INELASTIC PANEL ZONE DEFORMATION DEMANDS IN STEEL MOMENT RESISTING FRAMES

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ABSTRACT

INELASTIC PANEL ZONE DEFORMATION DEMANDS IN STEEL MOMENT RESISTING FRAMES

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Panel zone is one of the significant parts of beam-column connections in steel structures. Until the 1994 Northridge Earthquake, a few experimental research and parametric studies had been carried out to understand the behavior of the panel zones. However, after the Northridge Earthquake, it was observed that beam-column connections were unable to show presumed seismic performance. Therefore, current design codes needed to be revised to improve seismic performance of connections in general and panel zones in particular. In this research, panel zone deformation demands are examined using explicit three dimensional finite element models and considering different parameters. For this purpose, a frame model with two different beam-column configurations was developed in order to observe the effects of beam depth, the axial load level and the level of seismicity. The frame models were analyzed under twenty different ground motion records. Local strain demands at the panel zones as well as the global frame deformation demands are evaluated. Analysis results revealed that AISC Specification designs allowed panel zone yielding; however, panel zones designed according to FEMA 355D showed minimal yielding for both shallow and deep beam configurations. Based on the analysis results, local shear strain demands in panel zones were expressed as a function of interstory drifts and normalized panel zone thicknesses.

Keywords: Steel Frames, Panel Zone, Finite Element Analysis, Seismic Design, Beam Depth

ÖZ

MOMENT AKTARAN ÇELİK ÇERÇEVELERDE İNELASTİK KAYMA BÖLGESİ DEFORMASYON TALEBLERİ

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Çelik yapılarda kayma bölgesi kolon kiriş bağlantılarında önem teşkil eden elemanlardan biridir. 1994 Northridge depremine kadar, kayma bölgesi davranışını anlamak için birkaç deneysel ve parametrik çalışmalar yapılmıştır. Fakat, Northridge depreminden sonra, kolon kiriş bağlantılarının öngörülen sismik performansı gösteremedikleri gözlemlenmiştir. Bundan dolayı, mevcut yapı şartnamelerinin genel olarak bağlantı elemanlarının ve özellikle kayma bölgelerinin sismik performansını geliştirmek için revize edilmesine ihtiyaç duyulmuştur. Bu çalışmada, belirgin üç boyutlu sonlu eleman modeli kullanarak ve farklı değişkenler altında kayma bölgeleri deformasyon talepleri incelenmiştir. Bu doğrultuda, kiriş derinliği, eksenel yük seviyesi ve depremsellik etkilerini gözlemlemek için iki farklı kolon kiriş yapılandırılmasına sahip bir çerçeve sistemi oluşturulmuştur. Çerçeve modeli yirmi farklı yer hareketi kaydı kullanılarak zaman tanım aralığı analizlerine tabii tutulmuştur. Kayma bölgelerindeki bölgesel gerilme talepleri yanı sıra genel çerçeve deformasyon talepleri değerlendirilmiştir. Analiz sonuçları şunu göstermiştir ki AISC şartnamesi kayma bölgesinin akmasına izin verirken, FEMA 355D kriterlerine göre dizayn edilmiş kayma bölgeleri hem kısa hem derin kiriş konfigürasyonunda en az akma değerleri göstermiştir. Analiz sonuçlarına dayanarak, kayma bölgerindeki bölgesel kesme gerilmesi talepleri katlar arası ötelenme ve birimlenmiş kayma bölgesi kalınlığının fonksiyonu olarak ifade edilmiştir.

Anahtar Kelimeler: Çelik Çerçeveler, Kayma Bölgesi, Sonlu Elemanlar Analizi, Sismik Dizayn, Kiriş Derinliği

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CHAPTER 1

INTRODUCTION

1.1 Description of a Panel Zone

Panel zone is the region in the beam column connections which is bounded by the column flanges and continuity plates as shown in Figure 1.1. During a seismic event panel zones can undergo inelastic deformations which can participate into the energy dissipation capacity of moment resisting frames. The influence of the panel zone contribution to the inelastic response became more important after the 1994 Northridge Earthquake. The observed damage and extensive deformations in the panel zones led to changes in the capacity calculations of the panel zones in design provisions.

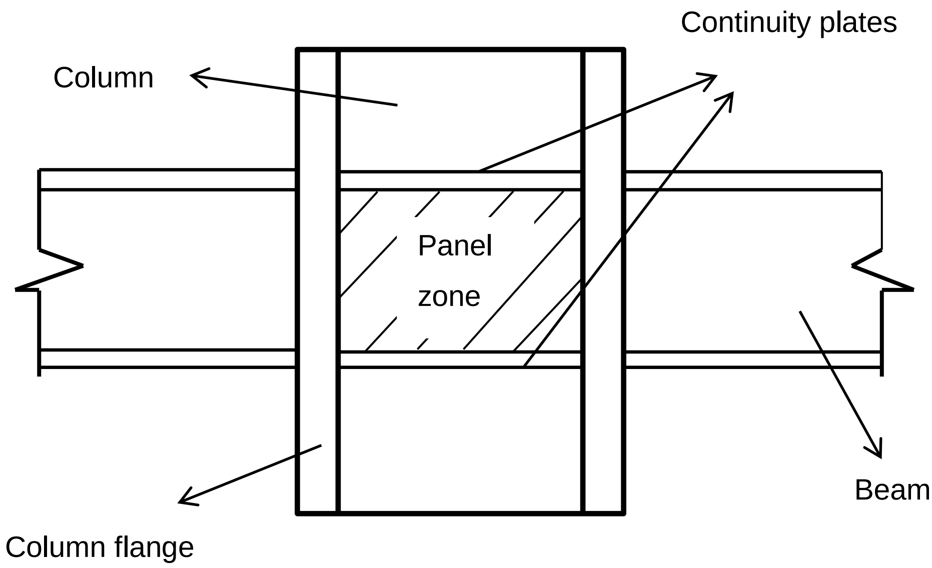

Figure 1.1 Panel Zone Region

1.2 Importance and Behavior of Panel Zones

Steel moment resisting frames are designed to exhibit inelastic behavior during seismic events. Majority of the yielding is expected to take place at beam ends by plastic hinge formation. This yielding mechanism is controlled by applying weak beam strong column concept at the design stage. Panel zones are subjected to very high shear forces which can potentially cause yielding in these regions. In fact moderate yielding of the panel zones participates to the energy dissipation capacity and often times be desirable. On the other hand, inadequate design of panel zones can lead to large inelastic deformations which cause undesirable connection behavior as shown in Figure 1.2. This was a typical observation after the 1994 Northridge earthquake and fracture formation at panel zones initiated research projects to investigate connection behavior in general and panel zone behavior in particular. In a satisfactory panel zone design the strength and stiffness loss should be averted while allowing for some level of inelastic action that contributes to energy dissipation capacity.

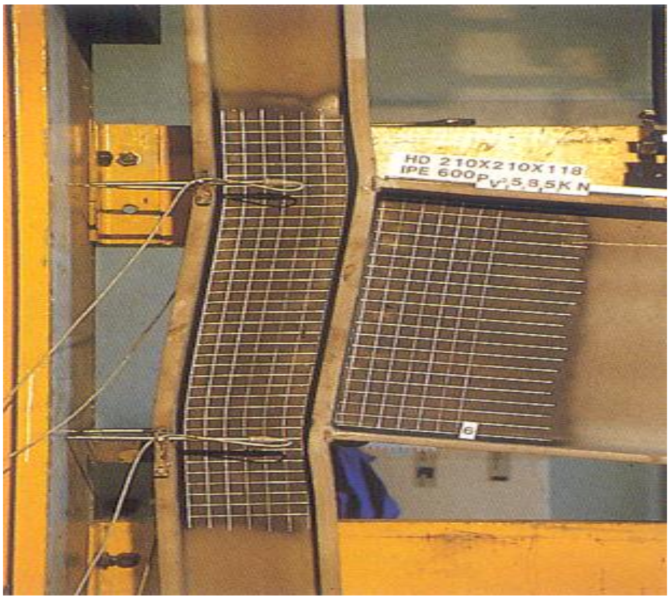

Figure 1.2 Excessive Joint Distortion (Adopted from AISC)

1.3 Background

A thorough literature survey on panel zone has been performed as a part of this thesis. Experimental studies and numerical studies are presented separately for clarity.

1.3.1 Experimental Studies

Bertero (1968) stated that there were insufficient data to predict the inelastic behavior of the multistory steel framed structures under dynamic actions. In the light of the test results carried out in Japan, Bertero concluded that it was necessary to investigate the panel zone deformations for predicting the behavior of a structure. He suggested a testing program which covers beam,column and panel zone and their contribution to the inelastic deformations. A subassemblage that represents the interior beam-to-column connection was selected. Gravity loads on the beams and axial force in the column were applied to the test set-up simultaneously. The loading program was applied quasi-statically in order to make the rate of strain high enough to introduce significant variations. Bertero drew conclusions from the available test data as the panel zone yielding led to a decrease in the subassamblage stiffness and, although, reinforcing of the panel zone increased the yielding and ultimate strength of this region, there was a reduction in stiffness under cyclic loading.

Krawinkler H., et al., (1971) conducted experimental and analytical studies on beam-to-column sub-assemblages to obtain quantitative information regarding the lateral stiffness-story displacement relationships for unbraced steel frames. For this purpose, four specimens, two of which are from identical shapes were prepared to simulate a typical upper story and a lower story of the 20 story 4 bay office building. The samples have been tested under both vertical and lateral forces. To simulate the characteristic action of a severe earthquake, quasi-static loading had been applied. In the light of the experiment results, Krawinkler, et al. concluded the following;

- Connections should be designed to provide a balance between the inelastic deformations taking place in connection and beam. It would lead too large demand on beam rotation capacities. Designing the panel zone rigid in order to behave elastic under severe earthquake is not advantageous.

- The connections showed a large reverse strength beyond yielding.

- The panel zone distortions had to be limited to a ductility ratio of six to avoid local and weld failures.

- The panel distortions have led to a significant reduction in the elastic and inelastic stiffness of the sub-assemblage.

Tsai, et al. (1995) performed ten seismic beam to column connection tests in order to examine seismic design of moment resisting steel frame connections. Experimental set up were prepared by placing the column horizontally and connecting the beam to column vertically. Four different beam sections which had various plastic section modulus ratio of beam flange to entire beam section were selected. A36 steel grade was used for all beam specimens. All beam lengths were 2.11 m long. All columns were selected as W 14x159 with A572 Grade 50 steel. Eight mm doubler plates were welded to the panel zones. Cyclically increasing displacements were applied to the specimens during the test until failure was observed. Tsai, et al (1995) concluded that when the panel zones were reinforced properly, they increase the overall inelastic deformation capacity of the connections.

1.3.2 Numerical Studies

Krawinkler H. (1978) pointed out that shear force effects in joints have to be taken into account in the design of steel frames. Because, joints were able to transmit the high shear forces through a column. In the light of his past experimental data, Krawinkler claimed that shear stresses were the highest at the center of the panel zone and reduces toward to the four corners. So, a mathematical model was prepared for strength and stiffness calculations. This model consisted of an elastic-perfectly plastic shear panel surrounded by rigid boundaries with springs at four corners which were representing the stiffness of elements surrounding the panel zone (Figure 1.3).

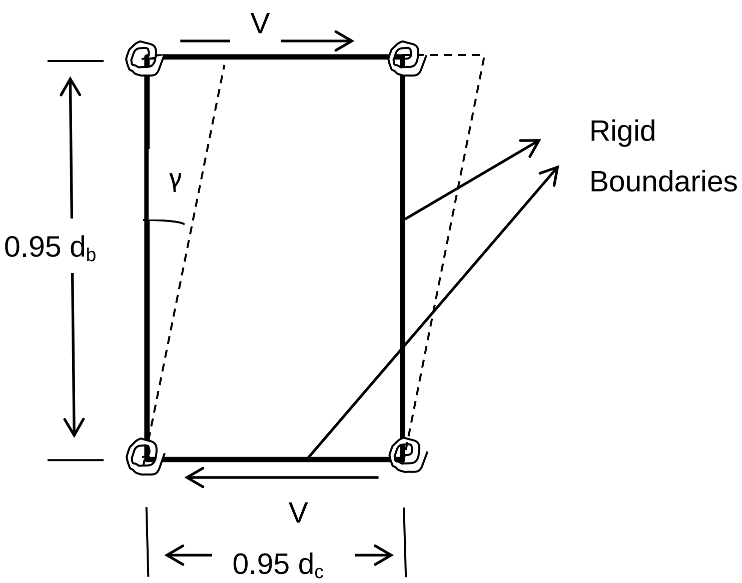

Figure 1.3 Mathematical model for panel zone

The elastic stiffness was defined as;

$$K_e = \frac{V}{\gamma} = 0.95 d_c t G \qquad (1.1)$$

where, V = design shear force in joint, γ = angle of shear distortion, d_c = column depth, t = column web thickness, G = shear modulus of steel

This equation was valid until $\gamma = \gamma_y = F_y/\sqrt{3}$,where F_y = yield strength of column. After yielding the spring stiffness surrounding by panel zone is expressed;

$$K_s = \frac{M}{\theta} = \frac{E b_c\, t_{cf}^2}{10} \qquad (1.2)$$

where, M = moment in beam at face of column, θ = curvature, E = modulus of elasticity of steel, b_c = width of column, t_{cf} = thickness of column flange

The post elastic stiffness can be expressed;

$$K_p = \frac{\Delta V}{\Delta \gamma} = \frac{1.095 b_c t_{cf}^2 G}{d_b} \tag{1.3}$$

where, d_b = beam depth

When the post elastic stiffness was assumed as $\Delta\gamma = 3\gamma_y$, ultimate shear strength of the joint could be computed as,

$$V_u = K_e \gamma_y + 3K_p \gamma_y \tag{1.4}$$

Finally, $K_e \gamma_y = V_y$ the equation could be rewritten as

$$V_u = V_y \left(1 + \frac{3K_p}{K_e}\right) = 0.55 F_y d_c t \left(1 + \frac{3.45 b_c t_{cf}^2}{d_b d_c t}\right) \tag{1.5}$$

Krawinkler concluded that maximum strength and stiffness of frames was reached when all joints were designed for the maximum shear force that could be developed.

Tsai and Popov (1990) studied two representative steel frames with different panel zone designs to analyze the seismic panel zone design effect on elastic story drift. For this study, the six-story, 4 bay symmetrical rectangular building was used with two different panel zone designs. Doubler plate thicknesses were designed according to the 1988 UBC provision. At the two left columns, the doubler plates were designed for minimum strength of panel zone. At the two right columns the doubler plates were designed for intermediate-strength panel zones. Four different models were prepared with computer program ANSR-1. Two of these models were formulated without the use of flexible joints. Instead of this 50 % and 0 % offset were utilized. The other two models were prepared with clear lengths for beams and columns and allowing panel zones to deform in shear. For all models the story shear were set as a triangular shape. The 20-story, 4 bay symmetrical rectangular building was used for three different panel zone design. Strong panel zone, intermediate strength panel zone, and minimum strength panel zone were designed. Under SEAOC 1988, Uniform Building Code 1988 provisions seismic forces were applied for five different cases. Two of these cases were set for 50 % and 0 % rigid offsets. The weak panel zone design led the top story drift 254 mm. However, the roof displacement was recorded nearly 178 mm with strong panel zone design. Tsai and

Popov concluded that while the seismic design codes permit thinner column webs and reduced the panel zone doubler plate thickness, the weak panel zone would lead to larger story drifts.

El-Tawil S., et al., (1999), has prepared a finite element model to examine inelastic panel zone behavior, the effect of the panel zone yielding on the connections, and make a comparison between current design provisions. For this purpose, three different configuration of an exterior beam-to-column connection were analyzed. In the first series, the effect of column web thickness on inelastic behavior of the connection was examined. In the second series, the effect of beam depth and in the last series, the effect of column flange thickness was examined. For the all series, A36 Gr. Steel was used for beams and A572 Gr. 50 steel was used for columns. El-Tawil, et al., used a mixture of 4-node shell and 8-node brick reduced integration elements to model the panel zone in this study. The 8-node brick elements were used at the intersection between the beam bottom flange and column. The rest of the sub assemblage was modeled with shell elements. Multipoint constraints were used to provide compatibility between the shell and brick elements. All analyses were done with a computer program, ABAQUS. El-Tawil concluded that weak panel zones made beam plastic rotation demands smaller, but at higher plastic connection rotations, the shear stress conditions were more critical. Therefore, weak panel zones could cause brittle or ductile fracture at higher connection plastic rotations. El-Tawil added that FEMA 267A (1997) design provision estimated the panel zone strength reasonable with different beam depths, but overestimates the strength of connection with very thick column flanges.

Kim and Engelhardt (2002) prepared mathematical models for describing monotonic and cyclic load-deformation response of the panel zone. They indicated that the panel zone yield moment and elastic stiffness were proportioned with increase of the ratio of the column flange thickness to column depth. In previous studies, just shear deformation mode was included in the panel zone model (Krawinkler, 1978). In their study, for monotonic loading both bending and shear deformations modes were included in the panel zone model. This model was based on quadri-linear panel zone moment deformation relationship. Dafalias' bounding surface theory combined with Cofie's rule for movement of the bound line was adopted for the cyclic loading model. For all models available experimental data were compared with findings. They concluded that the models explained the effects of material yielding and strain hardening. After yielding and strain hardening, the panel zone strength would decrease due to shear buckling of panel zone or fracture of the column or beam flanges at the corners of the panel zone. Kim and Engelhardt also mentioned about the effectiveness of the doubler plates. Based on their models and experimental data comparison, the doubler plates were not so efficient in panel zone strength and stiffness. The researchers also added that in many past experiments the doubler plate effect was less than 50 % percent.

Hsiao K.J., et al. (2007), proposed that the 2003 International Building Code (IBC) suggested that contribution of the panel zone deformations to overall story drifts must be included in the mathematical model of the steel moment resisting frame systems. To do so, two types of models, finite element model and line-element model were evaluated in this study in order to compare the differences between two models. A total of 16 beam-column combinations, both exterior and interior sub-assemblages, were prepared for each type of models. The finite element models for nonlinear static analysis of exterior and interior frame with welded flange plate connection was constructed by a computer program NISA/DISPLAY. The models were consisted of brick and wedge elements. The line elements were set analyzed with computer program SAP2000. A point load was applied to the tip of the columns for both finite element model and line-element model. Hsai, et al., concluded that the finite element model could accurately calculate the overall story drifts by taking the panel zone deformations into account. Moreover, the line element with point sized joint model also computed the elastic story drifts fairly accurately if the beam depth was less or equal to 530 mm. However, the line element model calculated the story drift more than the finite element model at the usage of deeper beams.

