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Modeling of PZ in Haunch Connections Systems

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***Abstract*—**Modeling of Panel Zone (PZ) seismic behavior, because of its role in overall ductility and lateral stiffness of steel moment frames, has been considered a challenge for years. There are some studies regarding the effects of different doubler plates thicknesses and geometric properties of PZ on its seismic behavior. However, there is not much investigation on the effects of number of provided continuity plates in case of presence of one triangular haunch, two triangular haunches and rectangular haunch (T shape haunches) for exterior columns. In this research first detailed finite element models of 12tested connection of SAC joint venture were created and analyzed then obtained cyclic behavior backbone curves of these models besides other FE models for similar tests were used for neural network training. Then seismic behavior of these data is categorized according to continuity plate's arrangements and differences in type of haunches. PZ with one-sided haunches have little plastic rotation. As the number of continuity plates increases due to presence of two triangular haunches (four continuity plate), there will be no plastic rotation, in other words PZ behaves in its elastic range. In the case of rectangular haunch, PZ show more plastic rotation in comparison with one-sided triangular haunch and especially double-sided triangular haunches. Moreover, the models that will be presented in case of triangular one-sided and double-sided haunches and rectangular haunches as a result of this study seem to have a proper estimation of PZ seismic behavior.

***Keywords*—**Continuity plate, FE models, Neural network, Panel zone, Plastic rotation, Rectangular haunch, Seismic behavior

I. Introduction

THERE has been a lot of research on haunch connection system, the necessity of addition of the haunch system came true when it has been seen many defect point in common connection in the pre Northridge earthquake, and this was a threat which was potentially reduces the safety level of buildings. The performance of these buildings effectively invalidated the building code and professional practices used before the earthquake for the seismic design and evaluation of steel moment frame structures. an immediate consequence was the wide spread recognition that it was not clear how damaged buildings could be reliable or economically repaired, how undamaged buildings should be retrofitted or how new steel moment frame structures should be design. The common defect and failure model in WUF-B connection was brittle failure of weld line which was located between beam flange and column flange. In other word, most damage occurred at the typical welded flange-bolted web moment connection detail that was widely used in construction prior to the Northridge earthquake. Experimental research on existing steel beam to column connections presents a major part of the effort to characterize and understand what has happened to steel moment frame buildings in the Northridge earthquake, accordingly, a set of full scale and medium scale beam column connections, constructed following the pre-Northridge practice, was tested to evaluate the performance of the standard connection and the adequacy of current repair and retrofit methods.

A series of experimental tasks consisted of test of repaired common connection of first series in pre Northridge. The repaired term was modification of connection after failure of connection. The fracture of moment connection in the Northridge earthquake exhibits a variety of origins and paths. In general, fracture was found to initiate at the root of beam flange CJP weld and propagate through the beam flange, the column flange, or weld itself. In some instances fracture extended through column flange and web. The backing plate which was generally left in place, produced a mechanical notch at the weld root. Cyclic test were conducted on twelve specimens constructed by SAC joint venture and also other some experimental works has been done by popov [1], whittaker [2], blonodet [3], Engelhardt [5]. Fragile behavior of WUF-B observed in SAC experiments, resulted in change of formation and connection types, therefore some kind of modification were made to connections, such as cover plate, one sided haunch and double sided, to improve the performance of connection.

The fracture which happened near groove weld of beam flange to column and propagation of cracks into the beam web close to the shear tab and first bolt hole can cause some kind of modification in the term of repaired in WUF-B connection. Modification consisted of re-welding of some welds line which collapsed during first test or removal of deformed or cracked plates and substitution of new one. Result of such a repayment could appear questionable and it may be poor.

Uang and bondad [7] studied the effect one sided bottom haunches one repaired specimen to improve the performance of damaged WUF-B connections. The philosophy of using any type of haunch is going away the plastic hinges and large deformation from welding line close to column flange, These implementations could improve the total rotation of beam to 0.03 radians and sometimes it reaches to .04in some specimen.

