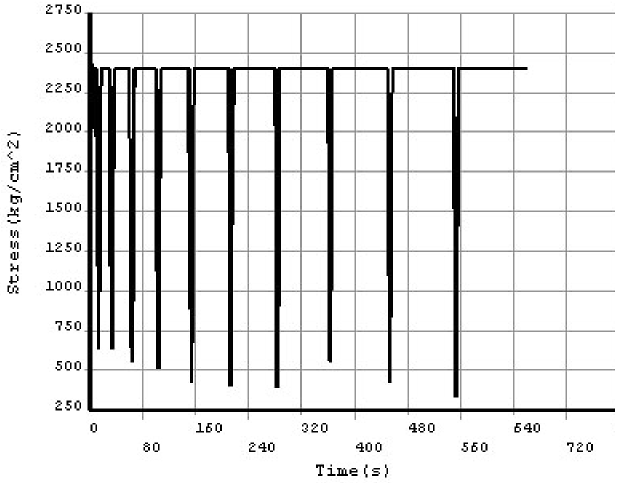


Figure 3: MA2 load-displacement curve.



(a)



(b)

Figure 4: (a) MA1 stress-time curve, (b) MA2 stress-time curve.

Stress distributions just at beside columns differ in these two specimens. The stress-time curves for these 0two specimens have been shown in Figures 4a and 4b. Note that the time has a virtual concept in this analysis. For more comparisons for these two specimens, the stress distribution within the width of the beam flange has been shown in Figure 5.

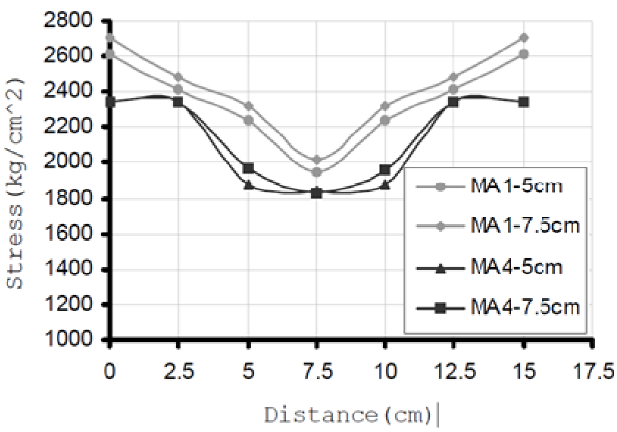
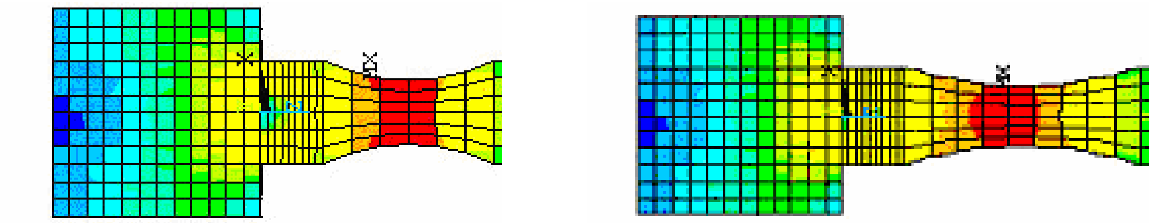


Figure 5: Stress distribution within the width of the beam flange.

(b)

Figure 6: (a) MA2 stress-time curve, (b) MA7 stress-time curve.