Castro J.M., et al. (2008), presented the main differences between the European and the U.S. design provisions for panel zones in this study. Castro, et al., indicated that AISC, FEMA and, Eurocode had differences in calculating panel zone strength, the evaluation of panel zone demand and contribution of the axial and shear force in the column. Cruciform and multistory sub-structure models were prepared to investigate the design provisions. Both models consisted of IPE 400 beams and HE 340A columns. Panel zone strength varied from 70-110 % of the total plastic capacity of the connecting beams. The sub-structures were modeled with OpenSees v 1.7.3 using different types of finite elements. Beams and columns were modeled with a refined mesh. For the panel zone, a new model, 4 external nodes with 13 independent springs were utilized. All springs were assigned rigid except panel zone springs which behaved according to tri-linear behavior based on Krawinkler model. Nonlinear static pushover analyses were conducted by controlling the top node to a target drift to 4% of the story height. Castro, et al., indicated that strong panel zone reduced the plastic hinge rotation in the beams and weak panel zone resulted in high distortional demands. He concluded that Eurocode and AISC design provisions could result to weak panel zones. This was the result of calculating the panel zone strength with overestimating flexural contribution of the column flanges. However, FEMA-350 calculated panel zone strength based on the shear capacity of the column cross-section. The panel zone design according to FEMA-350 resulted in better outcomes.

Brandonisio G., et al. (2011), studied a limit value of geometric slenderness of panel zones, the ratio of panel zone width (b) to panel zone thickness (t), beyond that the panel zone did not buckle under shear forces. They prepared a finite element model by varied aspect ratios and increasing panel zone slenderness, and then the findings were compared with American and European design provisions. For this purpose, 52 finite element models have been prepared with computer program ABAQUS 6.7. Four node shell elements have been utilized for the specimens, with the number of integration points through the element thickness equal to five. The panel zone area was meshed densely, however other parts of the external sub-assemblage was meshed coarsely. The nonlinear finite element analyses have been done by using the modified Riks method. The sub-assemblage was loaded through eight nodes at the tip of the beam by applying a monotonic displacement history. The target displacement of the beam tip was arranged to 250 mm which corresponded to 25 % of the interstory drift. Brandonisio concluded that U.S. codes' limitations to the panel zone slenderness were capable of avoid panel zone shear buckling. On the other hand, Eurocode 3 limitations were not at the safe side. The result of finite element analyses and the available experimental data showed that the panel zone slenderness should be lower than 0.3 for avoiding shear buckling of the panel zone.

Brandonisio G., et al. (2012), worked on the mechanical behavior of the panel zone and discussed the European and American code differences about panel zone design. For this purpose, a nonlinear finite element analysis was performed and experimental test results carried on by different researchers were compared. The parametric finite element analysis was conducted on ten models of beam to columns connections. Panel zone aspect ratios varied from 0.29 to 1.29. A finite element program, ABAQUS 6.7 was used for the nonlinear analysis. Four node shell elements have been utilized for the specimen models. The panel zone area was meshed densely, however other parts of the external sub-assemblage was meshed coarsely. The nonlinear finite element analyses have been done by using the modified Riks method. The beam end was displaced through eight nodes by applying a monotonic displacement history. The analysis was performed until the target displacement of 250 mm was reached. Brandonisio, et al., concluded that American design code was good at panel zone shear strength prediction. However, European practice overestimated the panel zone strength up to 60 %. The authors also added that continuity plates should not be included in the panel zone capacity calculations.

1.4 Scope of the Thesis

The main objective of this thesis is to investigate panel zone behavior in steel moment resisting frames. The Northridge earthquake of January 17, 1994 challenged the design provisions. Pre-Northridge connections were believed that they were capable of developing large plastic rotations without significant strength loss. However, brittle fractures were experienced at beam-to-column connections. So, after 1994 Northridge earthquake design codes revised connection capacity calculations. The AISC Specification, FEMA 355 D recommendations, studied throughout the thesis, suggest different panel zone design. This situation might lead to different connection performance under seismic loads. Hence, it is important to know how the differences between these codes affect panel zone behavior.

Majority of the studies completed so far concentrated on the behavior of sub-assemblages and isolated connections with panel zones. The key question that needs to be answered is the level of demand on panel zones during seismic actions. Past research have only focused on the elastic frame behavior and identified the influence on panel zone deformations on the lateral drifts. There is a clear need for a study that focuses on the inelastic behavior of moment resisting frames with yielding panel zones. In this thesis, a finite element parametric study has been undertaken to evaluate the level of demands under seismic actions. A three story five bay frame with different beam and column sizes were analyzed using explicit dynamic finite element method to address these issues.

In Chapter 2, the panel zone design concepts according to Turkish Seismic Code, AISC Specifications, FEMA 355 D, and Eurocode 3 are studied to indicate similarities and differences among them. In Chapter 3, finite element modeling details are presented and the results of the experimental research conducted by Krawinkler (1971) are used for verification of the finite element model. In Chapter 4, the details and results of the parametric study are presented. Finally, conclusions are presented in Chapter 5.

CHAPTER 2

DESIGN OF PANEL ZONES ACCORDING TO VARIOUS NORMS

A literature survey which covers provisions of Turkish Seismic Code, AISC Specification and AISC Seismic Provision, FEMA 355D, and Eurocode 3 for sizing panel zones was conducted. These specifications have provisions for determining the shear force demand on the panel zone and the resistance provided by the panel zone. First, panel zone shear force demand calculations according to these provisions are presented and explained individually. Then, panel zone shear strength capacity calculations are given.

2.1 Panel Zone Shear Force Demand

The panel zone shear force demand is defined differently according to Turkish Seismic code, AISC specification, FEMA 355D, and Eurocode 3. The following sections present how the shear force demand is calculated for panel zones.

2.1.1 Turkish Seismic Code (2007)

The panel zone limited by the column and beam flanges shall be dimensioned according to necessary shear force and shear force capacity. The necessary shear force V_{ke} , of the panel zone shall be equal to 0.80 times of the sum of bending moment capacities of connecting beams at the face of column.

$$V_{ke} = 0.8 \sum M_{pbeam} \left[\frac{1}{d_b} - \frac{1}{h} \right] \tag{2.1}$$

where M_{pbeam} = plastic moment capacity of connecting beam, d_b = beam depth, d_c = column depth, h = average story height on above and below the joint.

The application of the moments at column face to determine shear strength of the panel zone recognizes that beam hinging will take place at a location away from the beam to column connection which will result in amplified effects on the panel zone shear, despite this a reduction factor of 0.8 on the beam yielding effects is included to the calculation of the necessary shear force of panel zone. However, El Tawil (1999) indicated that in some cases gravity loads might inhibit the development of plastic hinges on both sides of a column. However, this is not a case especially for one sided connections and at perimeter frames where gravity loads may be relatively small.

2.1.2 AISC Specification (2010) and AISC Seismic Provision (2005)

According to the AISC Specification, column web shear may be significant within the boundaries if the rigid connection of two members with their webs in a common place. Such webs must be reinforced when required force $\sum R_u$ for Load and Resistance Factor Design (LRFD) exceeds the column web available strength as shown in Figure 2.1.

For the design according to LRFD :

$$\sum R_u = \frac{M_{u1}}{d_{b1}} + \frac{M_{u2}}{d_{b2}} - V_u \qquad (2.2)$$

where M_{u1} = moments summation due to factored lateral loads ,and the moments due to factored gravity loads on the right side of the connection. (N mm), M_{u2} = difference of the moments due to factored lateral loads ,and the moments due to factored gravity loads on the left side of the connection. (N mm), d_{b1}, d_{b2} : beam depth (mm).

Conservatively 0.95 times the beam depth has been used for d_b . Krawinkler (1978) indicated that the effective shear area was not beam depth times web thickness (d_b x t_w) since the shear stress distribution was not uniform across the depth of the web and did not decrease linearly to zero through the column flanges, so it was conservative to multiply the actual shear area (d_b x t_w) by 0.95.

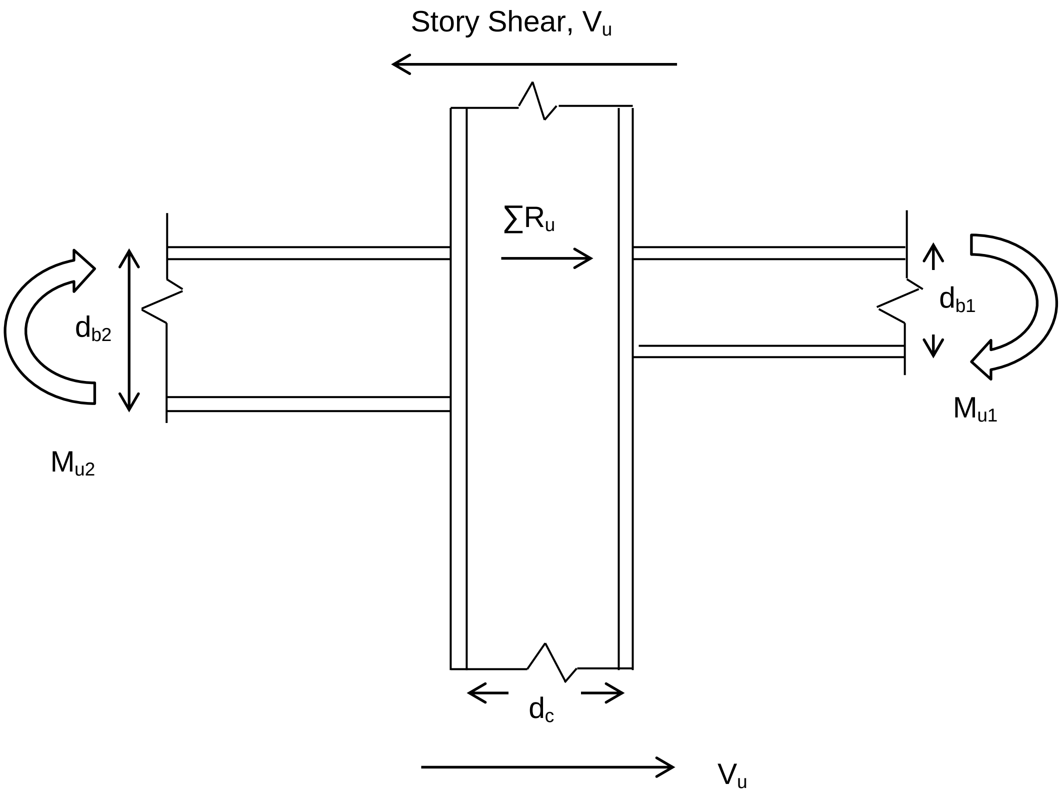

Figure 2.1 LRFD forces and moments in panel zone

2.1.3 FEMA 355D (2000)

Panel Zone Shear Force is calculated as follows according to FEMA 355D ;

$$V_{pz} = \frac{\sum M_{ybeam}}{d_b}\left(\frac{L}{L-d_c}\right)\left(\frac{h-d_b}{h}\right) \quad (2.3)$$

where V_{pz} = panel zone shear force, L = span length.

2.1.4 Eurocode 3: Design of Steel Structures (2005)

The resulting shear force $V_{wp,Ed}$ in the web panel should be obtained using:

$$V_{wp,Ed} = \frac{(M_{b1,Ed} + M_{b2,Ed})}{z} - \frac{(V_{c1,Ed} + V_{c2,Ed})}{2} \quad (2.4)$$

where, $M_{b1,Ed}$, $M_{b2,Ed}$ = bending moments, $V_{c1,Ed}$, $V_{c2,Ed}$ = column shear forces, z = lever arm (beam depth).

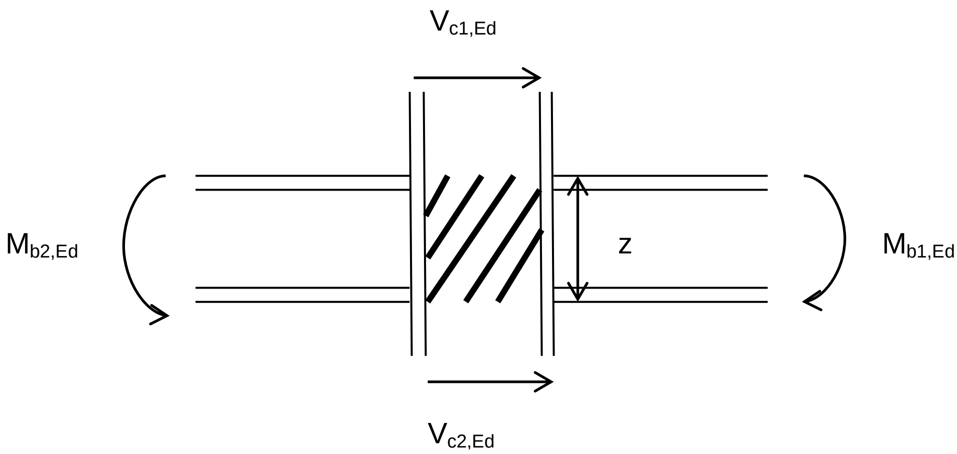

Figure 2.2 Shear forces and moments in web panel

2.2 Panel Zone Shear Strength Capacity

The panel zone strength capacity definitions are presented as follows:

2.2.1 Turkish Seismic Code (2007)

Shearing force capacity, V_p, shall be calculated with the following equation:

$$V_p = 0.6 F_y d_c t_w \left[1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_w} \right] \quad (2.5)$$

In order for the panel zone to have necessary shearing strength the following condition has to be provided:

$$V_p \geq V_{pz} \quad (2.6)$$

Minimum thicknesses, t_{min}, of the each web plates of column and reinforcing plates, if used, shall be provided with the following condition:

$$t_{min} \geq \frac{u}{180} \tag{2.7}$$

where b_{cf} = flange width of column section, F_y = specified minimum yield stress of the column web (MPa), t_{cf} = flange thickness of column section, t_w = total plate thickness in the panel zone including the reinforcing plates, u = length of the periphery of reinforcing plate, V_p = Shearing force capacity.

2.2.2 AISC Seismic Provision (2005) and AISC Specification (2010)

The available strength of the web panel zone for the limit state of shear yielding shall be determined as follows for Intermediate Moment Frames (IMF) and Ordinary Moment Frames (OMF):

$$\Phi = 0.90\ (LRFD)$$

for Special Moment Frames (SMF) :

$$\Phi = 1.0 (LRFD)$$

The nominal strength, R_n, shall be determined as follows:

(a) When the effect of panel zone deformation on frame stability is not considered in the analysis :

(i) For $P_r \leq 0.4P_c$

$$R_n = 0.60F_y d_c t_w \tag{2.8}$$

(ii) For $P_r > 0.4P_c$

$$R_n = 0.60F_y d_c t_w \left(1.4 - \frac{P_r}{P_c}\right) \tag{2.9}$$

(b) When frame stability, including plastic panel zone deformation, is considered in the analysis:

(i) For $P_r \leq 0.75P_c$

$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_w}\right) \tag{2.10}$$

(ii) For $P_r > 0.75P_c$

$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_w}\right)\left(1.9 - \frac{1.2P_r}{P_c}\right) \tag{2.11}$$

where A = column cross sectional area (mm^2), $P_c = P_y$ (N) (LRFD), $P_c = 0.6P_y$ (N) (ASD), P_r = required strength (N), $P_y = F_y A$, axial yield strength of the column (N)

If adequate connection ductility is provided and the frame analysis considers the inelastic panel zone deformations, the additional shear strength is provided by the factor $\left(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_w}\right)$. Krawinkler (1978) pointed out that when the panel zone web has completely yielded in shear, the column flanges increase panel zone strength. The post elastic stiffness of the joint is valid for a range $\Delta\gamma=3\gamma_y$.

2.2.3 FEMA 355D (2000)

FEMA 355D recommends that maximum plastic rotational capacity of panel zone was achieved at the balanced condition. The balanced condition is shear yielding of panel zone and flexural yielding of connecting beam at nearly the same load level. Experiments done by Lee et al. (2000) and Roeder (1996) have showed that the best performance would be achieved in a balance design of panel zone. Panel zone yielding is a yield mechanism not a failure mode. Moreover, ductile performance is more likely if the yield capacity of the panel zone is balanced with flexural yielding in the beam.

Flexural yielding of beam is calculated as:

$$M_{ybeam} = S\,F_{yb} \tag{2.12}$$

where, F_{yb} = yield strength of beam, S = elastic section modulus of the beam

Panel zone yielding is calculated as follows:

$$V_{yield} = A_{eff} \frac{F_y}{\sqrt{3}} \tag{2.13}$$

$$A_{eff} = 0.95 d_c t_w \tag{2.14}$$

$$V_{yield} = 0.55 F_y d_c t_w \tag{2.15}$$

Recommended balance condition for maximum plastic rotational capacity:

$$V_{pz} \leq (0.9) 0.55 F_y d_c t_w \tag{2.16}$$

Where, F_y = yield strength of column, d_c = column depth, t_w = column web thickness.

2.2.4 Eurocode 3: Design of Steel Structures (2005)

(1) The design methods are valid if $d / t_w < 69$

(2) For a single-sided joint, or for double-sided joint in which the beam depths are similar, the shear resistance $V_{wp,Rd}$ of an unstiffened column web panel, subject to a design shear $V_{wp,Ed}$ should be obtained using:

$$V_{wp,Rd} = \frac{0.9\, f_{y,wc} A_{vc}}{\sqrt{3} \gamma_{M0}} \tag{2.17}$$

$$A_{vc} = A - 2 b_c t_{cf} + (t_{cw} + 2r) t_{cf} \tag{2.18}$$

$$\gamma_{M0} = 1.0$$

where, $f_{y,wc}$ = column yield strength, A_{vc} = shear area, b_c = width of column, A = column area, t_{cf} = column flange thickness, t_{cw} = column web thickness, r: root radius of the column flange, γ_{M0} = partial safety factor

(3) The shear resistance may be increased by the use of stiffeners or supplementary web plates.

(4) Where transverse web stiffeners are used in the compression and the tension zone, the plastic shear resistance of the column web panel $V_{wp,Rd}$ may be increased by $V_{wp,add,Rd}$ given by:

$$V_{wp,add,Rd} = \frac{4M_{p,fc,Rd}}{d_s} \; but \quad V_{wp,add,Rd} = \frac{2M_{p,fc,Rd} + 2M_{p,st,Rd}}{d_s} \qquad (2.19)$$

where, $M_{p,fc,Rd}$ = the plastic moment resistance of a column flange, $M_{p,st,Rd}$ = the plastic moment resistance of a stiffener, d_s = the distance between the centerlines of the stiffeners

CHAPTER 3

FINITE ELEMENT MODELING DETAILS AND VERIFICATION

In this chapter the finite element modeling details are introduced with the verification of the model. Understanding of elastic and inelastic behavior of the steel connections has importance on efficient earthquake resistant design of steel frames. For this purpose, many studies have been conducted to understand the behavior of these elements. However, there are a few experimental studies performed to date on panel zones which can be employed for finite element model verification. The experimental study performed by Krawinkler et al. (1971) is considered as a benchmark in this thesis and the test results of this experiment are used in adjusting the finite element models.

At the beginning of 1970's Krawinkler et al. (1971), conducted an experiment to investigate inelastic behavior of panel zones. The researchers stated that it was important to appraise the strength, stiffness, ductility and energy absorption capacity of the structural elements of unbraced moment resisting frames. For this purpose, a subassemblage consisted of a column with two beams framing into it was prepared for experimental investigation. Four specimens were prepared and subjected to two different cyclic loadings in this research. However, only B-1 specimen test results were used for verification purpose of the finite element model in this thesis. Following section demonstrates the details of the experimental study carried out by Krawinkler et al. (1971).