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In these specimens, fractures occurred when beam web buckled. Other modes of fracture have been reported by popov and blondet [4]. They reported fractures happened on bottom beam flange at the end of the haunches at stiffener. Because of using continuity plate which is provided in PZ in front of the haunch flanges and strut action of haunch flanges, PZ of this kind of connection could not reach to significant plastic rotation. This matter is intensified when double haunches. There have been a lot of experiments on the double sided haunches, shuey and engelhardt [5] studied double sided triangular haunches, usually there is no failure mode on this type of connection and test always stops because of equipment. The total rotation on these specimen ,commonly, reaches 0,06 radians and top flange and bottom flange reach to their capacity to case partial buckle close to end of two haunches and stiffeners. Another type of haunches rectangular haunches (T shape haunch) which is used either one sided or double sided haunches. This kind of haunch may improve performance of connections. Experiments show that total rotation reaches .06 radians and beam rotation could reach 0.03 radians. The failure modes of ruptures are consisting of fracture of top flange weld and later fracture of shear tab bolts or top flange buckling and fracture in beam HAZ (heat affected zone), moreover; separation T shape haunch and beam bottom flange conjunction at weld area. This article tried to categorize plastic and elastic rotation according to type of connections and make a relation between mechanical properties and some rotational properties, and then using the extracted data to propose a model due to type of haunches.

II. Finit Element Modeling And Material Properties

Models selected for verifying the numerical modeling were taken from Popov [7], Whitakker [8], Blondet [4], Engelhardt [5] and Uang [6] and SAC joint test experiments [9,10]. The objective of those experiments was to investigate and to improve post-Northridge connection performance like modification of weld procedure as well as connections geometries. The test set up of these researches included one beam and a column resembling an exterior joint, in which beam web was connected to column through shear tab and bolt. The beam end was simply supported and applied load on the center of end beam imposed moment on connection. In addition, the end of column was simply supported in two ends. ANSYS [10] multi-purpose finite element modeling code is used to perform the numerical modeling of connections. FE models were created using the ANSYS parametric design language. The geometrical and mechanical properties of the connection model were treated as parameters, for example Yield strength (F_y) and ultimate strength (F_U). Numerical modeling of connection was done including following considerations. Using eight-node-first order SOLID45 elements and bolt shanks were modeled using SOLID64 element. ANSYS can model contact problem using contact pair element: CONTA174 and TARGE 170 which work together in a way that there is no penetration occurrence during the loading process. The interaction in adjacent surface between shear tab and web were modeled using mentioned contact element. Bolt heads and nuts were modeled as hexagonal and similar to real shape to simulate the frictional forces. Coulomb coefficient is assumed as 0.3.

III. Verification Of Finite Element Model

To evaluate the accuracy of finite element modeling approach, 12 finite element models were created according to actual test. For example, analytical and experimental hysteretic behaviors of beam plastic rotation versus applied moment are shown in Figs. 1 to 6.specimen UTA-1R is a one sided haunches. Specimen UCB-RN3 is a double sided haunch. Specimen UCSD-6 is a T shape haunch which hysteretic behavior is presented in Figs 1 to 6 as samples. From these figures, it can be seen the results obtained from finite element (F.E.) models have good agreement with test data. Differences between the numerical simulation and test result may be the result of several causes like numerical modeling simplification, test specimen defect or residual stress. It is worth mentioning that .The differences between the test data and the numerical models will grow in nonlinear portion of the curve. From figs. 1to 6 it can be seen that for specimen UCB-RN3, UTA-1R, UCSD-6 and the differences between the test data and the finite element modeling is noticeable. These differences are most likely rooted in test specimen defects like geometrical measurement or slippage in lateral bracing or supporting systems of test set up. Totally, it can be seen that analytical models have good agreement with test results and the results could be reliable for evaluating the PZ. In addition, the backbone curve of analytical moment and experimental Model of moment versus PZ shear strain are shown in Fig 7,8and 9for each categories of connections as samples. These curves were used for training neural network program, and new curve as Neural Network (N.N) output were reread from NN. The NN was consisted of three-layered PERCEPTRON net. All of the three data groups besides extra FE model which is modeled for filling the data gap (pointed out in next chapter) were the basis of behavioral model [11]. 16 models were used to train neural network and then 5 models were used to test the NN results. Finally, all of the curves were reread from N.N which can be seen in figs.7 to9. By considering the curves, one can see that N.N curves have good agreement with FE and Test Results. Data of training in this NN consist of shear ratio V_y/V_{PZMy}, full plastic moment (Mp) and type of connection.

