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Panel Zone Rigidity Effects on Special Steel Moment-Resisting Frames According to the Performance Based Design

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Abstract**—**The unanticipated destruct of more of the steel moment frames in Northridge earthquake, altered class of regard to the beam-to-column connections in moment frames. Panel zone is one the significant part of joints which, it's stiffness and rigidity has an important effect on the behavior and ductility of the frame. Specifically that behavior of panel zone has a very significant effect on the special moment frames. In this paper , meanwhile the relations for modeling of panel zone in frames are expressed , special moment frames with different spans and stories were studied in the way of performance-based design. The frames designed in according with Iranian steel building code. The effect of panel zone is also considered and in the case of non-existence of performance level, by changing in intimacies and parameter of panel zone, performance level is considered.

Keywords**—** steel moment frame, panel zone, performance based design

I. INTRODUCTION

After the Northridge earthquake in 1994, a number of steel moment-frame buildings were found to have brittle fractures of beam-to-column connections. Design practice before the Northridge earthquake in 1994, encouraged connections with relatively weak panel zones. In connections with excessively weak panel zones, inelastic behavior of the assembly is dominated by shear deformation of the panel zone. This panel zone shear deformation, result in a local kinking of the column flanges adjacent to the beam-flange to column-flange joint and further increases the stress and strain demands in this sensitive region. [1]

The panel zone is the region in the column web defined by the extension of the beam flange lines into the column (figure1). Lateral loads in moment frames develop high shear forces within the panel zone. The resulting deformations of the panel zone can have an important effect on the elastic and inelastic behaviours of the frames. [5] Previous research investigations have indicated that the panel zone has a ductile and stable behaviour. The concentration of some inelastic deformations in the panel zone may be employed to relieve the demand deformations on the beams. However excessive inelastic deformations in the panel zone may damage the connection and impair the global structural behaviour. Therefore, the extent of plastic deformations in the panel zone needs to be adequately assessed. [2]

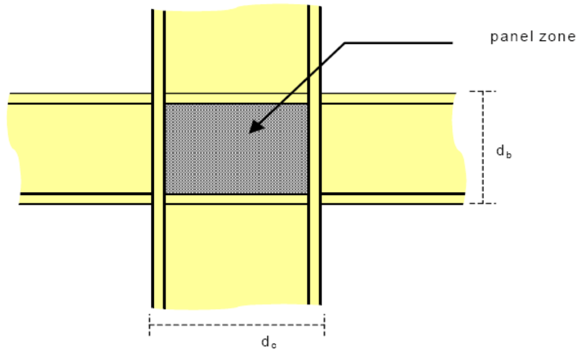

Figure 1. Definition of panel zone [13]

II. PANEL ZONE

2.1 Panel zone strength

Research performed by Krawinkler has shown that the strength of the panel zone consists of two components; shear in the panel itself, and flexure in the column flanges. The larger of these components is the panel zone shear, which is resisted by the web of the column with the stiffener plate(doubler plate) , if present. The joint panel zone shear strength can be obtained from the following formula:[8,10]

$$V_y = 0.55 F_y d_c t [1 + \frac{3 b_c t_{cf}^{\ 2}}{d_b d_c t}] \qquad (1)$$

Where b_c = the width of the column flange; d_b = the depth of the beam; d_c =the column depth; t = the total thickness of the joint panel zone including stiffener plates; and t_{cf} = the thickness of the column flange.

2.2 panel zone doubler plate

Panel zone stiffener plates (doubler plates) may be required to control panel zone yield and deformation. Stiffener plates provided to increase the design strength of the panel zone or to reduce the web depth thickness ratio shall be placed next to the column web and welded across the plate width along the top

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and bottom with at least a minimum fillet weld. The stiffener plates shall be fastened to the column flanges using either bolts or fillet welded joints to develop the design shear strength of the stiffener plate.[10]

2.3 modeling of panel zone

Most of the pioneering work on nonlinear panel zone modeling has been done by Krawinkler. A suitable model for modeling the nonlinear behavior of frames with yielding beams, columns, and panel zones is shown in Figure 2.