3.1 Benchmark Experiments of Krawinkler et al. (1971)

The researchers selected a 20 story – 4 bay office building prototype to simulate the real structure behavior and the prototype frame was designed according to "working or allowable stress design" philosophy and followed the requirements of the U.B.C (1967) and AISC specification (1969). Krawinkler et al. (1971) decided to prepare two different sub-assemblages in order to represent upper story and lower story behavior. The dimensions of the test set-up members were assigned by scaling down the prototype design. Since just the B-1 specimen test results are used in verification this specimen dimensions are presented in Table 3.1.

Table 3.1 Geometric Properties of Beam-Column Sections

	Column			Beam		
	Prototype	Specimen	Scale	Prototype	Specimen	Scale
	14 WF 228	8 WF 67		24 WF 68	14 B 22	
b (mm)	403.10	209.55	1.91	227.58	127.00	1.79
d (mm)	406.40	228.60	1.78	602.23	348.49	1.73
t_w (mm)	26.54	14.61	1.82	10.57	5.84	1.81
t_f (mm)	42.88	23.70	1.81	14.78	8.51	1.74
A (mm^2)	1703.32	500.38	3.4	508.00	164.34	3.1

The test set-up used in B-1 and B-2 specimens is illustrated in Figure 3.1. In this test set-up, steel grade was selected as 250 MPa (36 ksi) for all members. Full penetration groove welds were used to connect the beams directly to the column flanges. The effect of the floor system on the strength and stiffness of the subassemblage was not simulated. Out of plane movement was prevented. The lateral load, H, was applied at lower column tip. The gravity loads on column and beams were kept constant during the cyclic loading. The gravity loads applied throughout the experiment were 1509 kN (339.2 kips) on column and 52.6 kN (12 kips) on beams. The upper column tip was connected to a hinge support and the beam ends were supported by rollers.

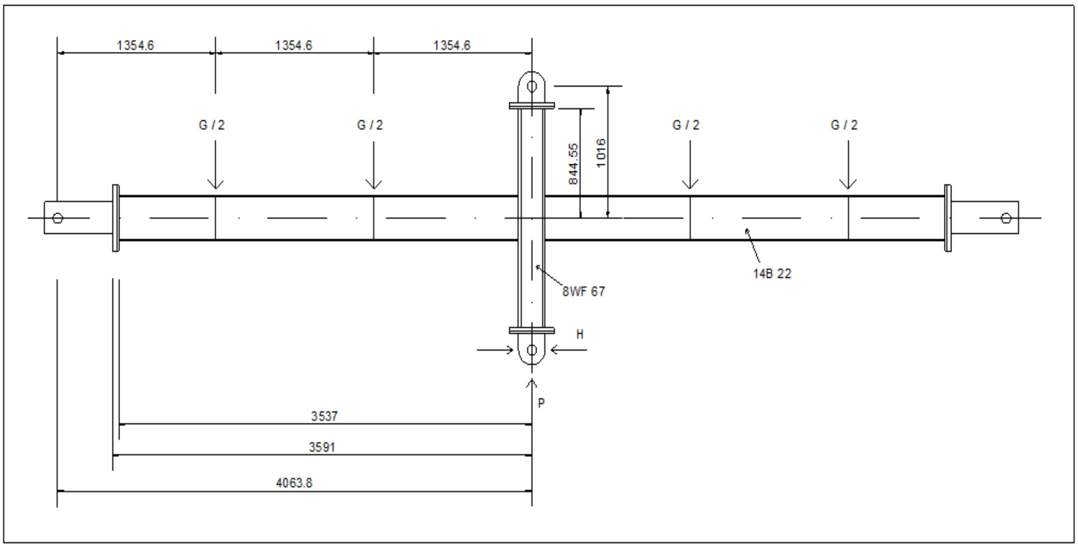

Figure 3.1 The test set up (dimensions in mm)

The specimens were subjected to two different loading programs. Loading program 1 was utilized for B-1 specimen (Figure 3.2). This program was prepared in order to investigate low cycle fatigue problem. Applied lateral load was increased by step wise manner with four symmetric cycles per step. The cyclic loading was continued until the specimen had failed.

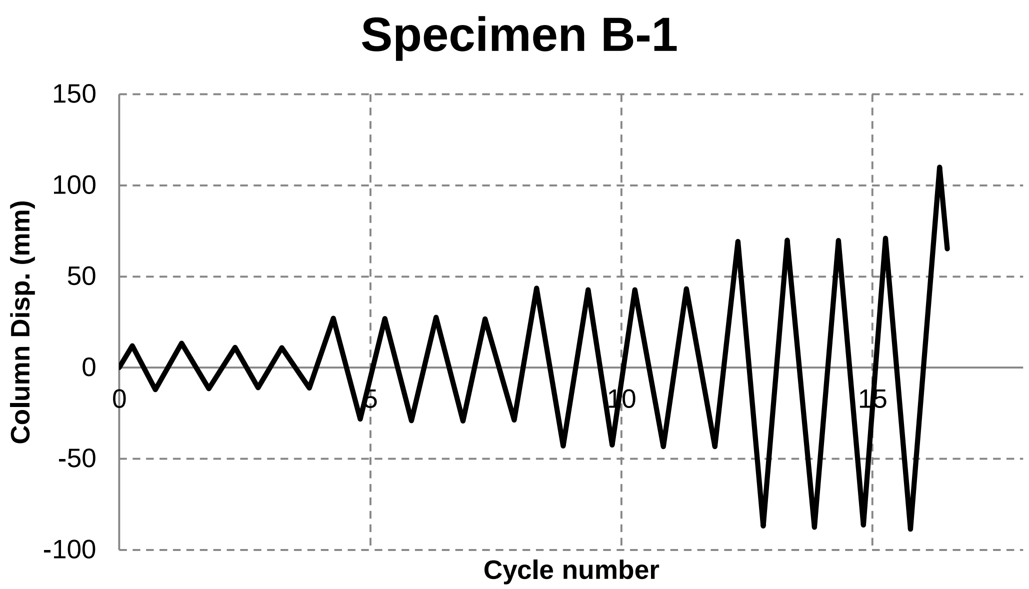

Figure 3.2 The loading program

The researchers indicated that there was a leakage problem in the hydraulic system during the loading program. This situation caused problems in keeping the lateral load at exact values. It could lead some errors while taking data records. Moreover, the test duration was two days, so strain aging might affect the response of the specimen.

Krawinkler et al. (1971) concluded that B-1 specimen stress level decreased from the center of the panel zone towards to the four corner. No local kinking was observed at the column flanges. The connection showed a high reserve strength after first yielding. The researchers added that inelastic deformations took place in the plastic hinge regions of the beam at the B-1 specimen.

3.2 Finite Element Modeling Details

LS-Dyna module of a commercially available computer program ANSYS 10.0 was used to perform finite element analysis of the B-1 specimen. The structural elements of the subassemblage were modelled with 4-node shell elements with both bending and membrane capabilities (shell163). The fully-integrated Belytschko-Tsay shell element formulation (KEYOPT(1) = 12) uses a 2 x 2 quadrature in the shell plane. The shell elements were meshed with quadrilateral elements and area meshing was used. Finite element mesh of specimen B-1 is given in Figure 3.3. Transient analysis type and explicit dynamics solution method was utilized for verification analysis. The material properties given by Krawinkler et al. (1971) were assigned to the model. The modulus of elasticity of steel was set to 200 GPa (29000 ksi) and Poisson’s ratio was considered to be equal to 0.3. The nonlinear material behavior was modeled using Von Mises yield criterion. Von Mises yield criterion means that yielding occurs when the strain energy associated with the shearing distortions reaches a critical value. For all elements, a bilinear strain-stress relationship with a hardening modulus of 2 GPa (290 ksi) utilized.

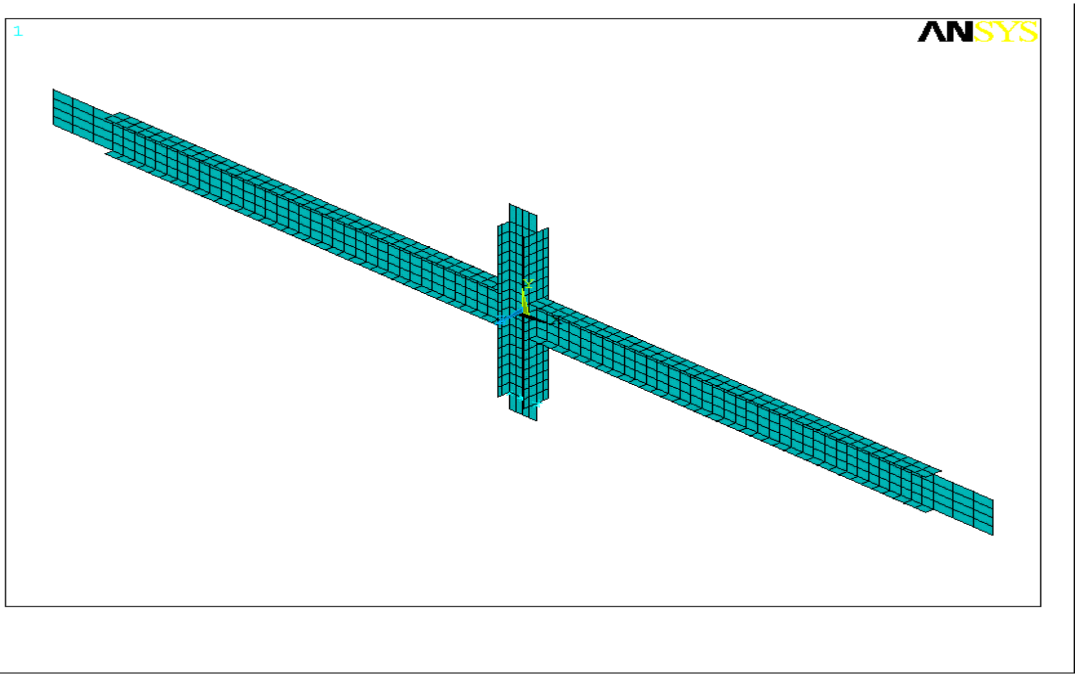

Figure 3.3 Finite element modelling

Out of plane movement of the beam ends and column tips were prevented by applying displacement boundary condition ($u_z = 0$). Vertical movement of the beams far ends was prevented to simulate roller support and vertical and horizontal movements of the upper column end were prevented to simulate hinge supports. The gravity loads on beams and column were applied at a single node as they were point loads. The lateral load on lower column end was also introduced at just a single node. Quasi-static analysis was appropriate to simulate the Krawinkler et al. (1971) experiment. Since LS-Dyna module of ANSYS 10.0 software was used for the finite element model, lateral loads were applied with time intervals between each other to simulate the quasi-static analysis and avoid oscillations. Moreover, an arbitrary 30 tons (66.14 Ibs) mass was distributed equally through 3 nodes at the lower column end to overcome oscillation problem. Five percent of critical damping was assigned as global damping of the subassemblage.

Numerical results and experimental observations were compared in Figure 3.4 and Figure 3.5. The results were presented on the different graphs to evaluate the findings well. The experimental data is plotted by digitizing the B-1 graph of lateral load (H) versus column displacement (δ) presented by Krawinkler et al. (1971). It is evident from comparisons that finite element analysis findings conforms the experimental results.

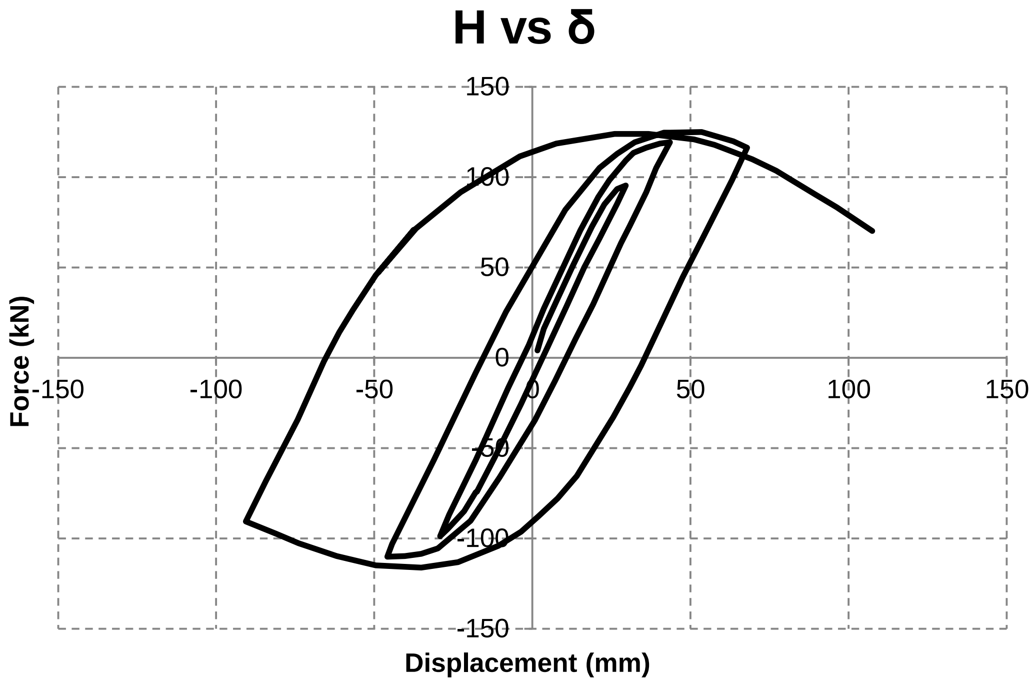

Figure 3.4 Test results by Krawinkler, et al.(1971)

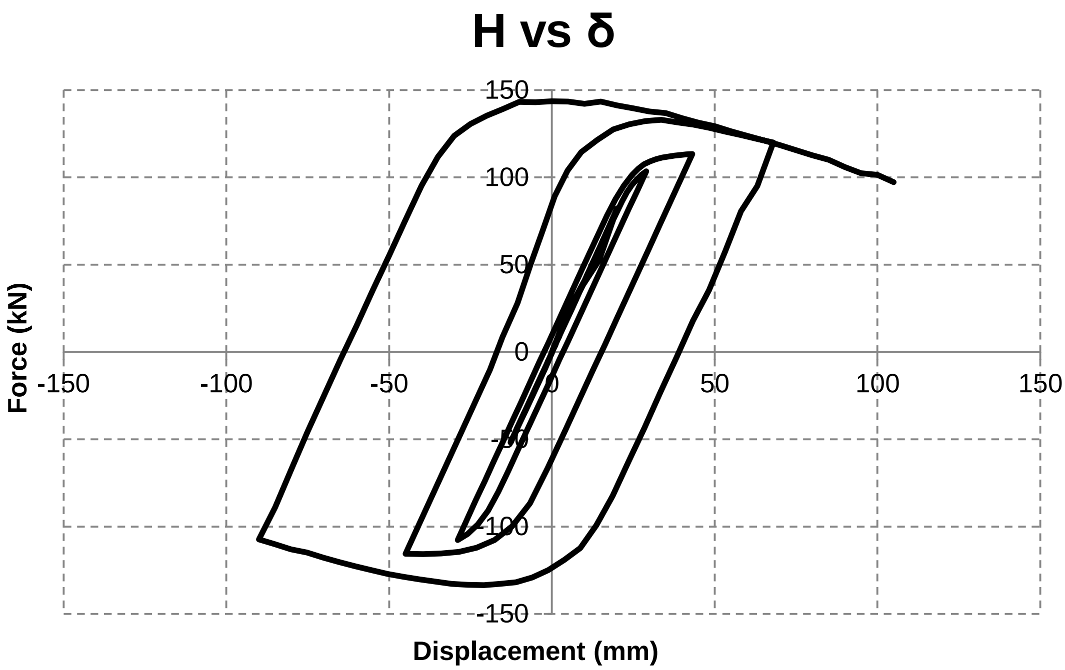

Figure 3.5 LS- Dyna Analysis Results

Based on comparison of the actual and predicted behavior of the subassemblage, it can be concluded that finite element models for parametric studies are able to simulate the response of structural elements.

CHAPTER 4

PARAMETRIC STUDY

Panel zone deformation demands are investigated in this chapter. A representative steel moment resisting frame was analyzed under twenty different ground motions to observe the demands. The panel zones were proportioned using the provisions given in different specifications and the demands for particular panel zone designs were obtained. Local strain demands at the panel zones as well as the global frame deformation demands are evaluated.

A steel moment resisting frame with 3 stories and 5 bays was modeled with commercially available finite element software ANSYS 10.0 and ANSYS LS-Dyna as shown in Figure 4.1. The story height was chosen as 3.5 m and the bay width was set to 8 m. All members were modeled using shell elements. The frame was prevented from out of plane movement and the column bases were fixed.

Several parameters were expected to affect the panel zone behavior and these parameters were changed to quantify the dependence of deformation demands on these. In general, the level of axial load on columns, beam depth, and level of seismicity are considered as the prime variables. A significant amount of time was devoted to select proper beam and column sections for the parametric study. For this purpose a simple panel zone design computer program was developed. This program selects proper column sections for a given beam section and level of axial load. Later, it calculates the required thickness of the doubler plate. In order to observe differences in behavior the analysis cases that require the highest amount of doubler plate thickness were selected. The beam depth is represented by two different beam sizes hereafter called as shallow beam and deep beam. All sections that were tried were European rolled shapes. The shallow beam design employs an HE 500A beam section and HE 650B column section. The deep beam design employs HE 900A beam section and HE 1000B column section.

In all cases the steel grade was assumed to be S355 with a yield stress of 355 MPa. In finite element modeling a bilinear stress-strain curve with a hardening modulus of 2 GPa was adopted.

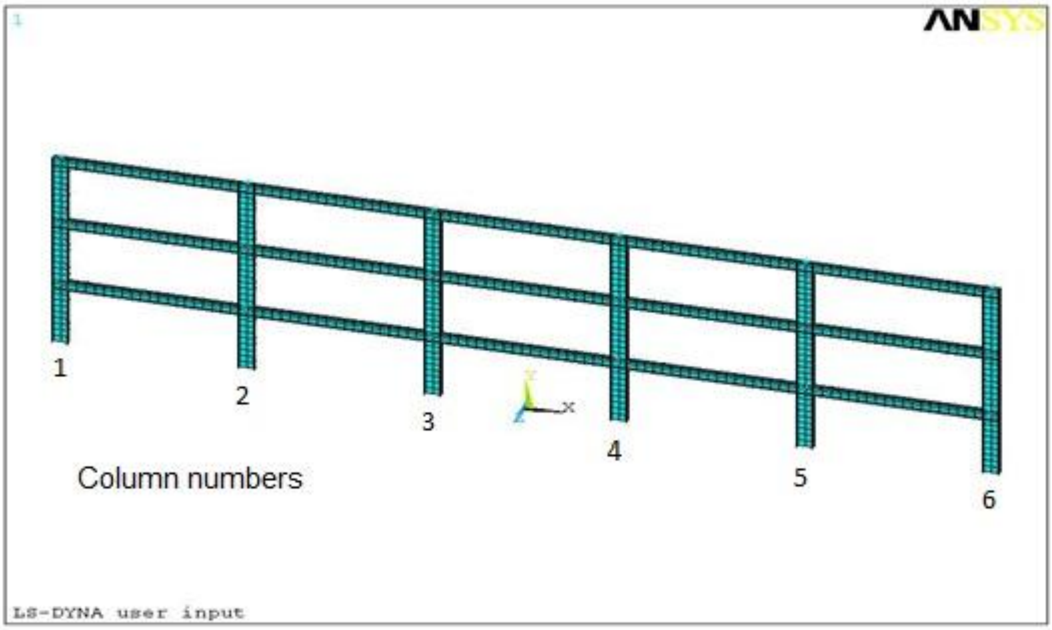

Figure 4.1 A single bay steel moment frame model

Concentrated masses were placed at every story at joint locations. The 600 tons story mass for shallow beam case and 1000 tons story mass for deep beam case were equally distributed over the each column tip at each story.