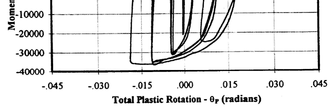


Fig. 1 Hysteretic behavior of test (UTA-1R) [10]

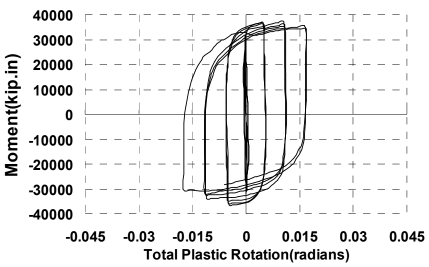


Fig. 2 Hysteretic behavior of numerical model (UTA-1R)

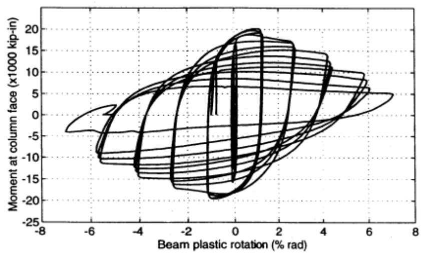


Fig. 3 Hysteretic behavior of test (UCB-RN3) [9]

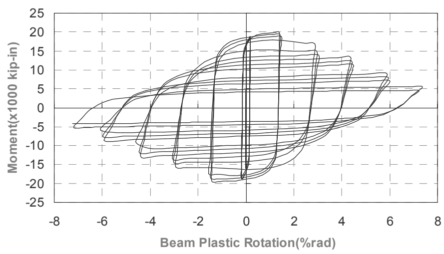


Fig. 4 Hysteretic behavior of numerical model (UCB-RN3)

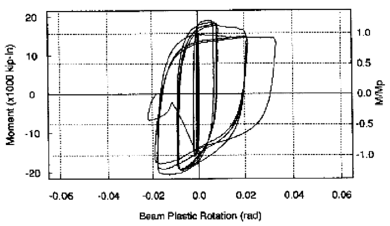


Fig. 5 Hysteretic behavior of test (USCD-6) [9]

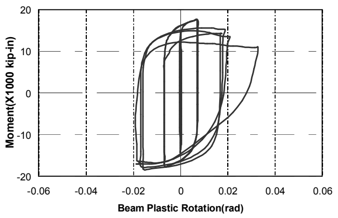


Fig. 6 hysteretic behavior of numerical model (USCD-6)

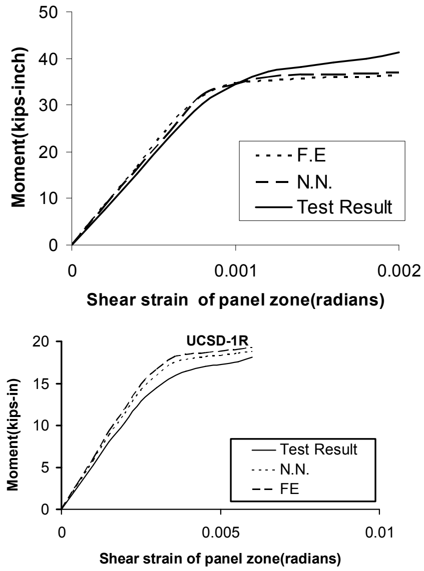


Fig 7 Result of FE and N.N. and experimental results for one sided haunch for specimen UCB-R2 and UCSD-1R