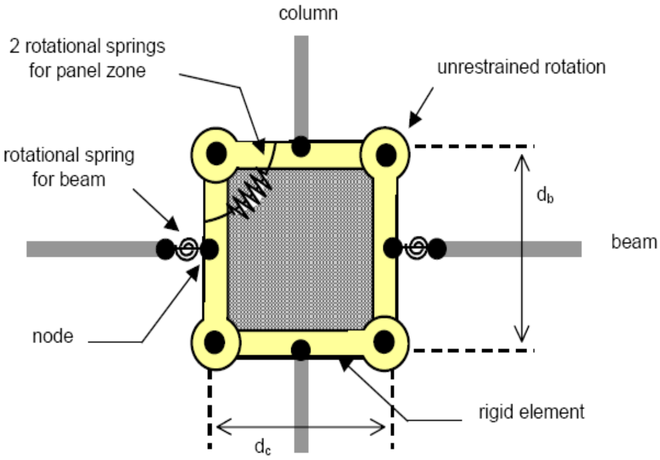


Figure 2. panel zone modeling[13]

This model holds the full dimension of the panel zone with rigid links and controls the deformation of the panel zone using two bilinear springs that work as a tri-linear behavior. The first slope past yield is steep and represents the behavior between the time that yielding is initiated and the full plastic capacity is reached. After the plastic capacity is reached, a small slope (2 %) or zero slopes may be used. This is shown in figure 3. Since yielding in the beams, columns, and panel zones is represented well by this model, the actual distribution of yielding throughout the structure will be represented well.

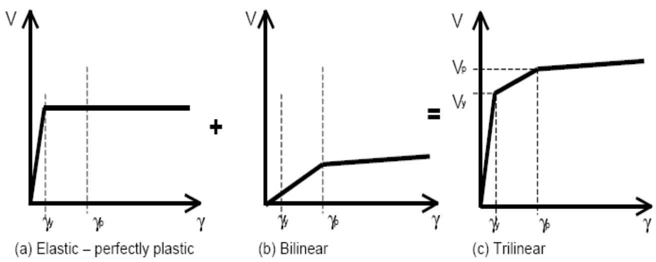


Figure 3. trilinear relationship for panel zone modeling

2.4 review on panel zone equations

2.4.1 Elastic range

Past researchers (Wang, Fielding and Krawinkler) computed the elastic stiffness of the panel element by considering pure elastic shear deformation of an effective shear area of the panel zone. They suggested the yield moment and elastic stiffness of the panel zone be taken as follows:[5]

$$M_y^{pa} = \frac{V_y d_b}{(1-\rho)} = \frac{\overline{\tau}_y A_{eff} d_b}{(1-\rho)} \qquad (2)$$

$$K_{el} = \frac{M_y^{pa}}{\gamma_y} = \frac{G A_{eff} d_b}{(1-\rho)} \qquad (3)$$

where V_y is the yield shear force of the panel zone, $\rho = \frac{(d_b - t_{bf})}{H_c}$, $\gamma_y = \frac{\overline{\tau}_y}{G}$, G is the elastic shear modulus, and $\overline{\tau_Y}$ is the Von Mises yield shear stress of the column web, based on shear and axial force interaction. The Von Mises yield shear stress, $\overline{\tau_Y}$, is taken as:

$$\overline{\tau}_y = \frac{\sigma_y}{\sqrt{3}} \sqrt{1 - (\frac{p}{p_y})^2} \qquad (4)$$

Where p and p_y are the axial force and the axial yield force on the column, respectively, and σ_y is the yield stress of the column web.

Fielding and Krawinkler considered the effective shear area A_{eff} equal to $(d_c - t_{cf}) t_{cw}$, and Wang considered the effective shear area A_{eff} of $(d_c - 2t_{cf}) t_{cw}$, where the subscripts 'c', 'f', and 'w' stand for column, flange, and web, respectively.