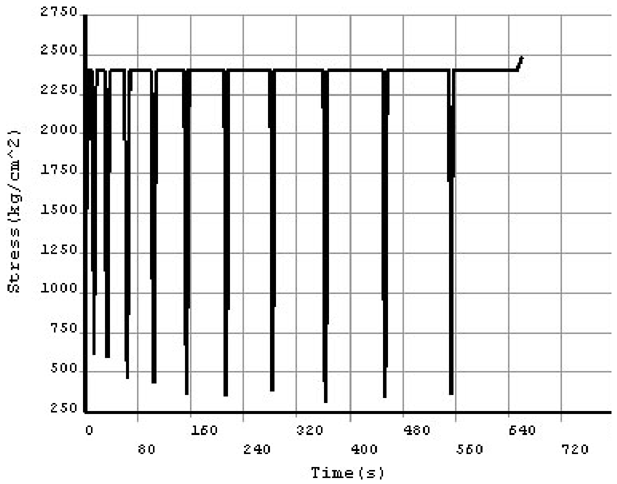


(a) (b)

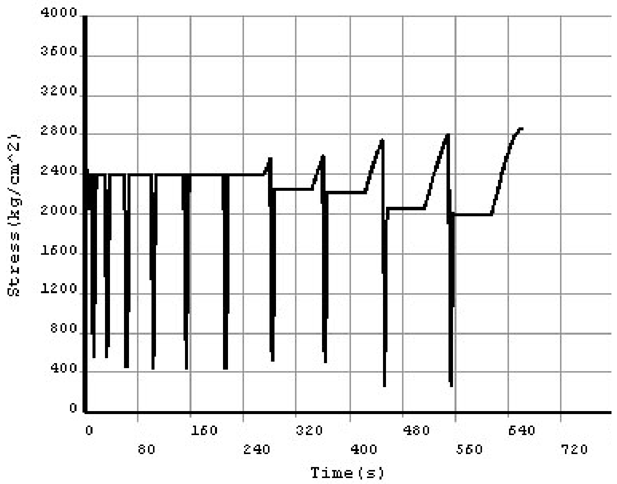
Figure 8: (a) MB5 stress counter, and (b) MB7 stress counter.

3.2 Determining Range of Parameter "a"

From the specimens in Table 1, appropriate values of parameter "a" is obtained. MA2 and MA7 results have been shown in Figures 6a and b. Results for MA3, MA5, and MA6 showed that their treatments similar to MA4. There is a concentration of stress just at beside of the column in MA2 and MA7. To more comparison of the response of specimens, the maximum absorbed strain energy and maximum absorbed plastic energy in width of one flange has been shown in Figure 7a and b.

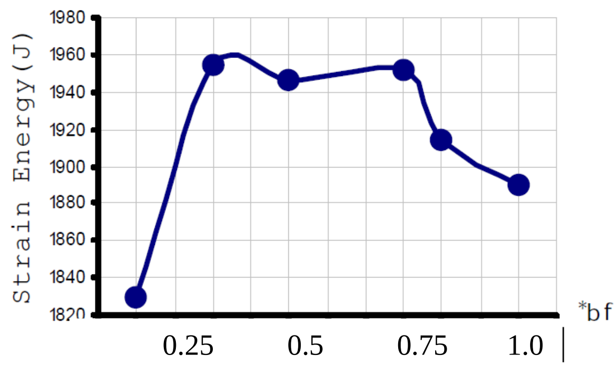


(a)

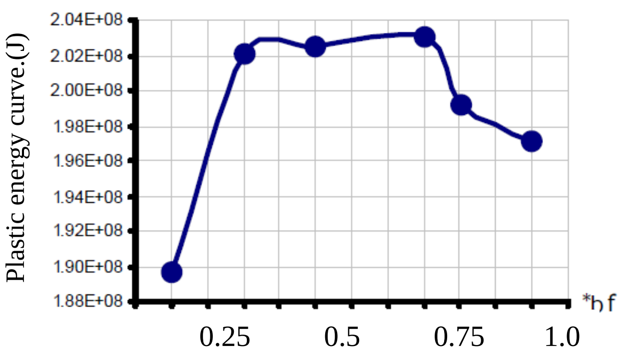


3.3 Determining Range of Parameter "b"

For investigating the length of the reduced section of beam flange (parameter "b"), the specimens of Table 2 have been analyzed. There was a concentration of stress just at beside of column at the ending stage of loading in MB1 (see Figure 2a). MB2 and MB5 had similar and desirable responses. The curve load-displacement and stress distribution just beside the column for MB2 has been shown in Figures 8b and c, but the separation of plastic hinges in MB6 and MB7 have a fundamental difference with specimen MB2 and MB5s. The plastic hinge is nonsymmetrical corresponding to the middle of the reduced part, Figure 9.