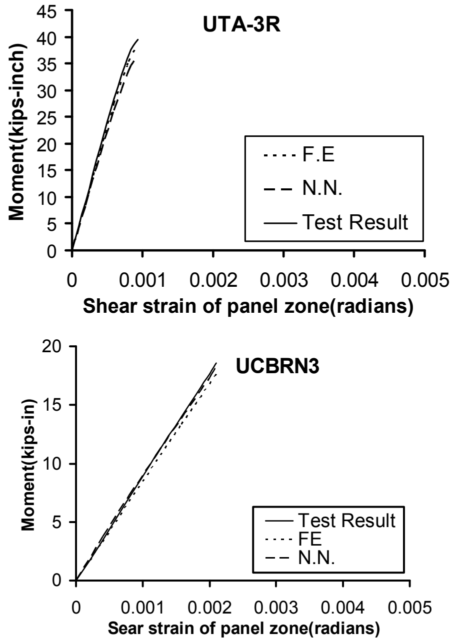
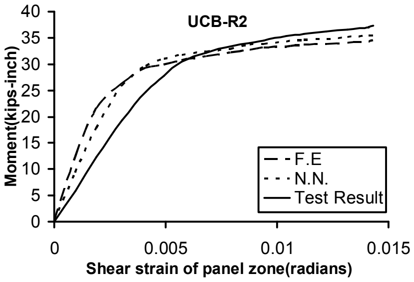
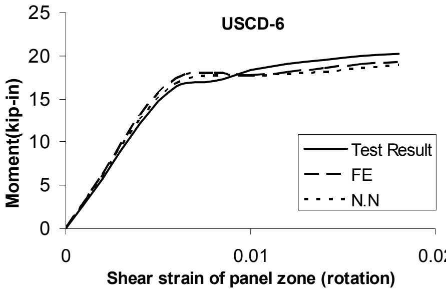


Fig. 8 Result of FE and N.N. and experimental results for double sided haunch for specimen UTA-3R and UCBRN3



(A)



(B)

Fig. 9 Comparison between result of FE and N.N. and experimental results for T shape haunch for specimen UCB-R2 &UCSD-6

There are different methods for determining yield point of panel zone. One of these methods is drawing two tangents at the beginning and end of backbone curve and finding the collision point of the two tangents as the yield point.

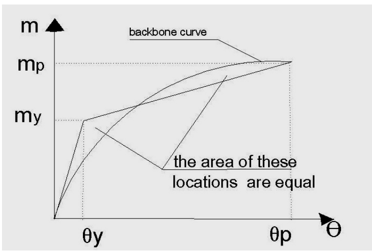


Fig. 10 Typical moment-rotation curve of connection and its reference moment

IV. Considering of Partial Ductility Ratio of One Sided Haunch Connection on PZ Seismic Behavior

Some experimental works including one sided-haunch connections have been considered. In most of the specimen, which were modeled in this study, a vertical plate could be seen in front of haunch flange on beam web. The most important reason to do this is preventing extension of the plastic area to the connection surface at column. The FE model and Von-Misses stress distribution of this model is shown in the Figs. 11 and 12.also PZ plastic and elastic rotation were shown in Figs. 13 and 14.considering this figures the trend are decreasing .there is little plastic rotation on PZ.