2.4.2 Post-elastic range

Fielding and Huang proposed a bi-linear relationship (figure4) for the panel zone behaviour in which the post-elastic stiffness K_{p-el} is defined:[5]

$$K_{p-el} = 5.2 \frac{G b_c t_{cf}^{\;3}}{d_b (1-\rho)} \qquad (5)$$

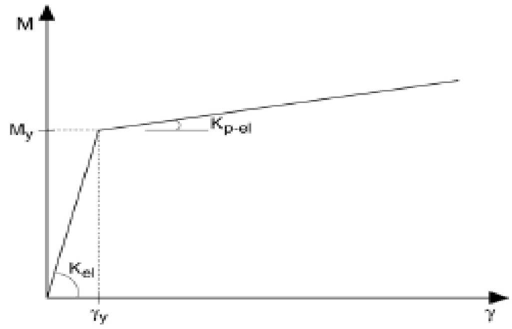


Figure4. bi-linear relationship for panel zone[2]

Krawinkler proposed a tri-linear (figure5) representation in which the post-elastic stiffness K_{p-el} and the second yield moment $M_{sh}{}^{pa}$ is:

$$K_{p-el} = 1.04 \frac{G b_c t_{cf}{}^2}{(1-\rho)} \tag{6}$$

$$M_{sh}{}^{pa} = M_y{}^{pa} + 3.12 \frac{\overline{\tau_y} b_c t_{cf}{}^2}{1-\rho} \tag{7}$$

Wang suggested the post-elastic stiffness K_{p-el}, as follows:

$$K_{p-el} = 0.7 G b_{cf} t_{cf}{}^2 \tag{8}$$

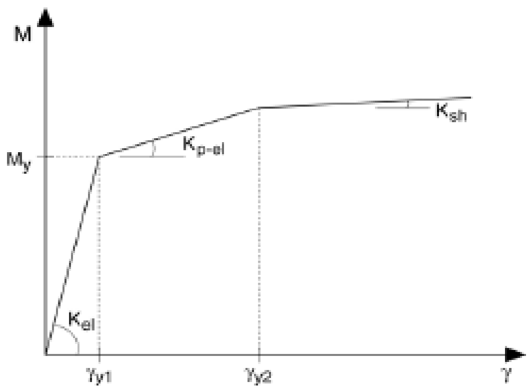


Figure5. Tri-linear relationship for panel zone [2]

Krawinkler and Wang assumed that strain hardening begins at $\gamma_{sh} = 4\gamma_y$ and $\gamma_{sh} = 3.5\gamma_y$, respectively.

The strain-hardening branch stiffness was suggested as follows:

$$K_{sh} = \frac{G_{st} A_{eff} d_b}{1-\rho} \tag{9}$$

Where G_{st} is the strain hardening shear modulus.

III. CAPACITY SPECTRUM METHOD

ATC-40 details the Capacity Spectrum Method (Freeman et al., 1975). Commonly this method applies to obtaining target displacement for performance based assessment of frames. In this approach the pushover curve is plotted as a "capacity curve," a form in which plotting occurs in the domain of modal response acceleration vs. modal response displacement, as opposed to base shear versus roof displacement. Modal displacement demand is determined from the intersection of the capacity curve with a demand curve that consists of the smoothed response spectrum representing the design ground motion, modified to account for inelastic structural response behavior.[12]

Figure 6 shows the Schematic representations of push over curves.