(a)



(b)

Figure 7: (a) Strain energy curve, and (b) Plastic energy curve.

For more comparison of the response of specimens, the maximum absorbed strain energy and maximum absorbed plastic energy in the flange width has been shown in Figure 10. The differences of these curves with Figures 8 and 9 are

For more comparison of the response of specimens, the maximum absorbed strain energy and maximum absorbed plastic energy in the flange width has been shown in Figure 10. The differences of these curves with Figures 8 and 9 are

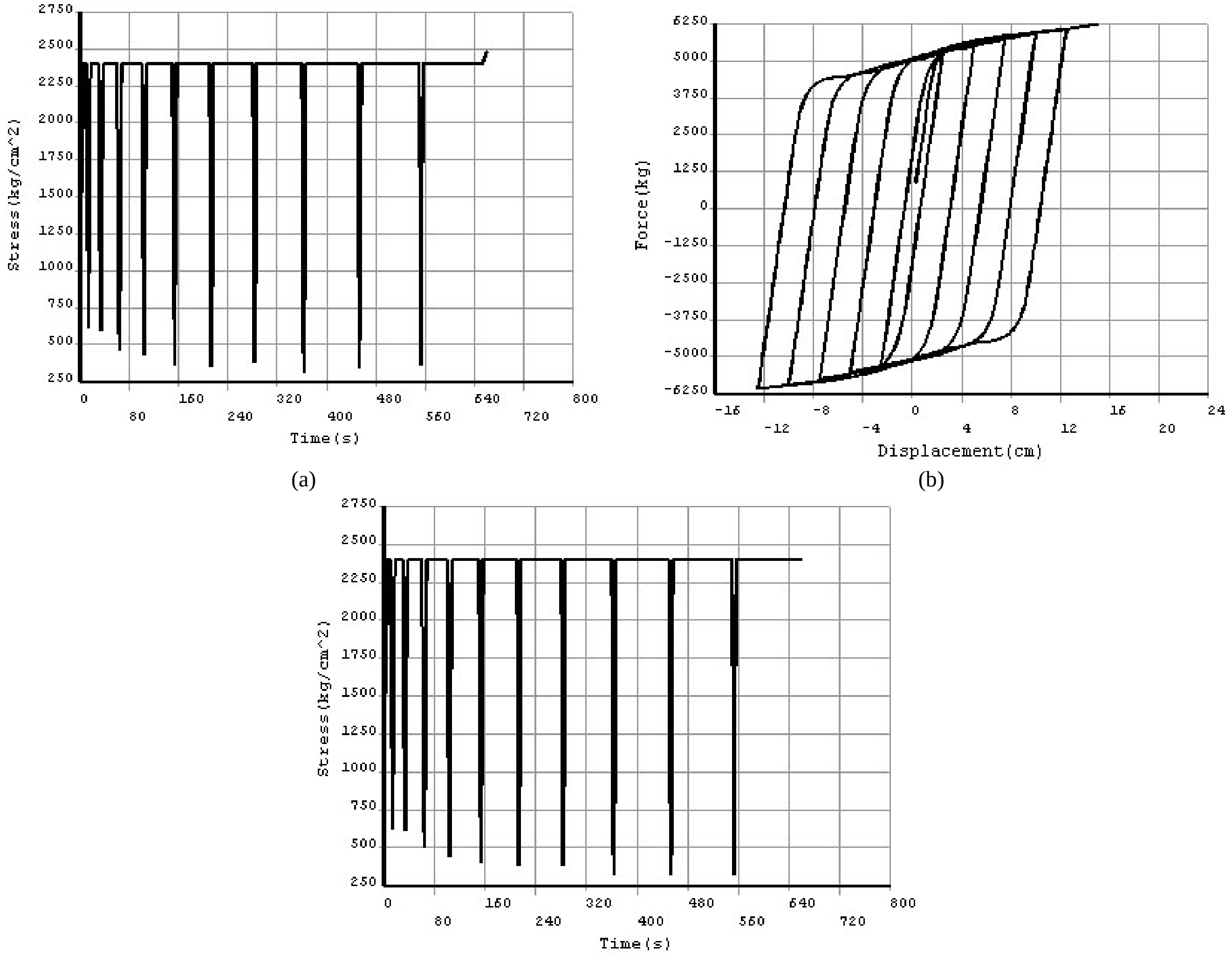


Figure 9: (a) MB1 stress-time curve, (b) MB2 load-displacement curve, and (c) MB2 stress-time curve.

due to their element size.

due to their element size.

3.4 Determine the range of parameter "c"

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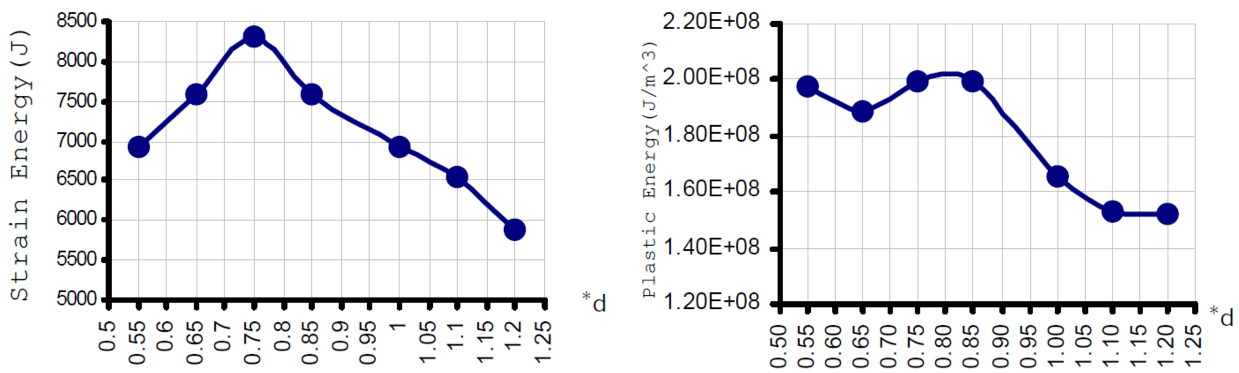
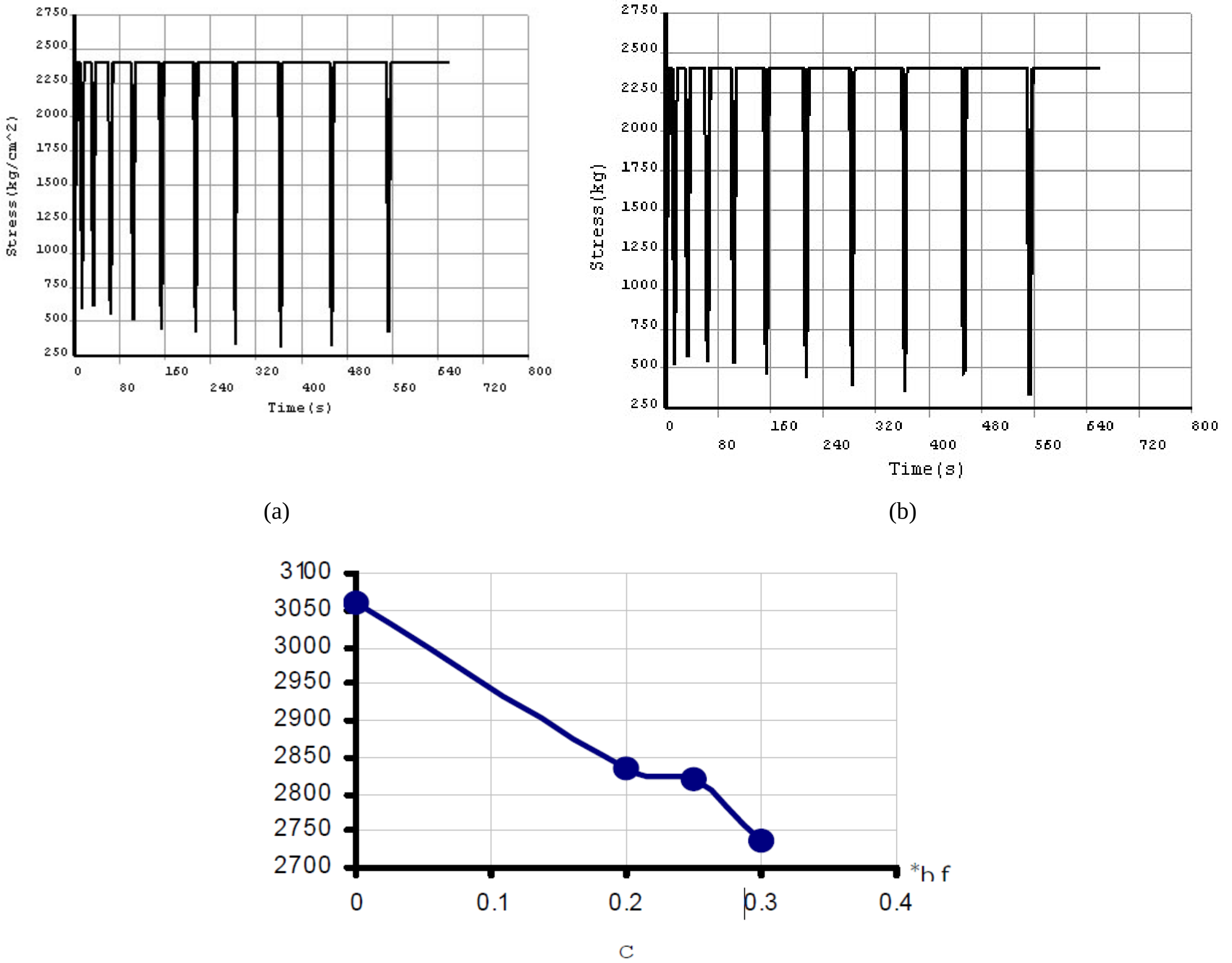