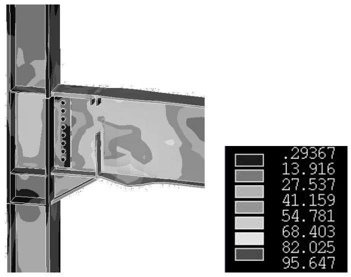


Figure 11 Von Misses distribution (ucsd-3R)

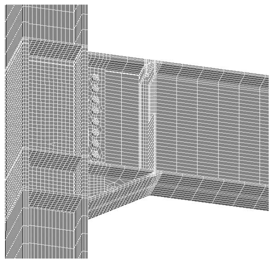


Fig. 12 FE model of specimen (ucsd-3R)

Because of lack of information, it was necessary to produce enough data; therefore, six FE models were created which were similar to six experimental works in this category and the differences was material properties, in order to cover the data gap besides data of the main model which is presented in Figs. 13 to14. Also partial ductility ratio is shown in In conclusion presence of cover plate could enhance nonlinear properties of PZ. V_{PZMy} is shear which is transferred from beam:

$$V_{PZMy} = \frac{\Sigma M_y}{d_b}\left(\frac{L}{L-d_c-2L_h}\right)\left(\frac{h-d_b}{h}\right) \quad (1)$$

Where V_{PZMy} is shear which is transferred from beam to PZ, L is beam length, h is column height , d_b is beam section depth and d_c is column section height, also M_y is the elastic moment capacity and M_p is ultimate plastic moment of the beam L_h is Haunch length.

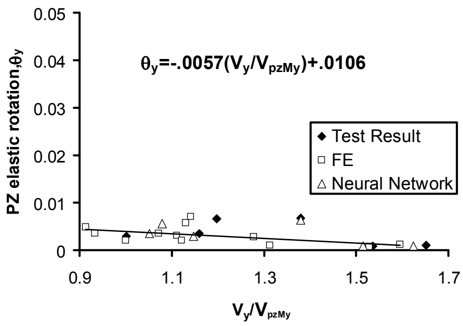


Fig. 13 PZ plastic rotation versus (V_y/V_{PZMy})

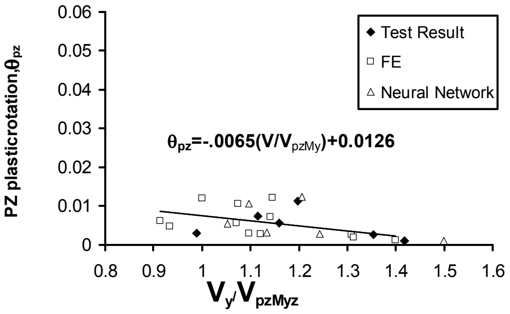


Fig. 14 PZ elastic rotation versus (**V_y/V_{PZMy}**)

The strength ratio (M_m / M_p) for this group is shown in the Fig.23. As it is obvious for all values of (V_y/V_{PZMy}), strength ratio (M_m / M_p) is bigger than 1.0. It shows that this type of connection cause the moment, reaches full plastic moment before fracture. The average of strength ratio (M_m / M_p) is 1.15. It means that this type of connection allows the beams to enter strain hardening before occurrence of collapse, which may allows for moments greater than M_p according to following equation:

$$M_{\max imum} = Z.\frac{F_{yb} + F_{tb}}{2} \qquad (2)$$

Where, Z is plastic modulus of beam section, F_{tb} is expected tensile stress of steel, F_{yb} is yield strength of steel, and M_{max} is the maximum moment before fracture. Further studies [16, 17] show that (M_m / M_p) could reach 1.26 to 1.42, the result of this section confirms this matter. PZ plastic rotation versus (V_y/V_{PZMy}) for one sided haunch connection is shown in fig 24. As it is mentioned, plastic rotation does not reach 0.02 radians and even it is less than 0.015, also PZ elastic rotation versus (V_y/V_{PZMy}) is shown in fig 25. In conclusions, PZ in this type of connection does not enter plastic area significantly. The most important causes of this phenomenon are three continuity plates and strut action of the haunch which restraints the boundary of PZ. In other words it means that it is stiffened.