As shown in Figure 4.1 the frame contains 6 columns. Two of these columns are exterior and the others are interior columns. The required panel zone thicknesses for interior columns were calculated according to AISC and FEMA 355D design provisions. Detailed calculations for panel zone thickness are given in Appendix A. The FEMA 355 D panel zone design recommendation was selected and named as PZ_{ref}. The FEMA 355D recommendations resulted in a doubler plate thickness of 26 mm (total web thickness of 42 mm) for the shallow beam case and a thickness of 18 mm (total web thickness of 37 mm) for deep beam case. Because there is a single beam that frames into the exterior columns, the required panel zone thickness for these columns are 21 mm and 18 mm for the shallow and deep beam cases, respectively. In the finite element models the exterior column panel zones were reinforced according to half thickness of the interior panel zones whether it is required. The column web thicknesses are 16 mm and 19 mm for HE 650B and HE 1000B columns, respectively.

The total shear strain of the joints, interstory drifts, and the top story displacement results of the frame were collected during a typical finite element analysis. The shear strain values were normalized with respect to yield shear strain of the panel zone. The yield shear strain of the panel zone is calculated with following formula:

$$\frac{F_{ycolumn}}{\sqrt{3}} = G.\gamma_{yield} \tag{4.1}$$

where, $F_{ycolumn}$ is the yield strength of the column taken as 355 MPa and G is the shear modulus taken as 79.3 x 10^3 MPa. The shear strain of the panel zone is calculated as γ_{yield} = 0.002585.

Twenty ground motion records listed in Table 4.1 were used for time history analyses. Acceleration response spectra for these ground motion records are given in Figure 4.2.

Table 4.1 Ground Motion Records List

Folder Name	Earthquake	Country	Location	Site Geology	M_w	PGA (g)
gm1	Imperial Valley	USA	El Centro Array #1, Borchard Ranch	Alluvium	6.5	0.141
gm2	Morgan Hill	USA	Gilroy Array #2 (Hwy 101 & Bolsa Rd)	Alluvium	6.1	0.157
gm3	Northridge	USA	Downey County Maint. Bldg.	Alluvium	6.7	0.223
gm4	Imperial Valley	USA	Meloland Overpass	Alluvium	6.5	0.314
gm5	Northridge	USA	Saticoy	Alluvium	6.7	0.368
gm6	Whittier Narrows	USA	Cedar Hill Nursery, Tarzana	Alluvium / Siltstone	6.1	0.405
gm7	Loma Prieta	USA	Capitola Fire Station	Alluvium	7.0	0.472
gm8	Northridge	USA	Rinaldi Receiving Station	Alluvium	6.7	0.480
gm9	Northridge	USA	Katherine Rd, Simi Valley	Alluvium	6.7	0.513
gm10	Imperial Valley	USA	El Centro Array #5, James Road	Alluvium	6.5	0.550
gm11	Chi Chi	Taiwan	CHY028	USGS(C)	7.6	0.653
gm12	Cape Mendocino	USA	Petrolia, General Store	Alluvium	7.0	0.662
gm13	Kobe	Japan	Takarazu	USGS (D)	6.9	0.693
gm14	Kobe	Japan	Takarazu	USGS (D)	6.9	0.694
gm15	Northridge	USA	Katherine Rd, Simi Valley	Alluvium	6.7	0.727
gm16	Düzce	Turkey	Bolu	Soil	7.1	0.754
gm17	Northridge	USA	Sepulveda VA Hospital	Alluvium	6.7	0.939
gm18	Tabas	Iran	Tabas	Stiff Soil	_	1.065
gm19	Morgan Hill	USA	Coyote Lake Dam	Rock	6.1	1.298
gm20	Northridge	USA	Tarzana Cedar Hill Nursery	Alluvium	6.7	1.778

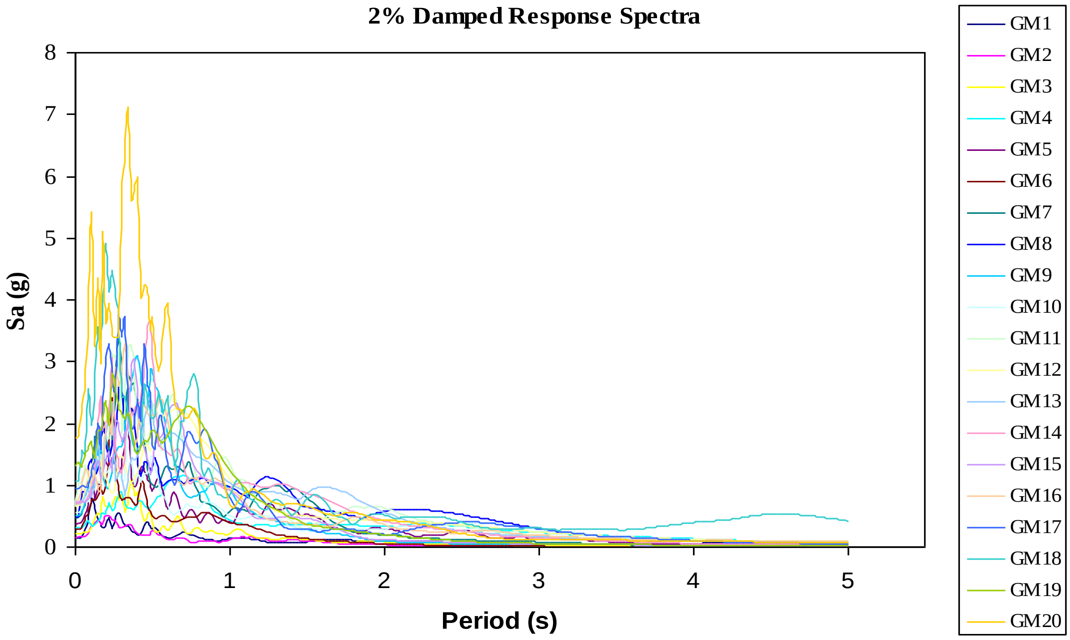

Figure 4.2 2 % Damped Response Spectra

The frames were analyzed using ANSYS 10.0 to determine the fundamental period of vibration. According to the analysis results, the fundamental period of vibrations are 0.8 sec and 0.24 sec for frames with shallow and deep beams, respectively.

4.1 Results of Analysis for Shallow Beam Case

4.1.1 Axial Load Level = 0.0 P_y

Five different interior panel zone thicknesses were investigated and compared to each other. These panel zone thicknesses were 0.4 PZ_{ref} which corresponded to unreinforced column web thickness, 0.6 PZ_{ref}, 0.8 PZ_{ref} which was AISC recommendation, 1.0 PZ_{ref} which corresponded to FEMA recommendation, and finally 1.2 PZ_{ref}. Top story displacement, interstory displacements, and shear strain of the panel zones results' were recorded and evaluated.

4.1.1.1 Results for Panel Zone Thickness Equal to 0.4 PZ_{ref}

The interior panel zones were set according to the 0.4 PZ_{ref}, which was actual column web thickness. The results for the case of 0.4 PZ_{ref} are illustrated in Figures 4.3 through 4.5.

The FEMA 355D requirement for exterior panel zones corresponds to a thickness of 21 mm. Since the actual column web thickness of HE 650B column is 16 mm, the actual column thickness divided by the required thickness ratio corresponds to 0.76. Figure 4.3 shows the deformation demands (shear strain divided by the yield shear strain) as a function of maximum interstory drifts. The exterior columns panel zones experienced up to 5.4 times the yield strain.

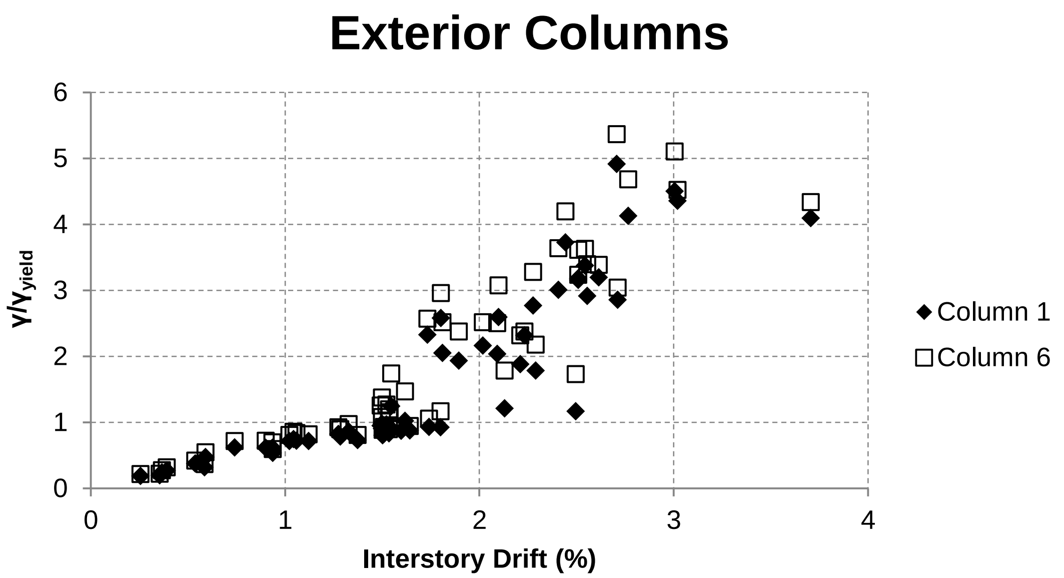

Figure 4.3 Analysis results of exterior columns with unreinforced panel zones

Interior columns of the frame highly yielded. All interior columns showed almost the same pane zone behavior. Actually, column 2 and 5 panel zones had almost the same records for both interstory drifts and normalized shear strains. These panel zones experienced a maximum of 13.8 times the shear yield strain as shown in Figure 4.4. However, column 3 and 4 panel zones shear yield strains were 6 % less than column 2 and column 5. The maximum top story displacement was recorded as 295 mm which corresponds to 3.7 % lateral drift. The base shear lateral displacement response is given in Figure 4.5. As shown in this figure the frame reached to its ultimate capacity and increasing lateral displacements results in an insignificant change in the base shear resistance. It can be concluded that the unreinforced column web thickness resulted in significant amounts of yielding in the interior panel zones. Because the requirements on the exterior panel zones are not excessive, the amount of yielding was much less compared to interior panel zones.

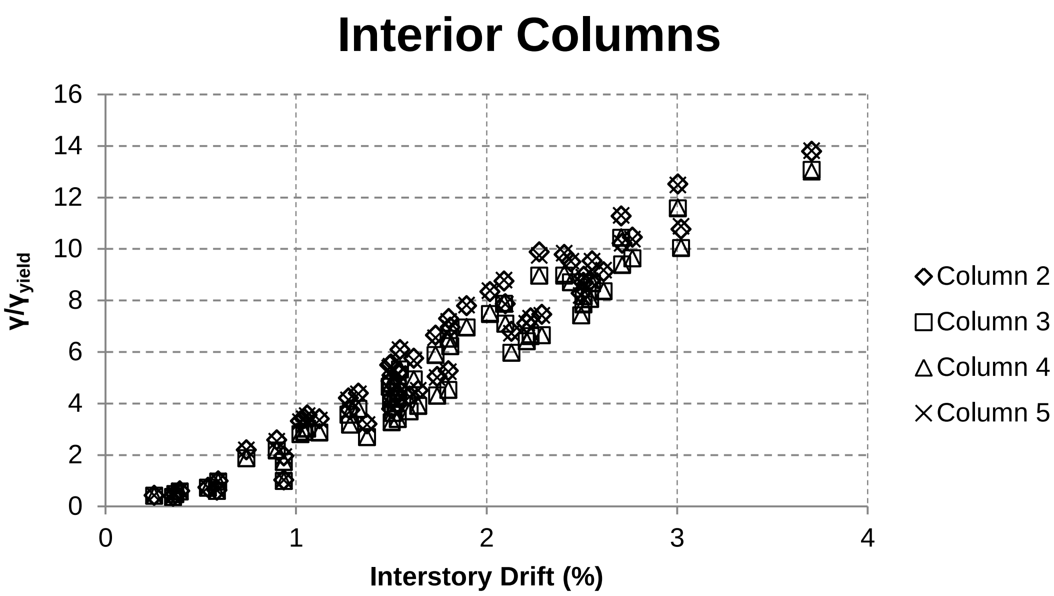

Figure 4.4 Normalized Shear Strain vs Interstory Disp. for interior columns

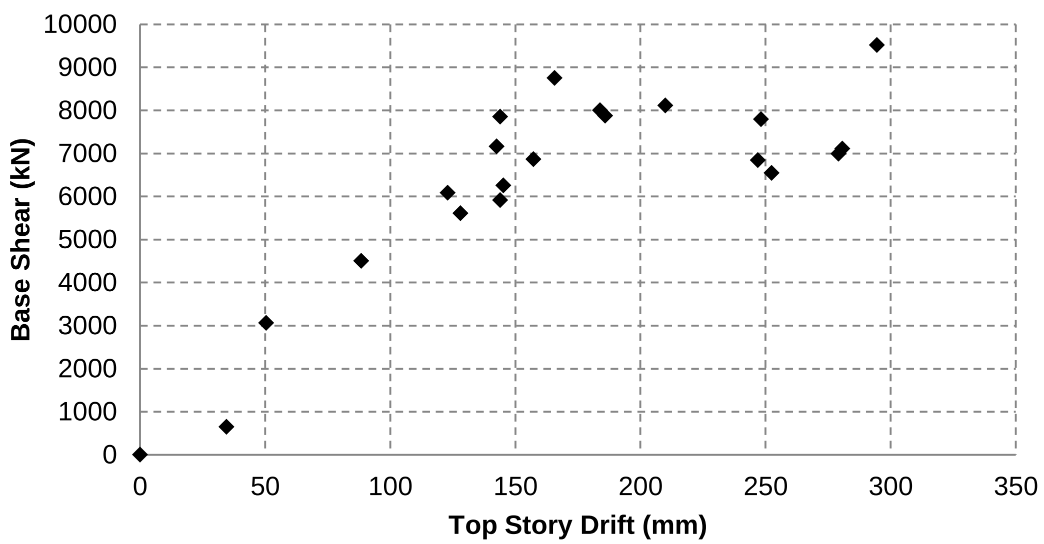

Figure 4.5 Base Shear vs Top Story Drift relationship

4.1.1.2 Results for Panel Zone Thickness Equal to 0.6 PZ_{ref}

The analyses set were repeated by reinforcing the interior panel zones to 0.6 PZ_{ref}, which corresponds to 25 mm. No change was applied to the exterior panel zones; these were kept at the thickness equal to 0.76 PZ_{ref}. The following results were obtained;

Exterior columns experienced nearly 6 times the yield strain. When compared with the previous case the behavior is similar. This is due to the fact that no change was applied to the exterior panel zones. The maximum interstory displacements were nearly 3.5 %. The interstory displacements and normalized shear strains were almost the same with respect to previous case as shown in Figure 4.6.

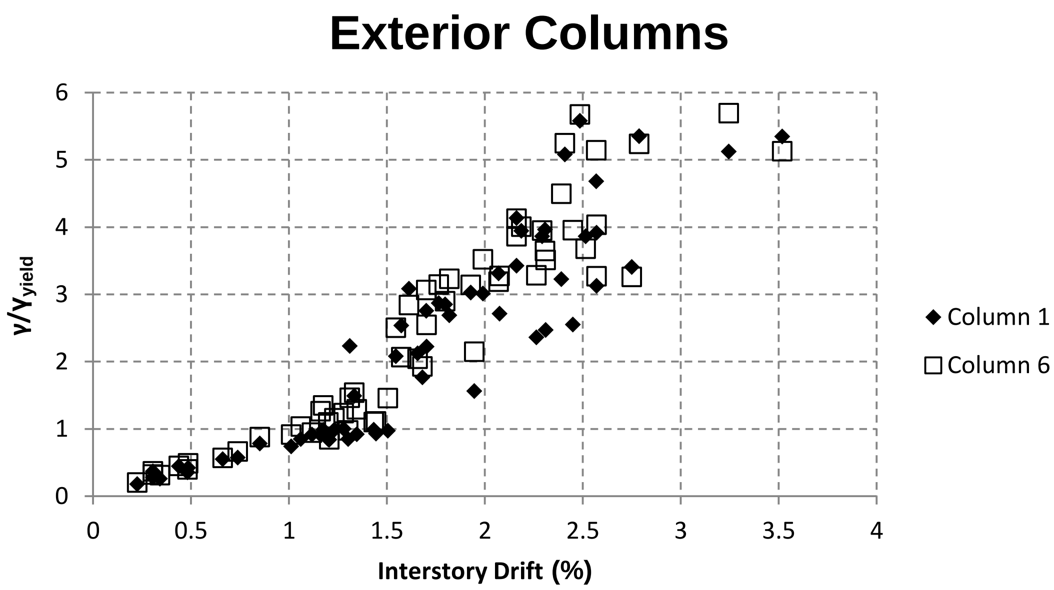

Figure 4.6 Normalized Shear Strain vs Interstory Disp. for exterior columns

All interior panel zones showed almost the same behavior. The shear strains of the panel zones experienced 10 times the yield shear strain as shown in Figure 4.7. The normalized shear strains of the interior panel zones decreased 30 % with respect to previous analysis set. The maximum top story displacement was recorded as 313 mm which was 6 % more than the previous analyses. The maximum base shear force decreased by 12 %, and recorded as 8400 kN as shown in Figure 4.8.