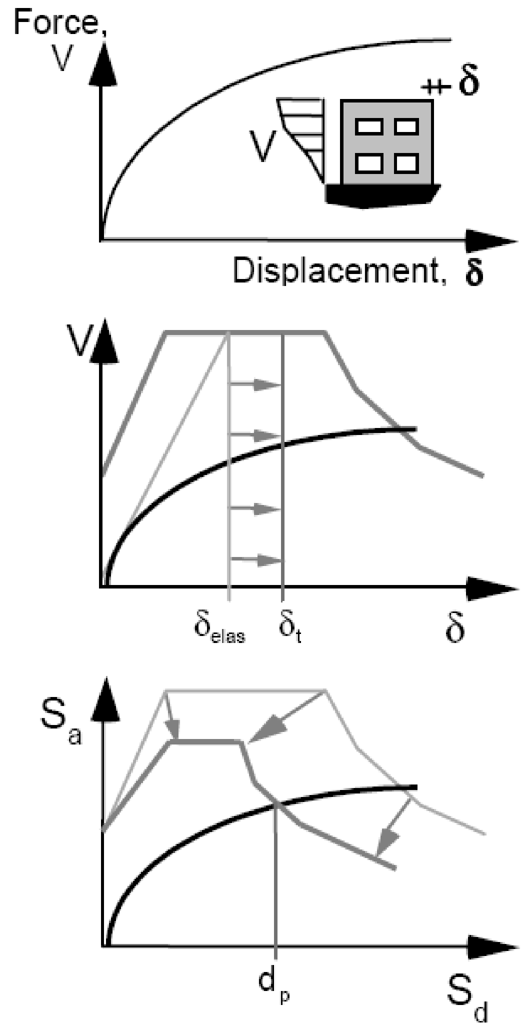


Figure 6. Schematic representations of push over curves[12]

IV. EVALUATIONS OF FRAMES IN ELASTIC REGIOUN

Most of the designs are performed according to the supposition of rigid panel zone. According to FEMA356[11] if the expected shear strength of panel zone exceeds the flexural strength of the beams at a beam column connection , and the stiffness of the panel zone is at the least 10 times larger than the flexural stiffness of the beam , direct modeling of the panel zone shall not be required. Therefore this case should be considered that if panel zone can not supply this strength and rigidity, hypothesis of rigid panel zone may cause the false estimation of forces and deformations of frame.

In this article , 2D special moment resisting frames(4 story with 3 span , 8 story with 4 span and 12 story with 6 span) studied in supposition of rigid panel zone , panel zone without doubler plate and panel zone reinforced by optimize doubler plate. Frame's behaviour were evaluated and compared without changing in beam and column profiles, by changing panel zone. In this paper, panel zone rigidity obtained by doubler plate thickness.

In all frames, width of spans is 5m and stories height is 3m and bottom story height is 2.8m. Sections for all columns are IPB and for beams is IPE. Structure designing and controlling the criterion of steel moment resisting frame was executed base on UBC97 [10] and Iranian standard No.2800 [7].

Linear analysis has been conducted using the ETABS program Version 8.4.8 and nonlinear analysis has been conducted by PERFORM-3D version 4 program. In PEROFRM-3D program, with assignment the panel zone element to the joint, Krawinkler tri-linear relation was considered for panel zone.[6] As an example, sections of beams and columns , designed for 8 story frame were showed in table 1.

	story	١	٢	٣	۴	۵	۶	٧	٨
Interior columns	IPB	۴۵٠	۴۵٠	۴٠٠	۴٠٠	٣۶٠	٣۶٠	٣٢٠	٣٢٠
Exterior columns	IPB	٣۶٠	٣۶٠	٣۶٠	٣۶٠	٣۴٠	٣۴٠	٢۴٠	٢۴٠
Beams	IPE	۴۵٠	۴۵٠	۴۵٠	۴٠٠	۴٠٠	۴٠٠	٣۶٠	٣۶٠

Table1. Sections assigned for 8story frame

4.1 Comparison between frames period

Comparison between frames period of fundamental mode in different cases of panel zone, has shown in figure 7.

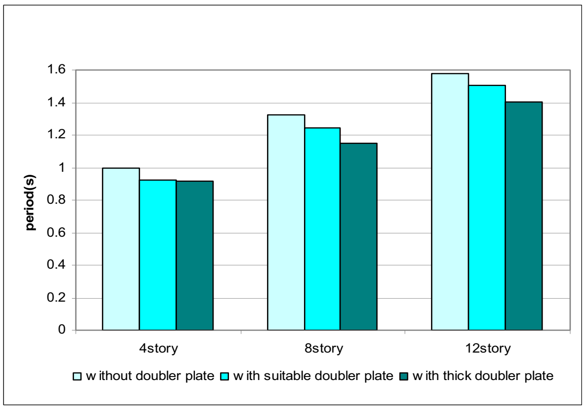


Figure 7. Comparison frames period

As it shown in diagrams, by increasing panel zone rigidity period in fundamental mode of frames is reduced.