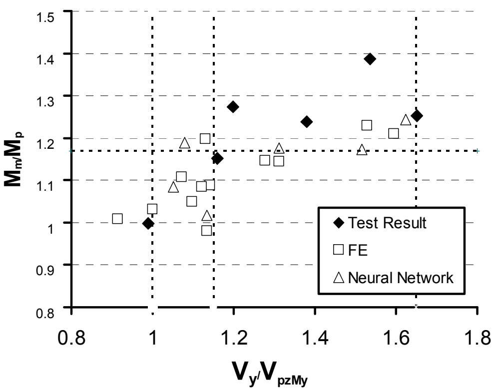


Fig. 15 M_m / M_p ratio versus (V_y/V_{PZMy})

V. Considering of Partial ductility ratio of double sided haunch connection on PZ seismic behavior

Another type of haunch is double-sided haunch. Von-Misses stress distribution for model UCB RN3 is shown in Fig. 14. The cyclic performance of beams, which was reinforced by this type of haunch, as it can be seen in Fig. 8 is essentially linear.

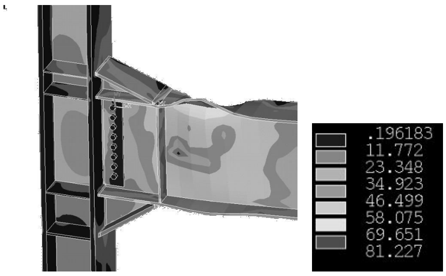


Fig. 16 Von Misses distribution (UCB RN3)

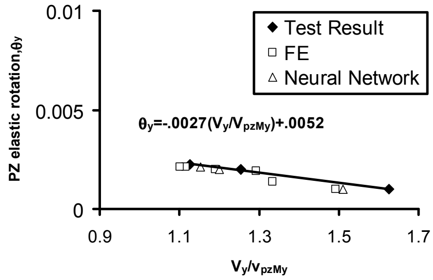


Fig. 17 PZ elastic rotation versus (V_y/V_{PZMy})

To get reliable results, some FE models (3 extra models) with variety of strength of PZ and beam to produce various (V_y/V_{PZMy}) ratio regarding geometry of three main specimens in the double-sided haunch case were built, and results of analyses were the basis of conclusions. In addition, elastic rotation of this type of connection is negligible in comparison with one-sided haunch as shown in the Fig. 16. Considering Fig. 15, it can be realized that slope of regression line in this group is less than other connections studied here. In other words, dependency of PZ seismic properties, in these connections, to shear ratio is negligible. It may be because of presence of four continuity plates and strut action of two flange haunches, on the PZ of this ensemble, which would stiffen the PZ considerably, as a result, especially in double sided haunch, there is no absorption of energy. it is worth mentioning that in these kind of connection ,there is not plastic rotation. In other word strut action of haunches and presence of four continuity plate prevent PZ enter the plastic area.it indicates that PZ elastic rotation has little dependency on (V_y/V_{PZMy}). The average value of strength ratio (M_m / M_p) of PZ in this category is about 1.17 and minimum value is 1.10. It means that beam moment in this type of connection could reach up to 1.1 of the full plastic capacity. It could be realized from Fig. 28 that slope of regression line in this group is less than others. It means that dependency of PZ seismic properties on shear ratio is negligible in these connections. They might be effects of presence of four continuity plates and strut action of two flange haunches, on the PZ of this ensemble, which stiffened the PZ considerably, especially in double sided haunch; there is no absorption of energy as a result.