Figure 10: (a) Strain-energy curve, and (b) Plastic energy curve.



(a) (b)

(c)

Figure 11: (a) MC2 stress-time curve, (b) MC3 stress-time curve, and (c) stiffness versus c parameter.

To investigate the depth of reduced section of beam flange (parameter “c”), the specimens of Table 3 have been analyzed. The analysis of MC1 showed that it was similar to MB3 (explained in section 3.3). For MC2 and MC3, stress-time curves have been shown in Figures 11a and b. Although in these two cases, the stress concentration did not occur at just beside the column, the system stiffness decreased considerably. For more comparison, the connection system stiffness has been shown for various values of c in Figure 11c.

4. CONCLUSION

The nonlinear response of RBS moment connections, with the column sections of the box, has been studied, and the stress distribution just near the column have been compared for the both welded and Reduced Beam Section (RBS) moment connections. The following conclusions were made:

1. The RBS connections have a desirable response in comparing with the common connections were utilized before 1994 Northridge earthquake. Using RBS connections, the problem of stress concentration at just beside of column can be mitigated.

2. Based on the analysis results on the geometric parameters of the RBS connections, the appropriate ranges for each parameter were determined. The gap from the column edge (“a”) should be in range of 0.5 b_f =< a =< o.8 b_f, the length of reduced section of beam flange (“b”) should be in range of 0.65d =< b =< d, and the depth of reduced section (“c”) should be in range of C =< 0.25 b_f. It is worth noting to mention that the section with a = 0.75 b_f and b=0.75d showed the best seismic performance.

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