4.2 comparisons between the story drifts

Comparison between story drifts of frames in different cases of panel zone rigidity shown in figure 8.

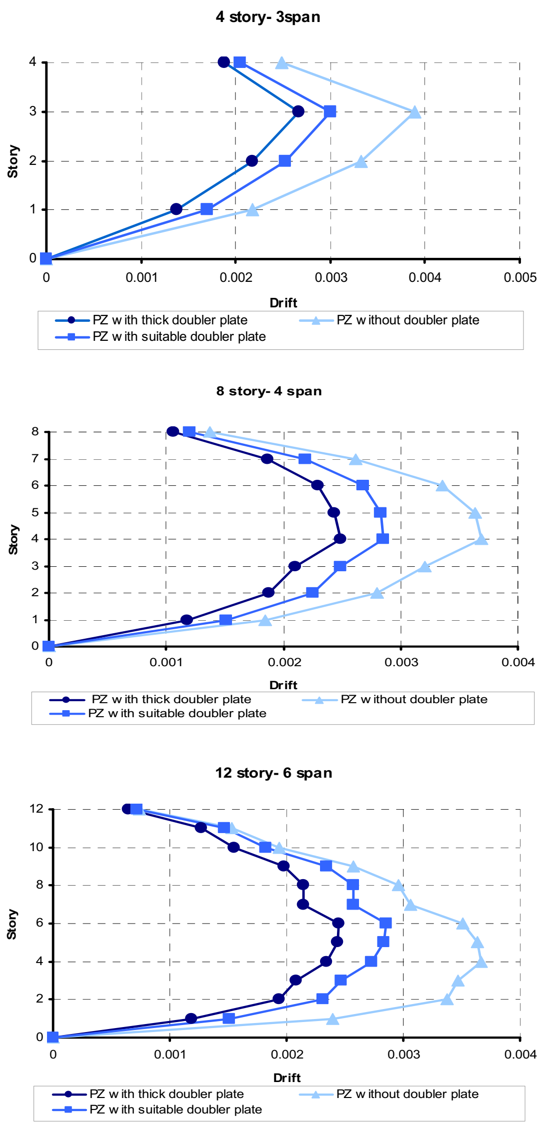


Figure 8. Comparison between story drifts of frames

As it shown in diagrams, by increasing panel zone rigidity story drifts are reduced.

V. EVALUATION OF FRAMES IN NONLINEAR RANGE

Pushover method was used for evaluating nonlinear behaviour of frames, and target displacement of frames was obtained by capacity spectrum method. Beams and columns plastic hinges and gravity load combination and lateral load distribution were attributed to frames based on FEMA 356[11] and "Instruction for Seismic Rehabilitation of Existing Buildings No. 360".[9]. Two pattern of lateral load distribution were considered for all frames. In 4 and 8 stories frames for the modal pattern, a vertical distribution proportional to the shape of fundamental mode was used, and for all frames, a uniform distribution consisting of lateral forces at each level proportional to the total mass at each level, was used for second pattern. But in 12 story frame , due to less than 75% of the total mass participated in fundamental mode , a vertical distribution proportional to the shear story distribution calculated by combining modal response from a response spectrum analysis , was used for modal pattern.

5.1 Comparisons between capacity spectrum curve of frames

Figure 8 has shown the comparison between capacity spectrums of the frames in different cases of panel zone.

As shown in diagrams, by increasing panel zone rigidity, shear capacity of frames increased.

According to the figure 9 panel zone has a significant effect on the capacity spectrum of frames with short height.