VI. Considering of Partial Ductility Ratio of T Shape Haunch Connection on PZ Seismic Behavior

Another type of connection which is investigated in SAC project is T shape haunch connection, also it is worth mentioning that in this investigation one sided T shape haunch and double sided haunches are considered. These categories consisted of specimen UCSD-6, UCBAN1 and RFSR2 which is originated from popov [7], Whittaker [2] and Uang [7] experiments. Usually, the stress concentration at the free end of the haunch eventually led to the haunch separating from the beam. Also, the web stiffeners did not push the critical zone away from the column face along the beam top flange. As it was stated in reported tests, the beam top flange fractured at column face due to high curvature of the buckled beam flange. In this section, as it is depicted three of FE main models (UCSD-6, UCBAN1 and RFSR2) have been built in addition for covering the gap of data three extra FE were built. Then backbone curve of PZ hysteretic behavior of these models were the basis of training N.N in addition to other type of haunch connection. FE model of specimen UCSD-6 and Von Misses stress distribution are shown in Figs. 18 and 19.

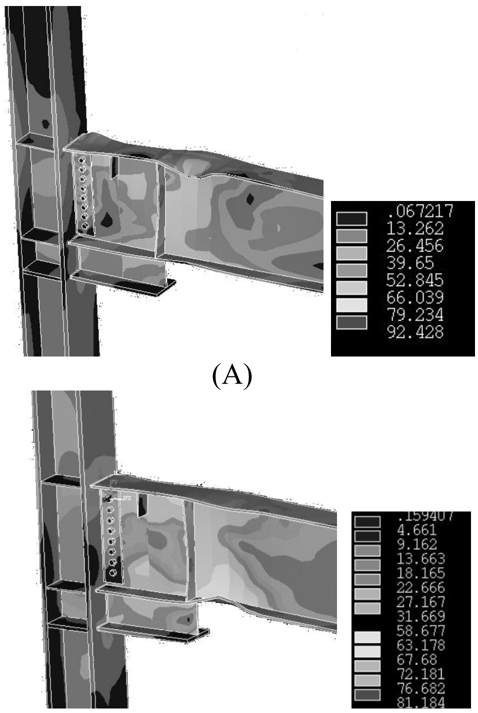


Fig. 18 Von Misses stress distribution of model ucsd-6 (loading step 15and 16)

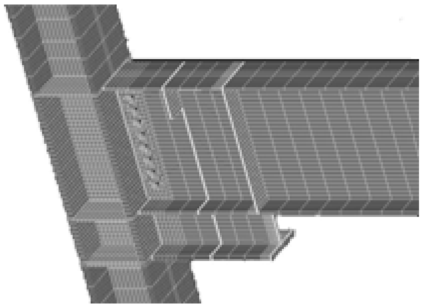


Fig. 19 FE model base on UCSD-6

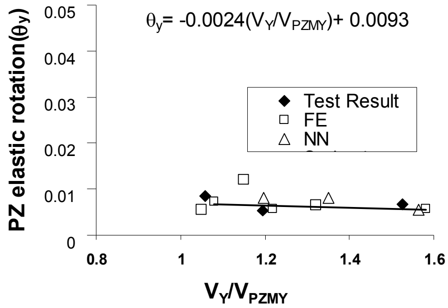


Fig. 20 PZ elastic rotation versus (V_y/V_{PZMy})

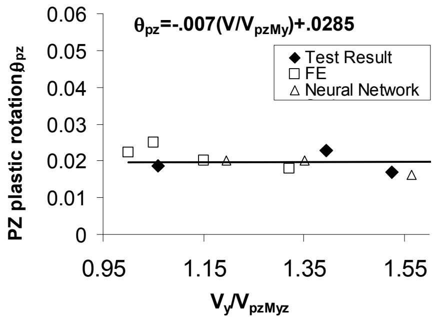


Fig. 21 PZ plastic rotation versus (V_y/V_{PZMy})

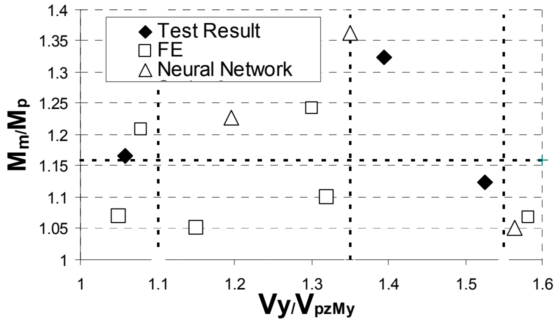


Fig. 22 M_m / M_p ratio versus (V_y/V_{PZMy})