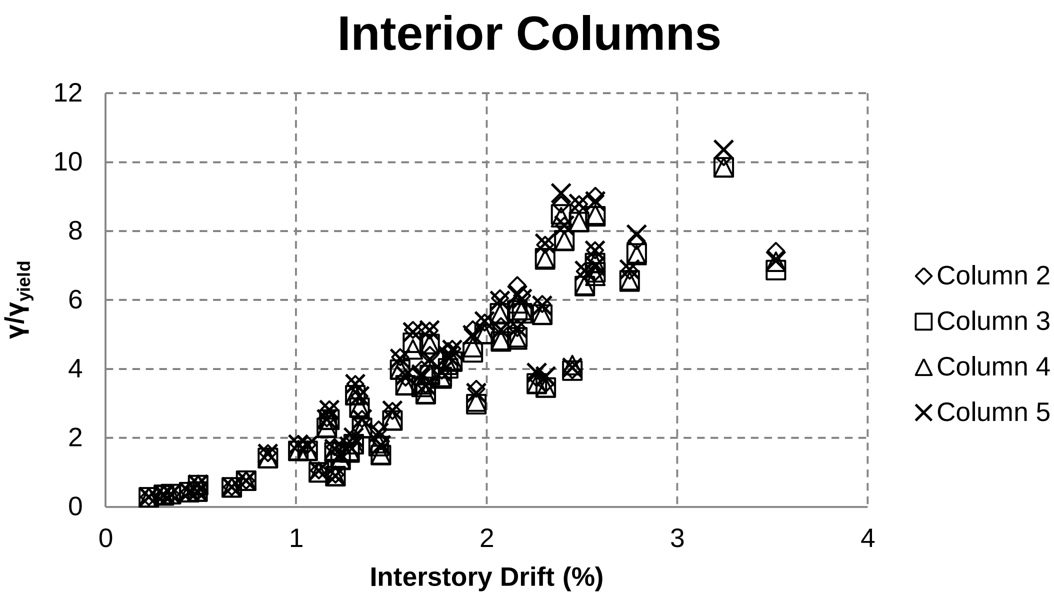

Figure 4.7 Normalized Shear Strain vs Interstory Disp. for interior columns

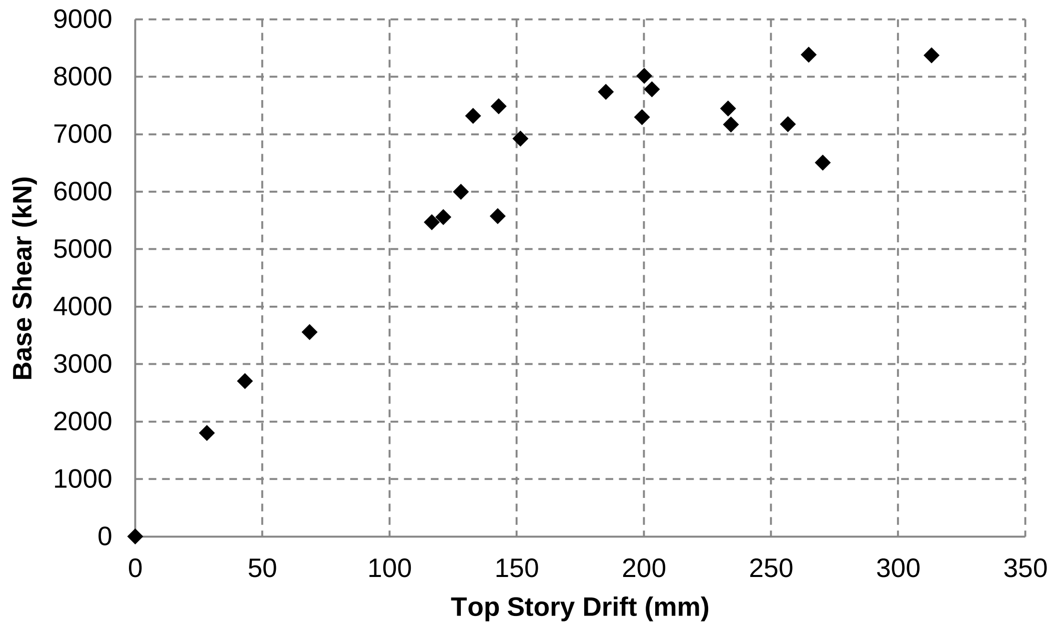

Figure 4.8 Base Shear vs Top Story Drift Relationship

4.1.1.3 Results for Panel Zone Thickness Equal to 0.8 PZ_{ref}

The analyses set were continued with reinforcing the panel zones according to 0.8 PZ_{ref}, which corresponds to 33 mm. This panel zone thickness corresponded to AISC Specification panel zone design. The exterior panel zones remained 16 mm and the interior panel zones were set to 33 mm. The following results were obtained;

Exterior columns experienced up to 4 times the yield strain as shown in Figure 4.9. The maximum interstory displacements were nearly 2.75 %. The interstory displacements decreased by 35 % according to unreinforced column web thickness results. The normalized shear strain values decreased by 37.5 %. The decrease in normalized shear strain can be attributed to the overall decrease in the frame lateral drifts.

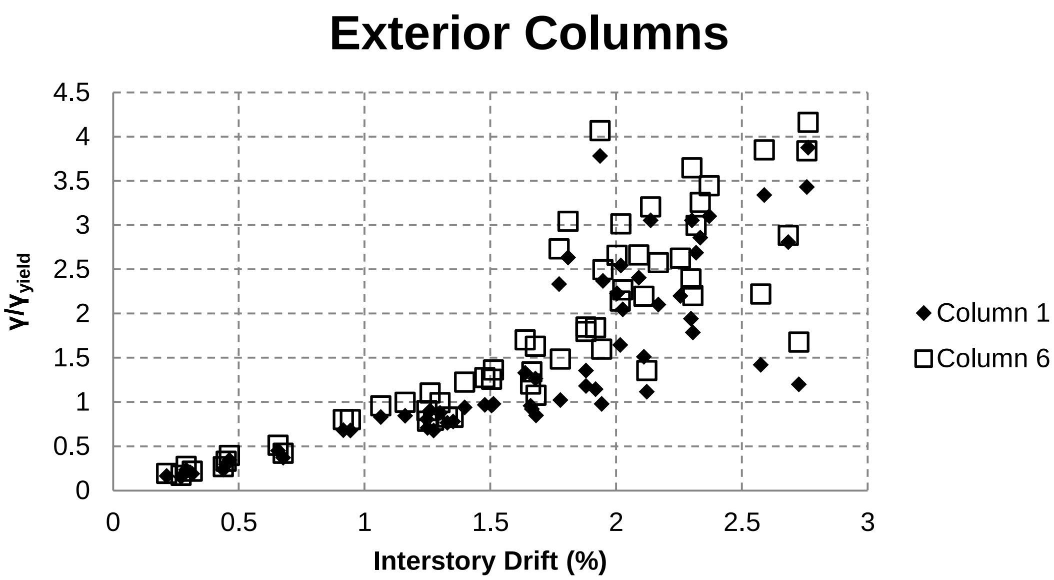

Figure 4.9 Normalized Shear Strain vs Interstory Disp. for exterior columns

Interior panel zones of the frame experienced 6.5 times the yield strain as shown in Figure 4.10. The normalized shear strains of the interior columns decreased by 50% when compared with the case 0.4 PZ_{ref}. The base shear lateral displacement response was given in Figure 4.11. The maximum top story was recorded 277 mm which was 7 % less than the previous analyses set. The maximum base shear was obtained as 9800 kN.

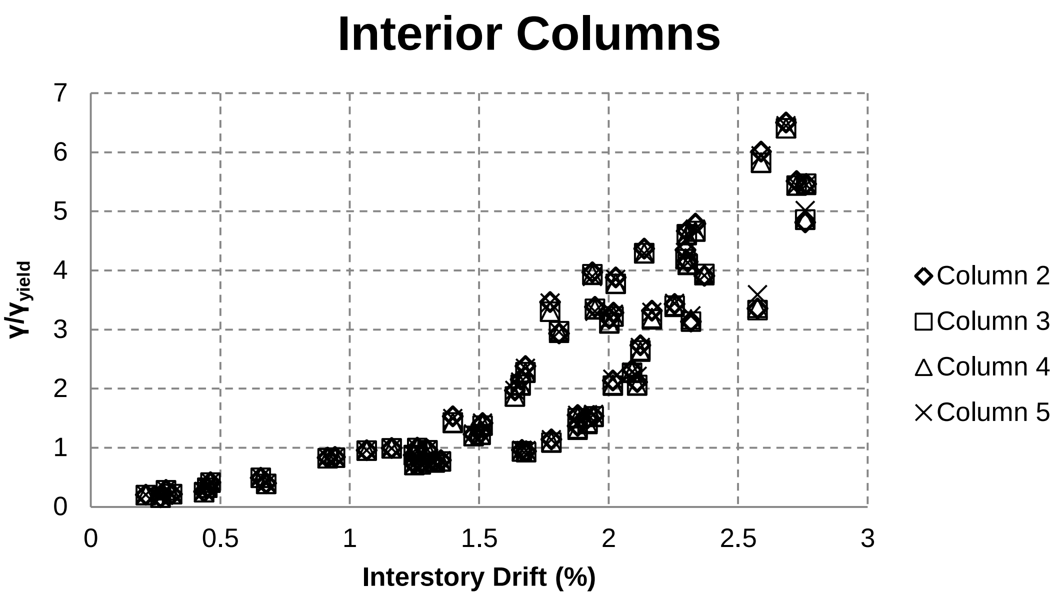

Figure 4.10 Normalized Shear Strain vs Interstory Disp. for interior columns

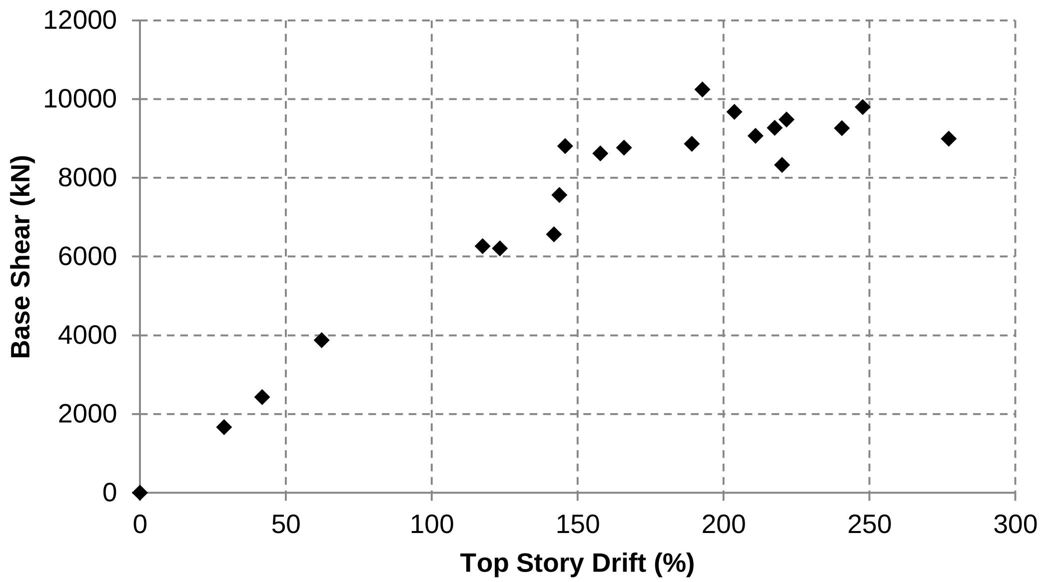

Figure 4.11 Base Shear vs Top Story Drift Relationship

4.1.1.4 Results for Panel Zone Thickness Equal to 1.0 PZ_{ref}

The panel zones were reinforced according to 1.0 PZ_{ref}, 42 mm which is the FEMA 355D recommendation for panel zone design. The exterior panel zones were 21 mm and the interior panel zones were 42 mm. The following results were obtained;

Exterior panel zones shear strain values were nearly equal to the yield shear strain or less as shown in Figure 4.12. The maximum recorded interstory displacement was 2.7 % which is very close to the previous results with 0.8 PZ_{ref}. However, the shear strain values decreased by 86 % with respect to 0.8 PZ_{ref}; and 93 % with respect to 0.4 PZ_{ref} which was unreinforced column web thickness. This decrease is due to the reinforcement applied to the exterior panel zones.

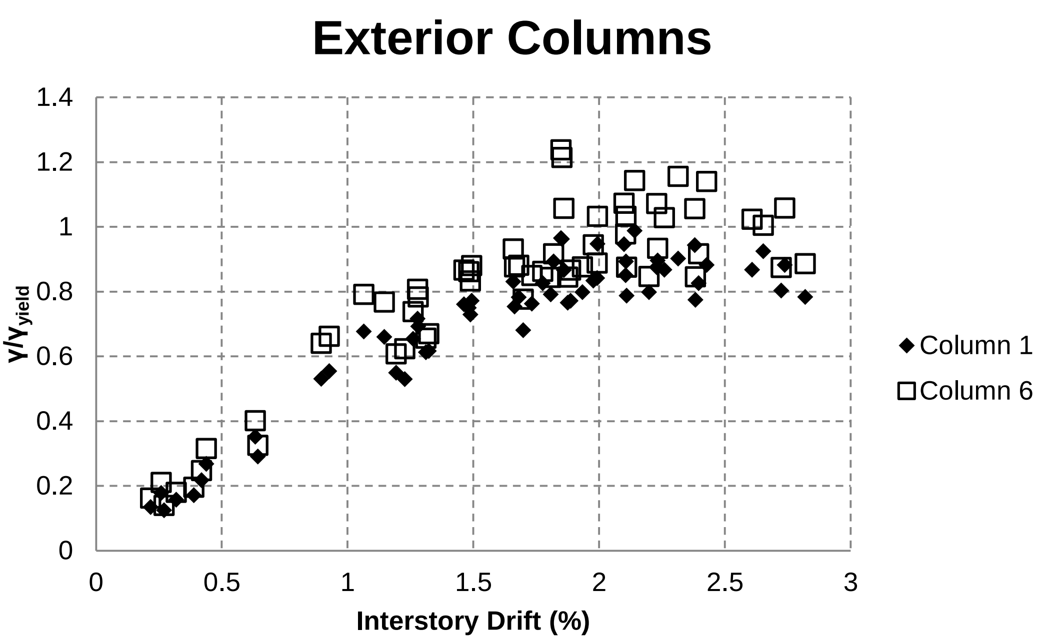

Figure 4.12 Normalized Shear Strain vs Interstory Disp. for exterior columns

The same situation at the exterior panel zones behavior was also observed at the interior panel zones. All shear strain values were less than or equal to the yield shear strain value as shown in Figure 4.13. The base shear lateral displacement response is given in Figure 4.14. The maximum top story displacement was obtained as 268 mm which decreased by 3 % with respect to previous analysis and the maximum base shear was 10460 kN.

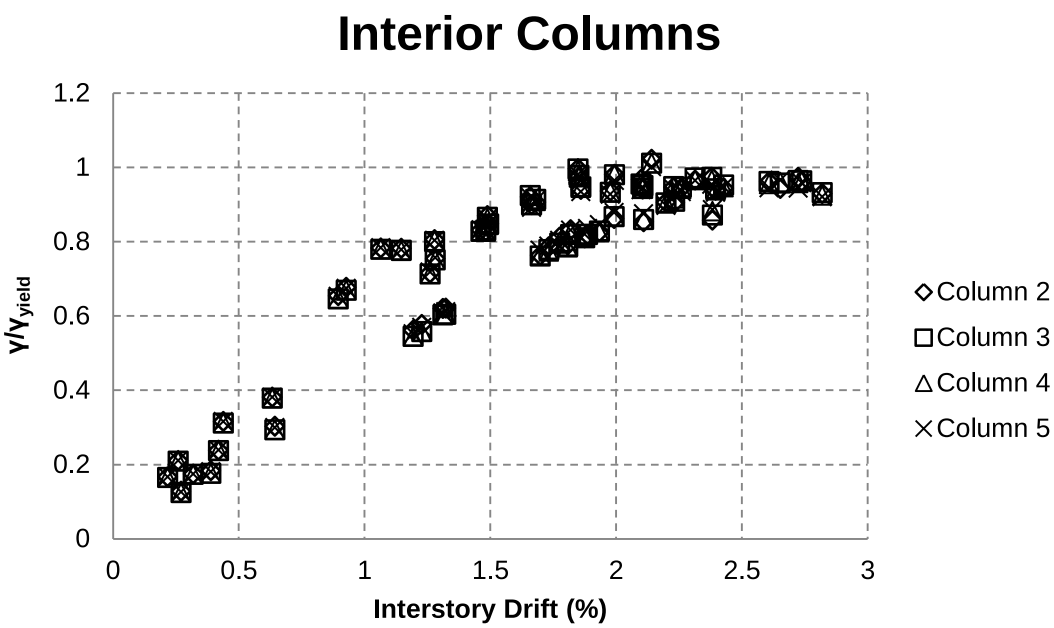

Figure 4.13 Normalized Shear Strain vs Interstory Disp. for interior columns

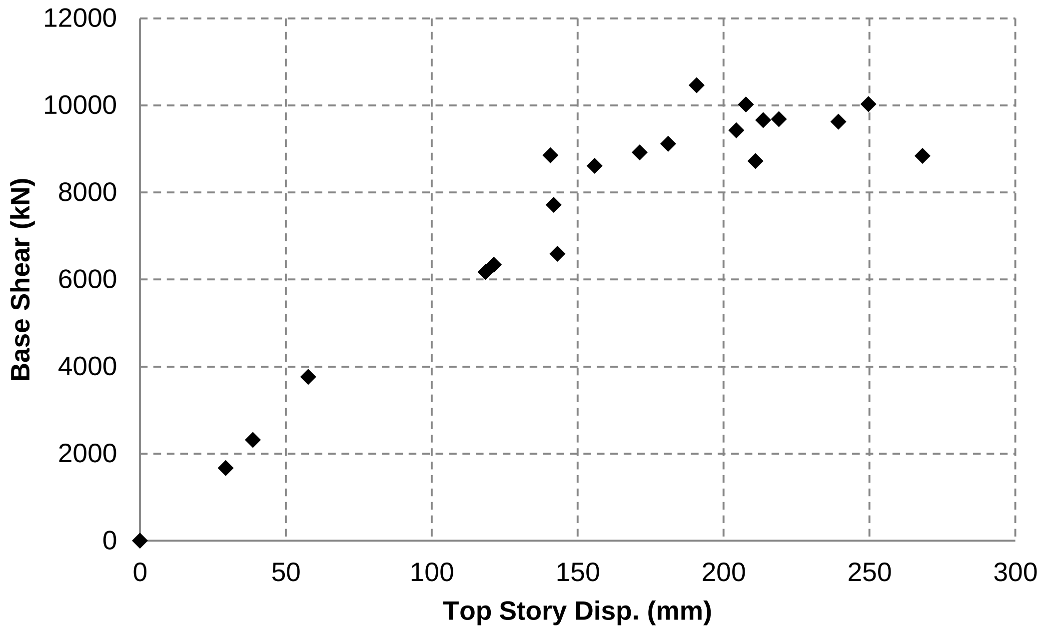

Figure 4.14 Base Shear vs Top Story Disp. Relationship

4.1.1.5 Results for Panel Zone Thickness Equal to 1.2 PZ_{ref}

In order to observe the behavior for cases with a large panel zone thickness beyond the FEMA 355D requirement, the panel zones were reinforced according to 1.2 PZ_{ref}, 50 mm. The exterior panel zones were 25 mm and the interior panel zones were 50 mm. The following results were obtained;

Exterior and interior panel zone shear strain values were less than the yield shear strain as shown in Figure 4.15 and 4.16. The maximum recorded interstory displacement was 2.7 % which was same as the previous results. The shear strain values decreased 7 % with respect to 1.0 PZ_{ref}. Plastic hinges were formed at the beam ends as predicted as shown in Figure 4.18. This figure shows the equivalent plastic strain values. The maximum top story displacement was 264 mm which was almost same as the previous result. The maximum base shear force decreased to 9300 kN as shown in Figure 4.17.