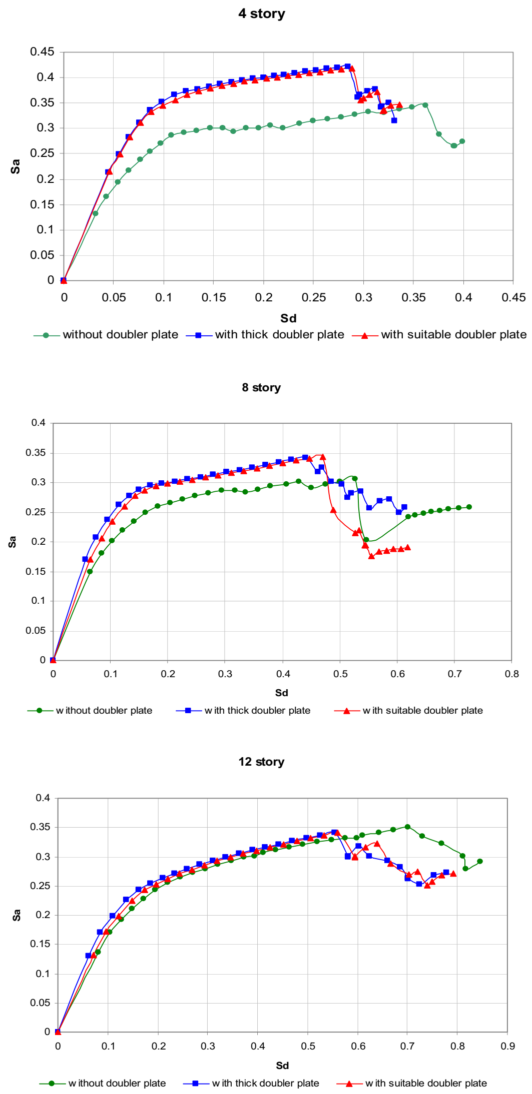


Figure 9. Comparison between capacity spectrum curves of frames in different cases of panel zone

5.2 Performance based assessment of frames

It was cleared by evaluating frames in BSE-1(~10%/50 year) earthquake hazard level[11] or DBE[9] , all frames except the 4 story frame without doubler plate , supplied life safety performance level (LS).

Figure 10 shows the performance levels of plastic hinge of frames. Numbers (1) , (2) and (3) in front of frame elements in figure 10 , are illustrative the cases of panel zone that they are: without doubler plate , with thick doubler plate and with suitable(optimum) doubler plate, respectively.

By evaluating the performance level of 4 story frame in BSE-1 earthquake hazard level , it was shown that if the frame is studied with the supposition of non-rigid panel zone and , with attribution of panel zone element in state without doubler plate joints , had passed collapse prevention performance level (CP) , whereas most of the beams.

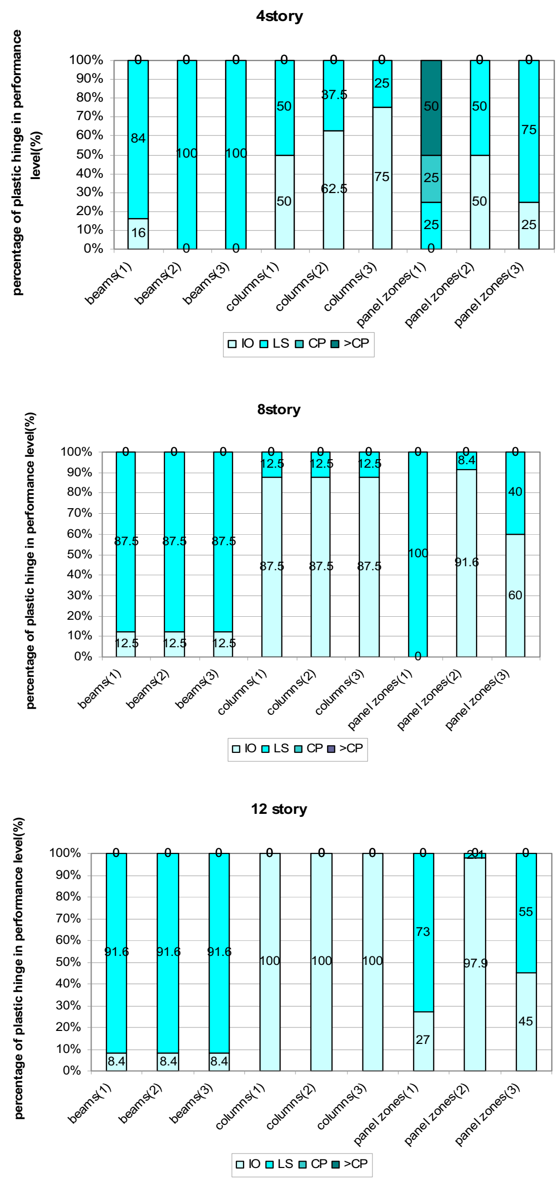
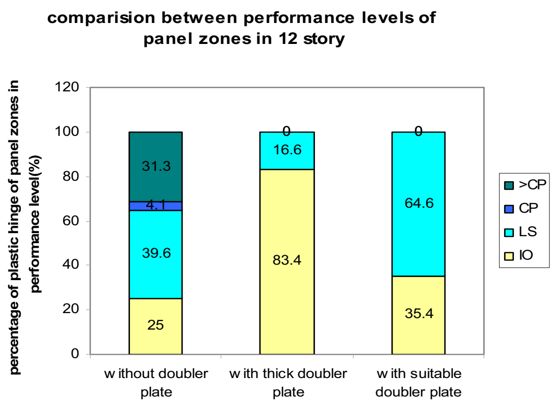