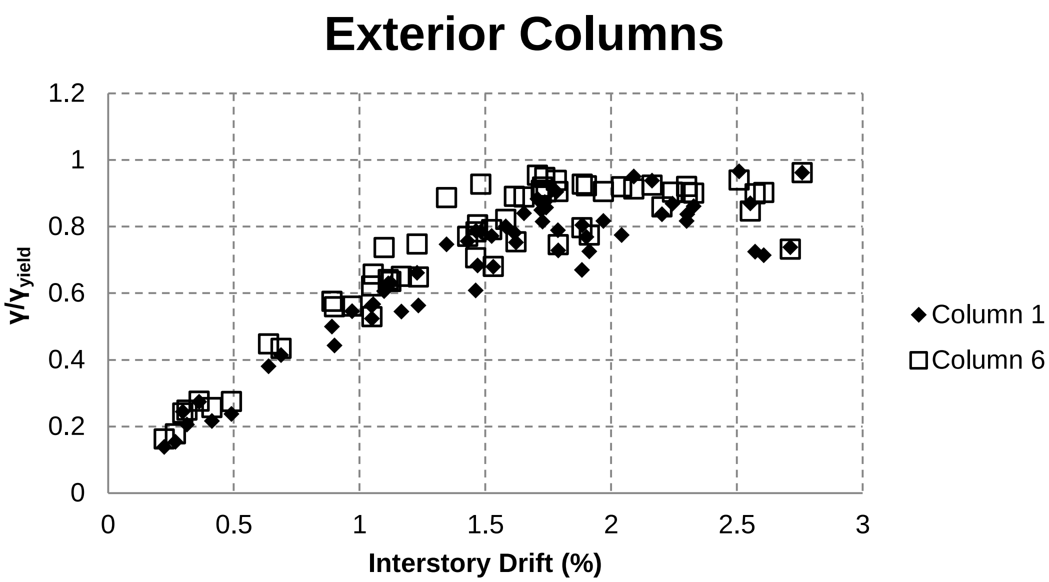

Figure 4.15 Normalized Shear Strain vs Interstory Disp. for exterior columns

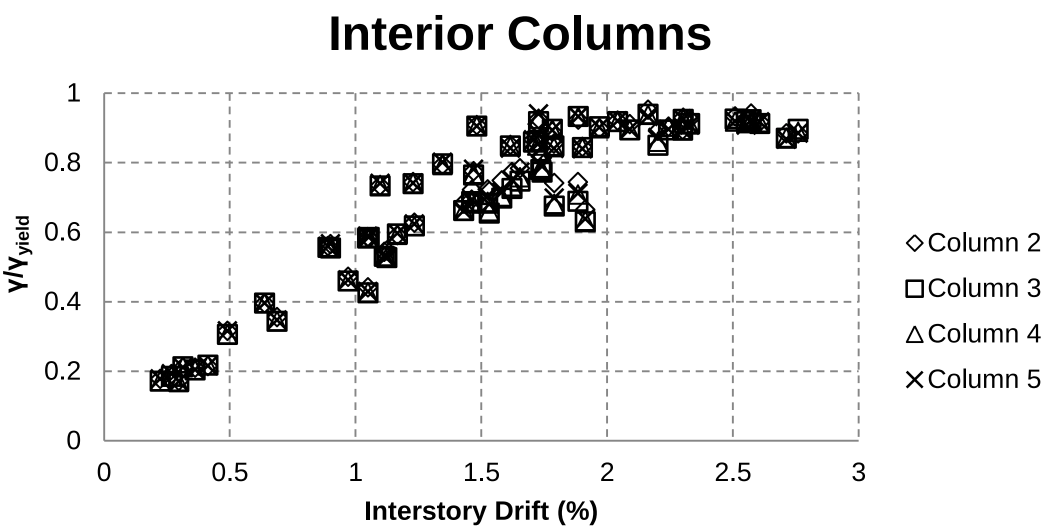

Figure 4.16 Normalized Shear Strain vs Interstory Disp. for interior columns

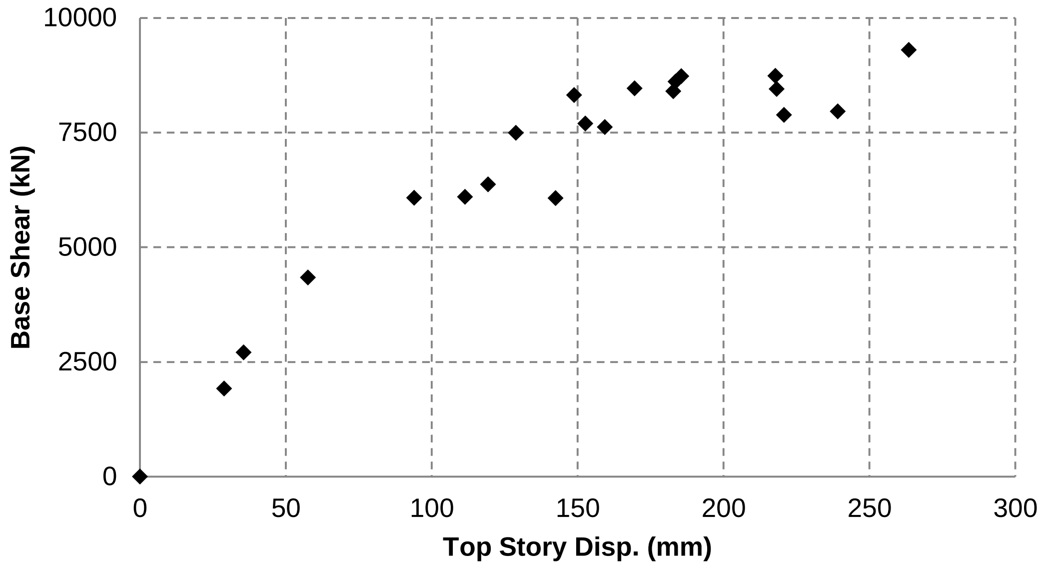

Figure 4.17 Base Shear vs Top Story Disp. Relationship

Figure 4.18 Plastic Hinge Formation

From these analyses results, it can be concluded that the interior panel zones of 0.4 PZ_{ref} experienced 14 times the yield shear strain values. 0.6 PZ_{ref} and 0.8 PZ_{ref} for panel zones led yielding of the beam column joints either. However, 1.0 PZ_{ref} saved the panel zones from yielding. The interstory displacements obtained from 0.8 PZ_{ref}, 1.0 PZ_{ref}, and 1.2 PZ_{ref} analyses series were almost the same. Since unreinforced panel zones were not sufficient, increasing panel zone thickness reduced the normalized shear strains and interstory drifts decreased significantly. The reference panel zone thickness showed a good performance under different seismic loads.

4.1.2 Axial Load Level = 0.3 P_y

The previous analysis did not consider the presence of axial loads on columns. The same analysis was repeated to investigate the effect of axial load on the columns. For this purpose axial loads were placed at the top story which will subject the columns at all stories the same level of axial force. A force level of 30 percent of the axial yield load (0.3 P_y) was considered.

The results of the two sets of analysis are compared in Table 4.2. In this table, the maximums of the investigated quantities are presented. In general, there was no difference in the maximum level of base shear obtained. The level of axial load had an influence on the level of lateral displacements and panel zone shear demands. It is evident from the results that as the panel zone is designed to be weaker the influence of axial loads is more pronounced. When the panel zone thickness is equal to 0.4 or 0.6 PZ_{ref} for interior panel zones, the interstory drifts increase by 22 and 35 percent, respectively due to addition of axial load. This is also reflected by a change in panel zone deformation demands. The deformation demands (γ / γ_{yield}) increase by 23 and 10 percent for these cases, respectively. When the panel zone thickness approaches to the thickness level proposed by FEMA 355D the influence of axial load on the columns become less pronounced. The panel zone demands as a function of the interstory drifts will be evaluated in later sections of this Chapter. The results for different levels of axial load will be combined to come up with demand curves.

Table 4.2 Comparision between 0.0 P_y and 0.3 P_y axial load on columns cases

	PZ_{ref}	$(\gamma / \gamma_{yield})_{max}$			Max. Interstory Disp. (%)			Max. Top Story Drift (mm)			Max. Base Shear (kN)		
		$0.0P_y$	$0.3P_y$	Diff. (%)	$0.0P_y$	$0.3P_y$	Diff. (%)	$0.0P_y$	$0.3P_y$	Diff. (%)	$0.0P_y$	$0.3P_y$	Diff. (%)
Interior Panel Zone	0.4	13	16	23	3.7	4.5	22	295	385	31	9518	9276	-3
	0.6	10	11	10	3.1	4.2	35	313	333	6	8370	8670	4
	0.8	6.5	7	8	2.8	2.8	0	277	280	1	10242	10240	0
	1.0	1.2	1.3	8	2.7	2.8	4	268	272	1	10457	10547	1
	1.2	0.93	1	8	2.7	2.8	4	264	270	2	10294	10894	6
Exterior Panel Zone	0.76	5.4	8.4	56	3.7	4.5	22	295	385	31	9518	9276	-3
	0.76	5.7	6	5	3.1	4.2	35	313	333	6	8370	8670	4
	0.76	4.2	5.5	31	2.8	2.8	0	277	280	1	10242	10240	0
	1.0	1	1.25	25	2.7	2.8	4	268	272	1	10457	10547	1
	1.2	0.96	1	4	2.7	2.8	4	264	270	2	10294	10894	6

4.2 Results of Analysis for Deep Beam Case

4.2.1 Axial Load Level = 0.0 P_y

The same type of analysis was conducted to investigate the effect of beam depth on the panel zone deformation demands. For this purpose, the analyses were repeated using a deeper beam section and an associated column section.

Four different interior panel zone thicknesses were investigated and compared to each other. These panel zone thicknesses were 0.5 PZ_{ref} which corresponded to unreinforced column web thickness as 19 mm, 0.8 PZ_{ref} which was AISC recommendation, 1.0 PZ_{ref} which corresponded to FEMA recommendation, and finally 1.2 PZ_{ref}. Top story displacement, interstory displacements, and shear strain of the panel zones results' were recorded and evaluated.

4.2.1.1 Results for Panel Zone Thickness Equal to 0.5 PZ_{ref}

The interior panel zones were set according to the 0.5 PZ_{ref}, which was the actual column web thickness. The results for the case 0.5 PZ_{ref} are illustrated in Figures 4.36 through 4.38.

The FEMA 355D documents do not require doubler plates for exterior panel zones. Since the actual web thickness of HE 1000B column is 19 mm, the actual column thickness divided by the required thickness ratio corresponds to 1.0. The exterior columns panel zones experienced up to 1 times the yield strain as shown in Figure 4.19.

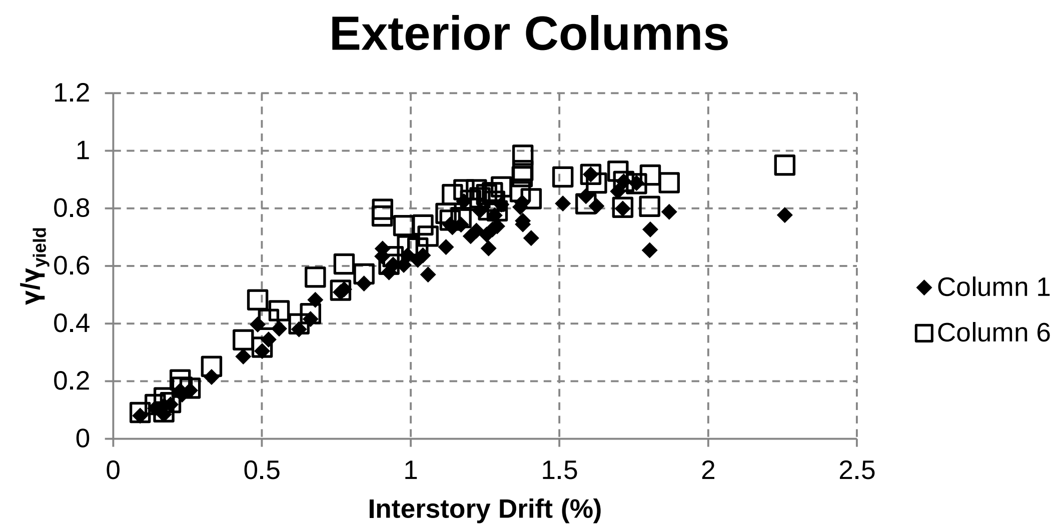

Figure 4.19 Normalized Shear Strain vs Interstory Disp. for exterior columns

Interior panel zones experienced 6.5 times the yield shear strain as shown in Figure 4.20. The maximum interstory drift was 195 mm and the base shear was 20700 kN as shown in Figure 4.21.

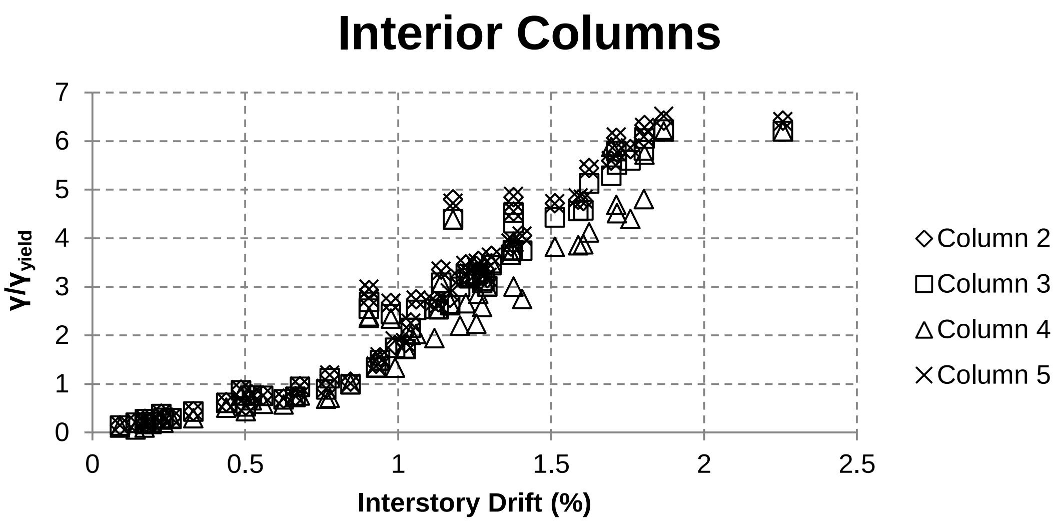

Figure 4.20 Normalized Shear Strain vs Interstory Disp. for interior columns

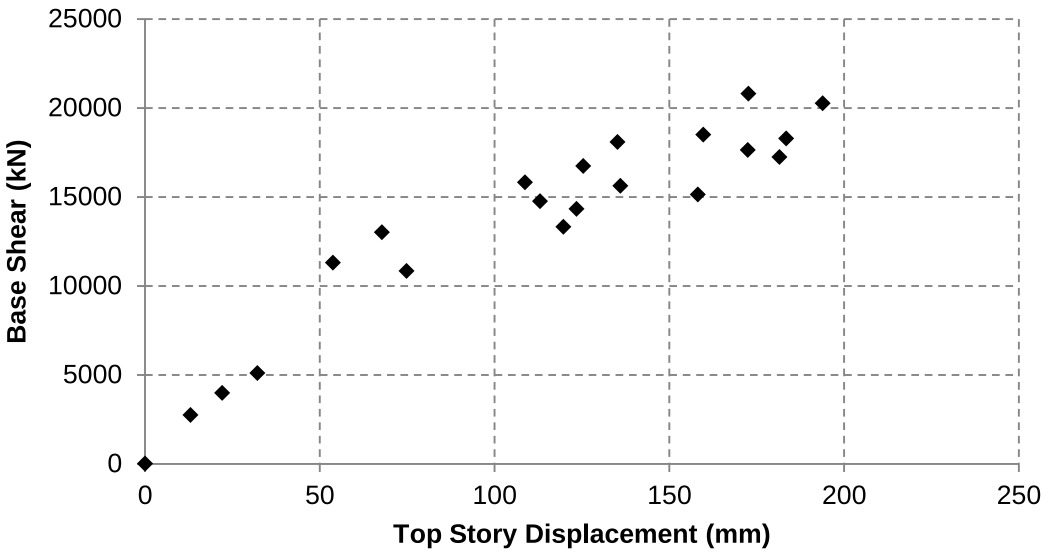

Figure 4.21 Base Shear vs Top Story Disp. Relationship

4.2.1.2 Results for Panel Zone Thickness Equal to 0.8 PZ_{ref}

The interior panel zone thicknesses were set according to 0.8 PZ_{ref}, 29 mm which was AISC recommendation for interior panel zone sizing. As it was shown from the previous analysis results that unreinforced column web thickness for exterior panel zones exhibited elastic behavior, the same results were also obtained for 0.8 PZ_{ref} analyses that exterior panel zones remained elastic as shown in Figure 4.22. However, the interstory drift was decreased 10% and recorded maximum two percent.

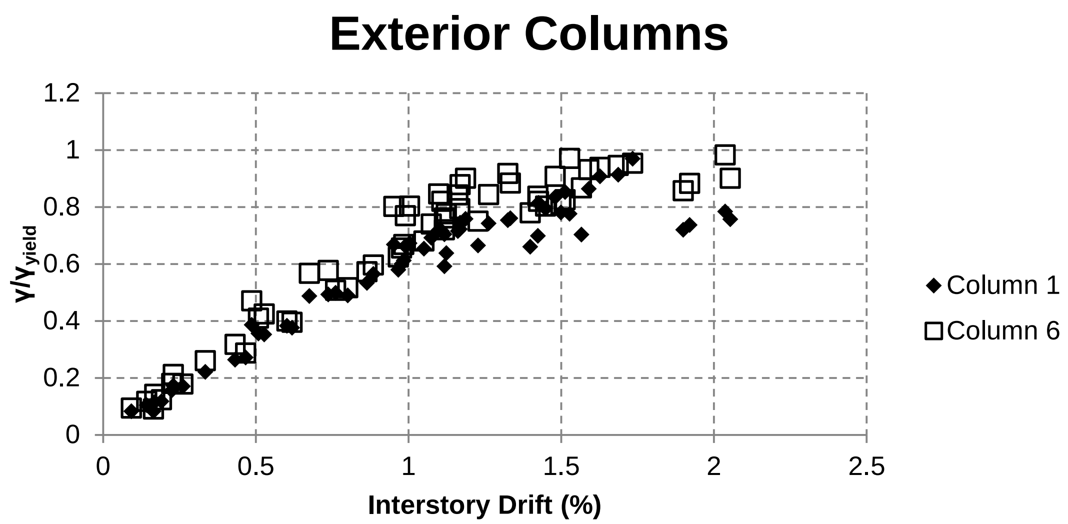

Figure 4.22 Normalized Shear Strain vs Interstory Disp. for exterior columns

Interior panel zones of the frame experienced 5 times the yield shear strain. The normalized shear strains decreased 37.5 % with respect to 0.5 PZ_{ref} as shown in Figure 4.23. The maximum top story displacement was recorded 193 mm and the max base shear was 22400 kN as shown in Figure 4.24.

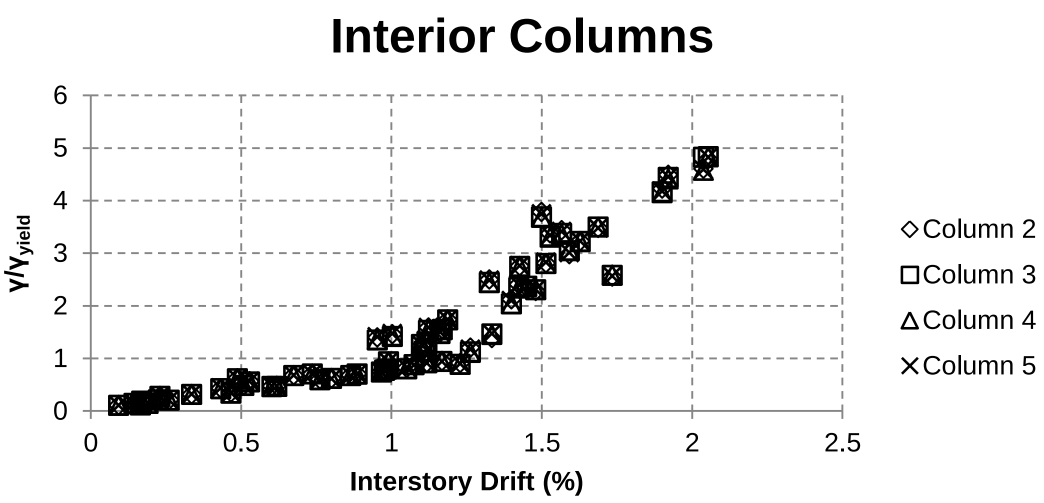

Figure 4.23 Normalized Shear Strain vs Interstory Disp. for interior columns

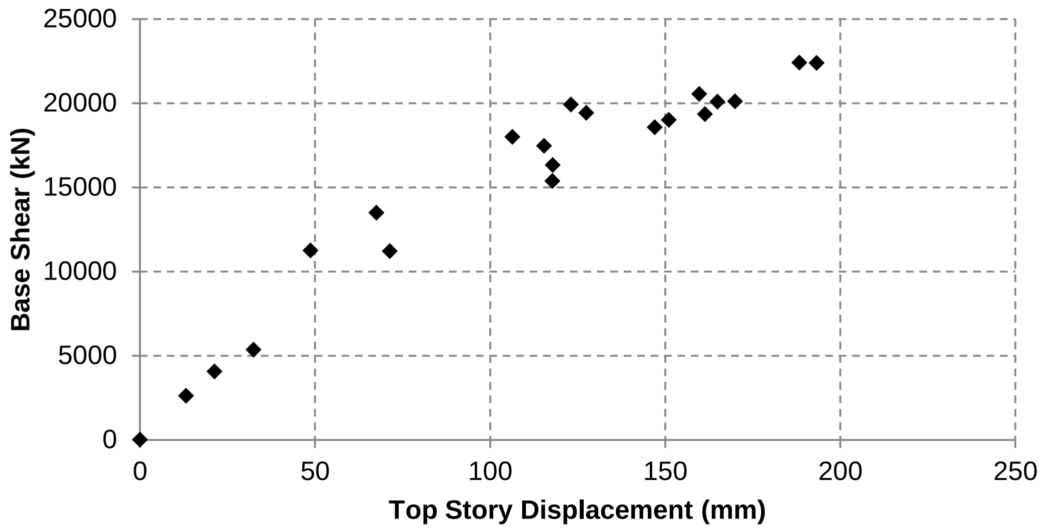

Figure 4.24 Base Shear vs Top Story Displacement relationship

4.2.1.3 Results for Panel Zone Thickness Equal to 1.0 PZ_{ref}

The interior panel zones reinforced according to 1.0 PZ_{ref}, 39 mm which was FEMA 355D recommendations for interior panel zones. The following results were obtained;

Exterior panel zones showed elastic behavior as predicted. However, interstory drifts were recorded maximum 2.5 % which there was a 20 % increase with respect to 0.8 PZ_{ref} analyses as shown in Figure 4.25.

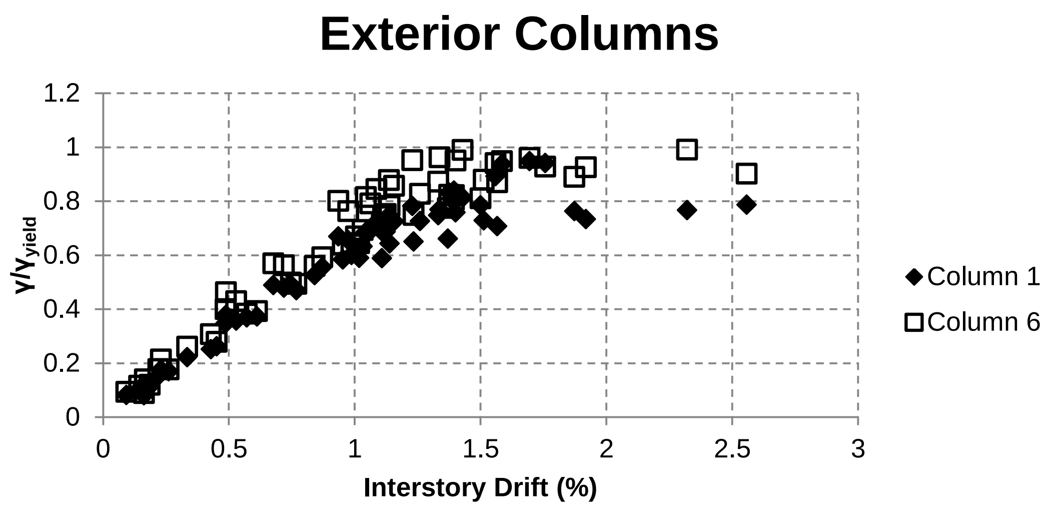

Figure 4.25 Normalized Shear Strain vs. Interstory Disp. for exterior columns

Unlike the shallow beam case, interior panel zones which were sized according to FEMA 355D recommendation experienced yielding. The maximum normalized shear strain was 1.7. However, the maximum shear strain decreased 66 % with respect to previous case as shown in Figure 4.26. The maximum top story displacement was 190 mm and the maximum base shear was 22670 kN as shown in Figure 4.27.

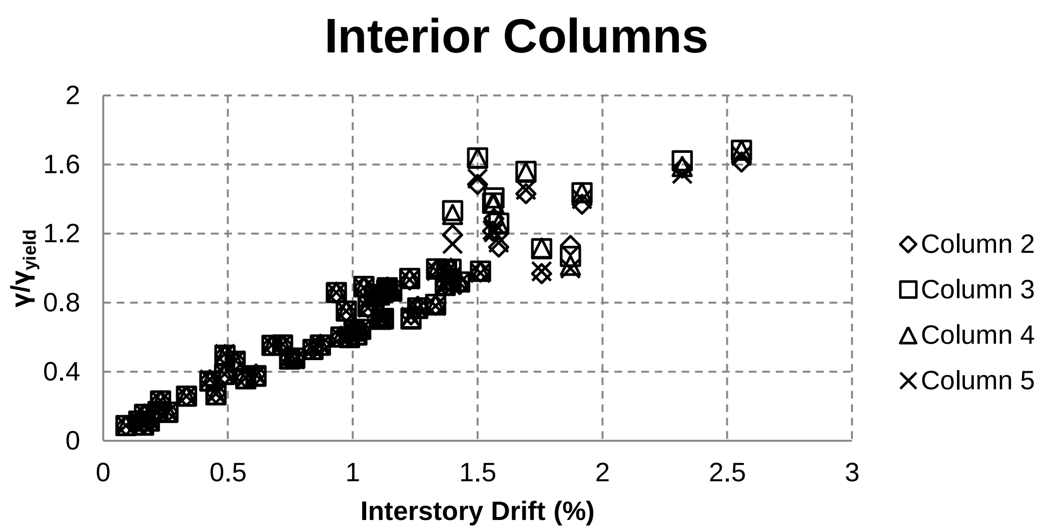

Figure 4.26 Normalized Shear Strain vs. Interstory Disp. for interior columns

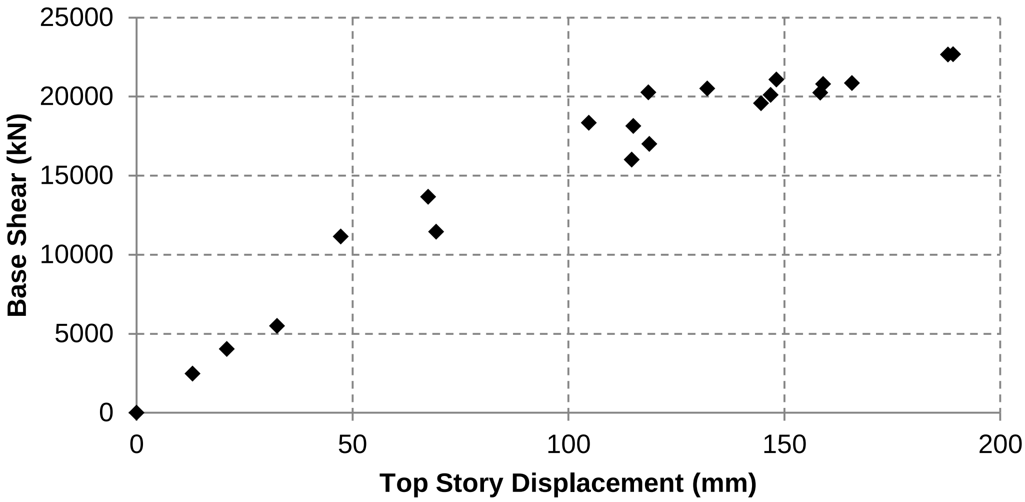

Figure 4.27 Base Shear vs. Top Story Displacement relationship

4.2.1.4 Results for Panel Zone Thickness Equal to 1.2 PZ_{ref}

In order to observe the behavior for cases with a large panel zone thickness beyond the FEMA 355D requirements, the panel zones were reinforced according to 1.2 PZ_{ref}, 44 mm. The exterior panel zones were 22 mm and the interior panel zones were 44 mm. The following results were obtained;

Exterior and interior panel zone shear strain values were less than the yield shear strain as shown in Figure 4.28 and 4.29. The maximum recorded interstory displacement was 2.5 % which was same as the previous results. The shear strain values decreased by 48 % with respect to 1.0 PZ_{ref}. The maximum top story displacement was 190 mm which was almost same as the previous result. The maximum base shear force decreased to 22670 kN as shown in Figure 4.30.

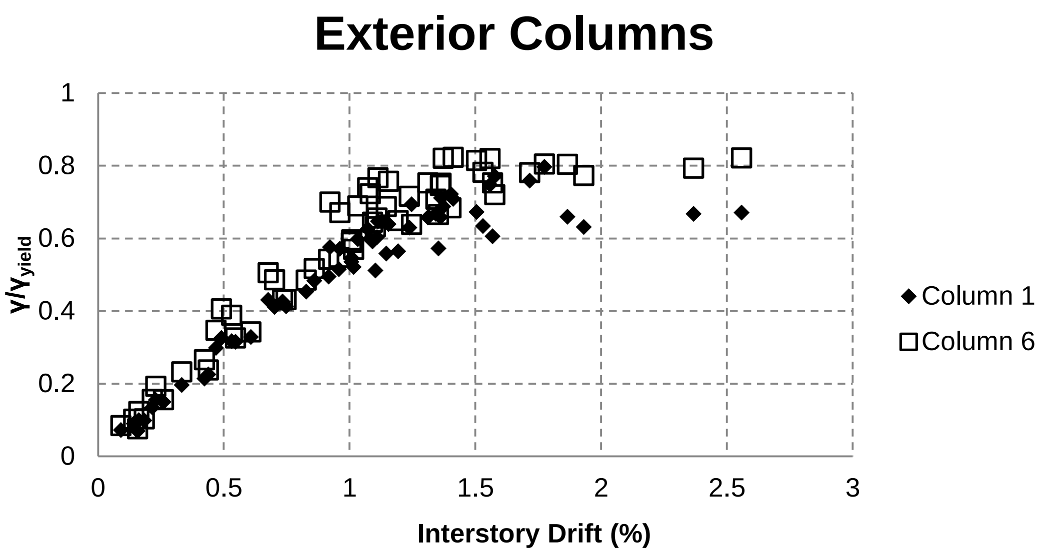

Figure 4.28 Normalized Shear Strain vs. Interstory Disp. for exterior columns

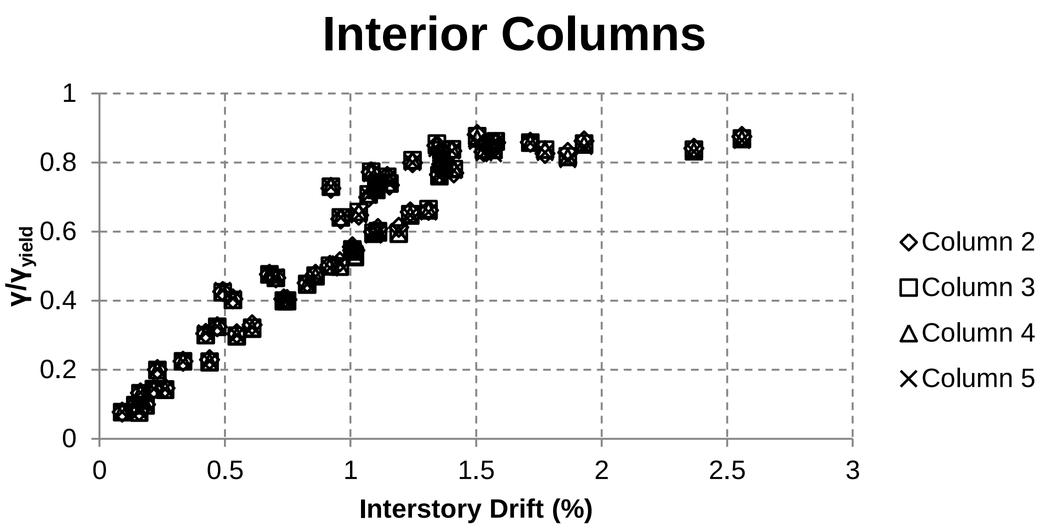

Figure 4.29 Normalized Shear Strain vs. Interstory Disp. for interior columns

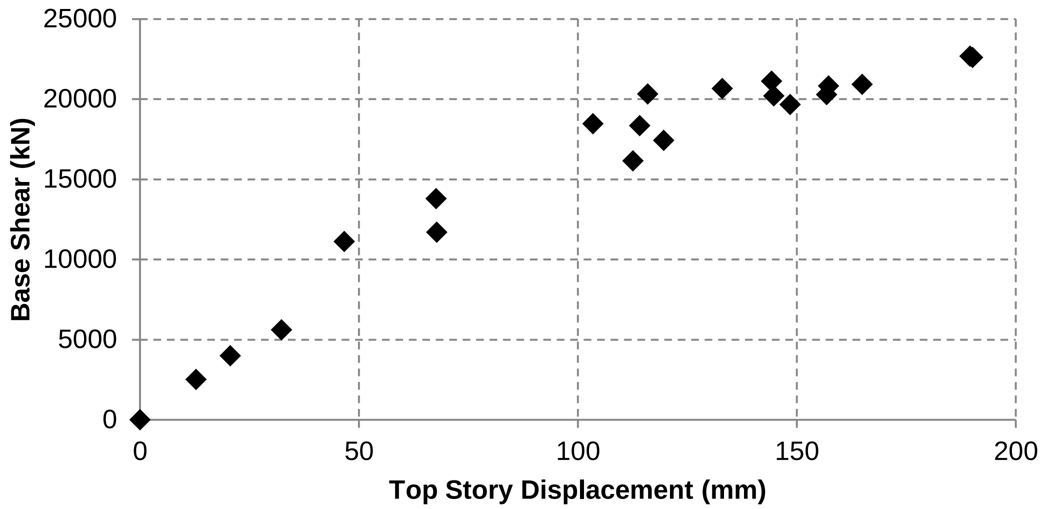

Figure 4.30 Base Shear vs. Top Story Displacement relationship

From these analyses results, it can be concluded that the unreinforced column web thickness, 0.5 PZ_{ref} was not sufficient for interior panel zones but enough for exterior ones. The panel zones have yielded up to 8 times of the yield shear strain values. There were no plastic hinge formations at the beam ends. 0.8 PZ_{ref} which was AISC recommendation for panel zones led yielding of the beam column joints. FEMA 355D specifications, 1.0 PZ_{ref} also led the panel zones yielding. The plastic hinges perfectly formed at the beam ends like the shallow beam case as shown in Figure 4.18. However, the interstory displacements obtained from 1.0 PZ_{ref}, and 1.2 PZ_{ref} analyses series were almost the same. Since unreinforced panel zones were not sufficient, increasing panel zone thickness reduced the normalized shear strains and interstory drifts decreased significantly.

4.2.2 Axial Load Level = 0.3 P_y

The previous deep beam analysis did not consider the presence of axial loads on columns. The same analysis was repeated to investigate the effect of axial load on the columns. The axial loads were placed at the top story which will subject the columns at all stories the same level of axial force. A force level of 30 percent of the axial yield load (0.3 P_y) was considered like the shallow beam case.

The results of the two sets of analysis are compared in Table 4.3. In this table, the maximums of the investigated quantities are presented. In general, there was no difference in the maximum level of base shear obtained. The level of axial load had an influence on the level of lateral displacements and panel zone shear demands. It is evident from the results that as the panel zone is designed to be weaker the influence of axial loads is less pronounced. When the panel zone thickness is equal to 0.8 or 1.0 PZ_{ref} for interior panel zones, the interstory drifts increase by 15 and 4 percent, respectively. This is also reflected by a change in panel zone deformation demands. The deformation demands (γ / γ_{yield}) increase by 12 and 45 percent for these cases, respectively. When the panel zone thickness approaches to the thickness level proposed by FEMA 355D the influence of axial load on the columns become more pronounced. The panel zone demands as a function of the interstory drifts will be evaluated in later sections of this Chapter. The results for different levels of axial load will be combined to come up with demand curves.

Table 4.3 Comparision between 0.0 P_y and 0.3 P_y axial load on columns cases

	PZ_{ref}	$(\gamma/\gamma_{yield})_{max}$			Max. Interstory disp. (%)			Max. Top Story Drift (mm)			Max Base Shear (kN)		
		$0.0P_y$	$0.3P_y$	Diff. (%)	$0.0P_y$	$0.3P_y$	Diff. (%)	$0.0P_y$	$0.3P_y$	Diff. (%)	$0.0P_y$	$0.3P_y$	Diff. (%)
Interior Panel Zone	0.5	6.5	7.4	14	2.3	2.4	4	194	197	2	20698	20727	0
	0.8	5	5.6	12	2	2.3	15	193	190	-2	20404	20416	0
	1.0	1.65	2.4	45	2.5	2.6	4	189	190	1	22668	22668	0
	1.2	0.86	1.02	18	2.5	2.5	0	185	189	2	22665	22758	0
Exterior Panel Zone	1.0	0.98	1.1	12	2.3	2.4	4	197	207	5	20698	20727	0
	1.0	0.93	1	8	2	2.3	15	193	190	-2	20404	20416	0
	1.0	0.9	0.95	6	2.5	2.6	4	189	190	1	22668	22668	0
	1.2	0.82	0.93	13	2.5	2.5	0	185	189	2	22665	22758	0

4.3 Estimation of Inelastic Panel Zone Shear Deformation Demands

The analysis results presented in the following section revealed that panel zone deformation demands are related to several parameters. The results are not significantly affected by the level of axial load. Therefore, the dependence of shear strain demands on the axial load can be neglected. Other parameters such as the panel zone thickness and interstory drift ratio are found to influence the demands significantly. An equation used to predict the deformation demands were developed as a part of this study. In order to develop such an equation data points for different axial load levels were combined. Separate plots were prepared for shear deformation demands for different normalized panel zone thickness ratios. These plots are given in Figure 4.31 through 4.35. The analysis results revealed that the demands increase as the interstory drift ratio increases. The level of normalized shear strain demand changes with the level of normalized panel zone thickness ratio. A single equation that represents the data was developed and can be expressed as follows:

$$Shear\ Distortion\ Demand = 0.66\left(\frac{t_w}{t_{wref}}\right)^{-1.52}(ISD)^{0.07\left(\frac{t_w}{t_{wref}}\right)+1.38} \tag{4.2}$$

where, t_w = column web thickness, t_{wref} = reference column web thickness (FEMA 355D), ISD = interstory drift (%).