Figure 10. Performance levels of plastic hinge of frames in BSE-1 earthquake hazard level.

were in immediate occupancy performance level (IO). In the next stage doubler plates with high thickness are attributed to panel zone element and again the performance level of frame studied. It was realized that with reducing deformations in panel zones, their performance levels will reach to (IO) in most of the joints , whereas deformations in beams increased and beams were placed in more ultimate performance level than column's and panel zone's performance level.

Therefore in the next stage doubler plates will be attributed to panel zone that will develop this element performance level and on the other hand beam and panel zone performance level will be adjacent , so that plastic hinges are created firstly in the beams and then in the panel zones and finally in columns. This method of plastic hinge formation in the structural frame is the most suitable method of losing energy obtained from seismic forces and creating suitable seismic function for structure.

Capacity spectrum of 4 and 8 story frames had not passed demand spectrum of BSE-2(~2%/50year) earthquake hazard level or MPE [9]. Target displacement of 12 story frame was found in BSE-2 earthquake hazard level and performance levels of plastic hinges shown in figure 11.



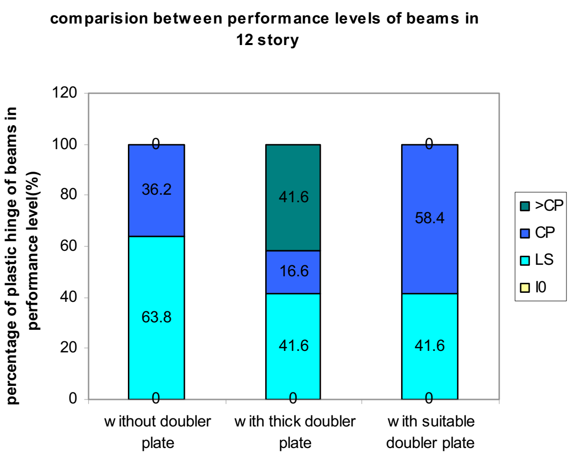


Figure 11. Performance level of plastic hinges of beams and panel zones of 12 story in BSE-2 earthquake hazard level.

only 12 story frame with optimum panel zone doubler plate was supplied collapse prevention performance level (CP).

VI. CONCLOSIONS

In frames with weak panel zone area, that the story drifts are more than permitted rate according to standards, the story drifts could be developed by reinforcing panel zone by doubler plate.

Panel zone rigidity has important effect on other elements of structure like beam. In evaluating performance level of the most considered frames, panel zone has performance level more critical than other elements like beam. By assignment doubler plates in the panel zone, increasing panel zone rigidity reduces deformations of this element and develops it's performance level. But it should be considered that high thickness of doubler plates and high rigidity of panel zone will increase nonlinear deformation rates in beam element. Therefore doubler plates should be attributed to the panel zone so that after creating plastic hinge in beams and losing most part of energy due to earthquake by this element, the panel zone can prevent beam destruction by losing earthquake energy with nonlinear deformations.

By increasing height of frames, the frames will show better function.

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