Comparisons of the estimates are also provided in Figures 4.31 through 4.35. From these relationships it can be concluded that for typical interstory drifts of 2 percent and higher the normalized panel zone thickness should be at least 0.6 to satisfy the recommendations given by Krawinkler et al. (1971).

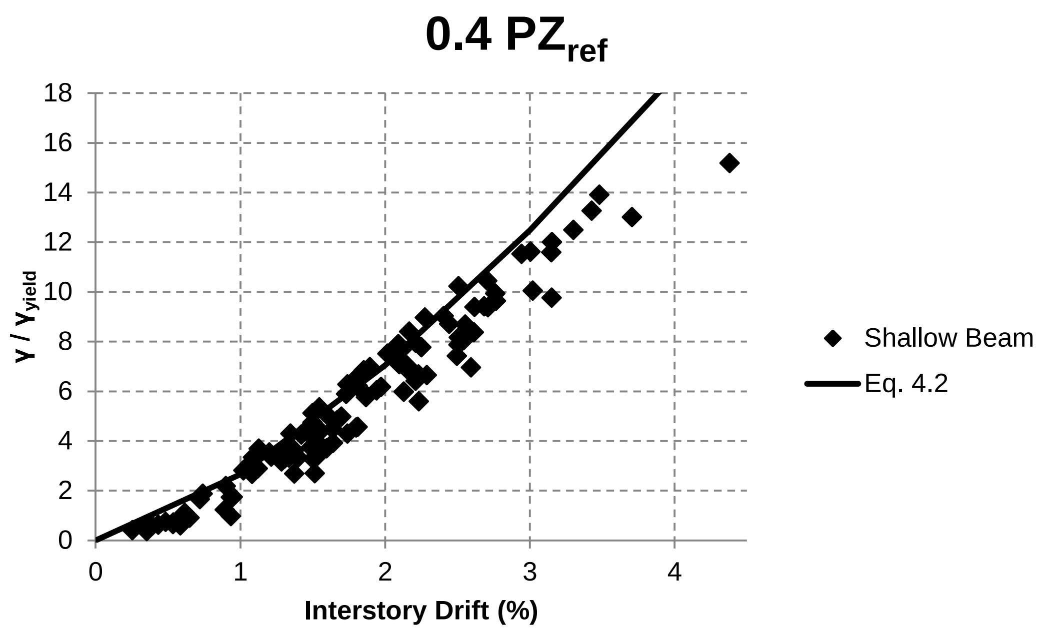

Figure 4.31 The 0.4 Case Panel Zone Shear Deformation Demands Curve

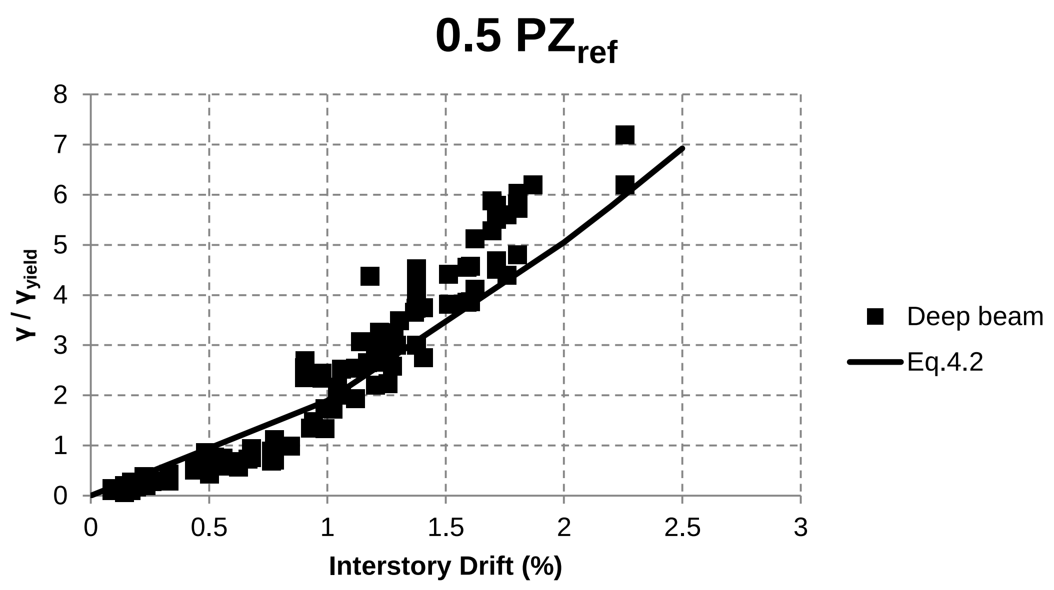

Figure 4.32 The 0.5 PZ_{ref} Case Panel Zone Shear Deformation Demands Curve

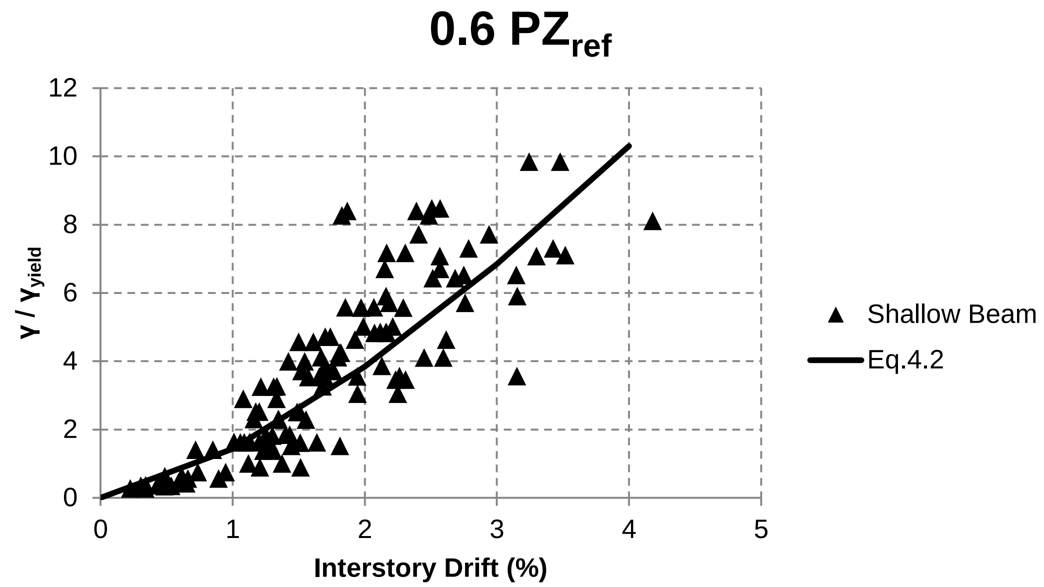

Figure 4.33 The 0.6 PZ_{ref} Case Panel Zone Shear Deformation Demands Curve

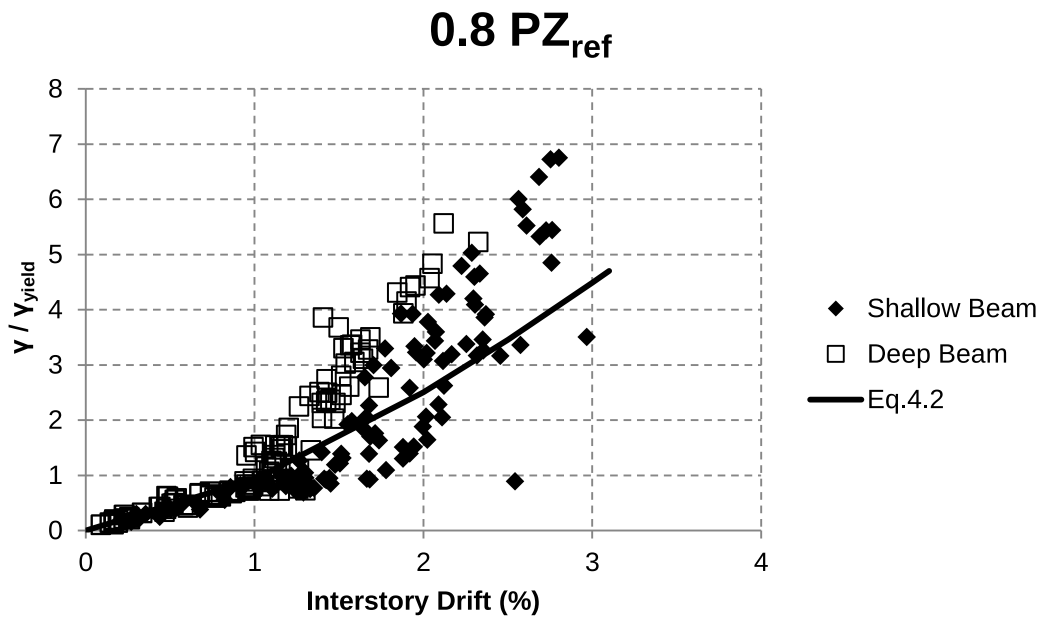

Figure 4.34 The 0.8 PZ_{ref} Case Panel Zone Shear Deformation Demands Curve

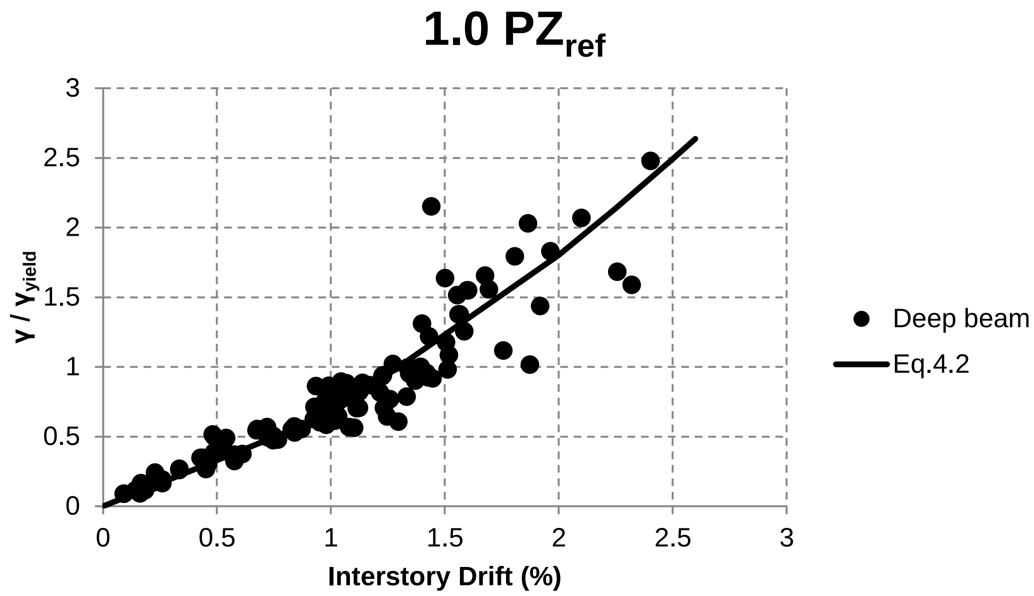

Figure 4.35 The 1.0 PZ$_{ref}$ Case Panel Zone Shear Deformation Demands Curve

CHAPTER 5

CONCLUSIONS

Panel zone deformation demands in steel moment resisting frames were studied using numerical analysis. A representative steel frame was analyzed to understand how the level of axial load on columns, beam depth, and the level of seismicity influence the deformation demands. The study encompassed different panel zone thicknesses that were determined using the provisions of different design specifications. Local shear strain demands at panel zones and the global frame deformation demands were evaluated in detail. The following can be concluded from this study:

- In this study, the actual column web thickness for the interior panel zones experienced significant amounts of yielding at both the shallow beam and the deep beam cases. However, since the requirements on the exterior panel zones are not excessive, the amount of yielding was much less compared to interior panel zones. Furthermore, the reference interior panel zone thickness responds differently to the shear strain demands at the shallow beam case and the deep beam case. At the shallow beam case, the interior panel zones remained elastic, but the interior panel zones showed inelastic behavior at the deep beam case. Because, the ratio of the beam depths is not same as the ratio of the shear force demands of these two beam sections. It must be also taken into consideration that the column properties except its depth do not participate in the shear force demand calculations for FEMA 355D, AISC Specification and Eurocode 3 design provisions. This situation results in smaller doubler plate thickness in deep beam sections compared to the shallow beam sections.

- The level of axial load had an influence on the level of lateral displacements and panel zone shear demands. It is evident from the results that as the panel zone is designed to be weaker the influence of axial load is more pronounced. However, when the panel zone thickness approaches to the thickness level proposed by FEMA 355D the influence of axial load on the columns become less pronounced.

- The shear force demand calculations between Eurocode 3, AISC Specifications and FEMA 355D are close to each other, but the panel zone capacity calculations are different as shown in Appendix A. AISC Specification and Eurocode 3 add the contribution of the column flanges to the capacity calculations, but FEMA does not. The panel zones designed according to FEMA 355D recommendations experienced the less yielding compared with the ones designed according to AISC Specification and Eurocode 3. Castro J.M., et al. (2008) also claimed that panel zone design recommendations of AISC and Eurocode lead to weak panel zone. It is recommended that at the high seismic zones, FEMA 355D provisions should be used for the design of panel zones of steel moment resisting frames.

- Panel zone deformation demands are quantified as a part of this study and these demands are represented by simple mathematical relationships that depend on the panel zone thickness and interstory drift ratio. The analysis results revealed that panel zones designed according to the FEMA 355D recommendations displayed elastic behavior. On the other hand, panel zones designed according to the AISC and Eurocode specifications showed inelastic behavior. The level of inelasticity is dependent on the level of lateral drift. In general panel zones designed according to AISC Specification experienced 5-6 times the yield shear strain at the interstory drifts of about 2 percent. While this level of inelasticity may be acceptable according to the study of Krawinkler et al. (1971), it may lead to premature failure of some type of beam-column connections. To avoid inelastic behavior, the recommendations of FEMA 355D can be followed. While some inelastic action takes place in the panel zones that are sized according to AISC Specification, the global force and displacement demands for frames with weak and strong panel zones are similar.

This study is limited to the design space developed by the methods explained in Chapter 2. Future research should consider the different connection types of panel zones. The reduced beam sections should also be studied in the future.

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APPENDICES

APPENDIX A : Panel Zone Shear Demand and Capacity Calculations

1) Panel zone design calculations for the shallow beam case are as follows:

$$Mp_{column} = 2598.6\ kNm \qquad Mp_{beam} = 1401.9\ kNm \tag{A.1}$$

According to AISC LRFD Design:

$$\sum R_u = \frac{M_{u1}}{d_{b1}} + \frac{M_{u2}}{d_{b2}} - V_u \tag{A.2}$$

$$\sum R_u = 2 \times Mp_{beam} \left(\frac{1}{d_b} - \frac{1}{h}\right) \tag{A.3}$$

Where M_{pbeam} = beam plastic moment capacity (kNm), d_b = beam depth (m), h = Story height (m)

$$R_u = 2 \times 1401.9 \left(\frac{1}{0.49} - \frac{1}{3.5}\right)$$

$$R_u = 4920.94\ kN \tag{A.4}$$

Required panel zone thickness:

$$R_n = 0.60 F_y d_c t_w \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_w}\right)$$

$$4920940 = 0.6 \times 355 \times 650 \times t_w \left(1 + \frac{3 \times 300 \times 31^2}{650 \times 490 \times t_w}\right) \qquad (A.5)$$

$$t_w = 33\ mm$$

According to FEMA 355D

$$V_{pz} = \frac{2 \times 1260.25}{0.49}\left(\frac{8}{8 - 0.65}\right)\left(\frac{3.5 - 0.49}{3.5}\right) \qquad (A.6)$$

$$V_{pz} = 4814.95\ kN$$

Required panel zone thickness

$$4814950 \leq 0.9 \times 0.55 \times 355 \times 650 \times t_w \qquad (A.7)$$

$$t_w = 42\ mm$$

According to Eurocode 3

$$V_{wp,Ed} = \frac{2 \times 1260.25}{0.49} - \frac{2 \times 1401.9}{3.5} \qquad (A.8)$$

$$V_{wp,Ed} = 4342.8\ kN$$

Required panel zone thickness

$$4342.8 = \frac{0.9 \times 355 \times A_{vc}}{\sqrt{3}} + \frac{2 \times M_{p,fc,Rd} + 2 \times M_{p,st,Rd}}{0.48} \qquad (A.9)$$

$$A_{vc} = 28630 - 2 \times 300 \times 31 + (t_{cw} + 2 \times 27) \times 31 \qquad (A.10)$$

$$M_{p,fc,Rd} = 26\ kNm \qquad M_{p,st,Rd} = 25\ kNm$$

$$t_{wc} = 30\ mm \qquad (A.11)$$

2) Panel zone design calculations for the deep beam case are as follows:

$Mp_{column} = 5275.3\ kNm$ $\qquad\qquad\qquad$ $Mp_{beam} = 3837.55\ kNm$

According to AISC LRFD Design:

$$R_u = 2 \times 3837.55 \left(\frac{1}{0.89} - \frac{1}{3.5}\right) \qquad (A.12)$$

$$R_u = 6430.8\ kN$$

Required panel zone thickness:

$$6430822 = 0.6 \times 355 \times 1000 \times t_w \left(1 + \frac{3 \times 300 \times 36^2}{1000 \times 890 \times t_w}\right) \qquad (A.13)$$

$$t_w = 29\ mm$$

According to FEMA 355D

$$V_{pz} = \frac{2 \times 3367.2}{0.89}\left(\frac{8}{8-1}\right)\left(\frac{3.5 - 0.89}{3.5}\right) \qquad (A.14)$$

$$V_{pz} = 6448.67\ kN$$

Required panel zone thickness

$$6448670 \le 0.9 \times 0.55 \times 355 \times 1000 \times t_w \qquad (A.15)$$

$$t_w = 37\ mm$$

According to Eurocode 3

$$V_{wp,Ed} = \frac{2 \times 3367.2}{0.89} - \frac{2 \times 3837.55}{3.5}$$

$$V_{wp,Ed} = 5374.53 \ kN \tag{A.16}$$

Required panel zone thickness

$$5374.53 = \frac{0.9 \times 355 \times A_{vc}}{\sqrt{3}} + \frac{2 \times M_{p,fc,Rd} + 2 \times M_{p,st,Rd}}{0.86} \tag{A.17}$$

$$A_{vc} = 40000 - 2 \times 300 \times 36 + (t_{cw} + 2 \times 30) \times 36 \tag{A.18}$$

$$M_{p,fc,Rd} = 34.51 \ kNm \qquad M_{p,st,Rd} = 69 \ kNm$$

$$t_{wc} = 26 \ mm \tag{A.19}$$