Besides, the elastic rotation of PZ versus (V_y/V_{PZMy}) are shown in Fig. 20 .in this figure it is seen that the slope of regression line is milder than one sided haunch system but range of variation of elastic rotation of PZ is similar to one sided haunch; while, the plastic rotation of PZ are higher than one sided haunches(nearly two times) according to Fig. 21 and ;the slop of plastic rotation versus ($\mathbf{V_y/V_{PZMy}}$) for T shape rectangular haunch connection and one sided haunch connection nearly are the same. But because of higher range of plastic rotation in T shape haunch. Fig. 22 shows M_m / M_p ratio versus (V_y/V_{PZMy}), it indicate if M_m reach to M_p before the connection fails or not? It is depicted that all connection could reach the M_m to $M_{p,}$ on the basis of this Fig the minimum of M_m / M_p ratio is 1.05 and the average is nearly 1.16, it means that for every (V_y/V_{PZMy}) ratio this type of connection could provide the situation in which beam reaches their full plastic moment and sometime uses strain hardening capacity.

VII. Proposed PZ Connection Model and Analysis Implication

A few researchers have proposed models of PZ for all type of connections in the past. In order to simulate the hysteretic behavior of the post-Northridge connections, the regression line for all groups were used to create a PZ model. This study attempts to predict the PZ behavior with respect to connection type (Fig. 23). Totally, the emphasis is on PZ and the beam seismic behavior that can be obtained from backbone curve of beam in experimental work. Thus, the proposed connection can account for the inelastic panel zone performance as well as connection fractures. Detail of the previous two-bilinear PZ model can be found in Gupta and Krawinkler [11]. Because of focusing on the PZ seismic behavior in this study for three types of connections (one sided haunch, double sided haunches, and T shape haunch), modeling of beam was ignored, and the backbone curve of beam and column, which were obtained from test results, was considered. The slope of nonlinear phase for PZ can reach to 6% according to FEMA355D [12], but it is validated when the PZ of WUF-B connection is considered. For these kind of connection the average of M_m / M_p as an end point and $\mathbf{V_y}$ as a start point of nonlinear phase are used. The connection model proposed in this study verified by comparing hysteretic curves obtained from analyses with those from test of a specimen for each group. Drain-2DX program was used for conducting analyses. As observed in Fig. 23 to 26, the panel zone with T shape haunch (UCSD-R2) has significant plastic rotation while PZ with one sided haunch connection has less plastic rotation (RN2). Also there is no plastic rotation on PZ with double sided haunches. The hysteretic behavior of each component of the connection agrees well with that observed from experimental test. Fig. 27 shows the total rotation of sub-assemblage versus total moment. As it has been shown experimental results have good agreement with analytical results. It means that proposed model of PZ which its rotation is part of total rotation has proper adaptability with experimental result and it has suitable accuracy to estimate PZ behavior.

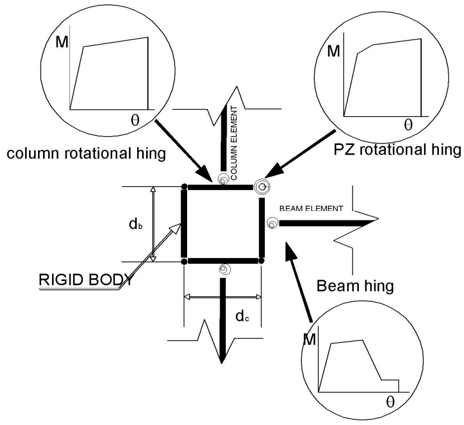


Fig. 23 PZ modeling used in this study

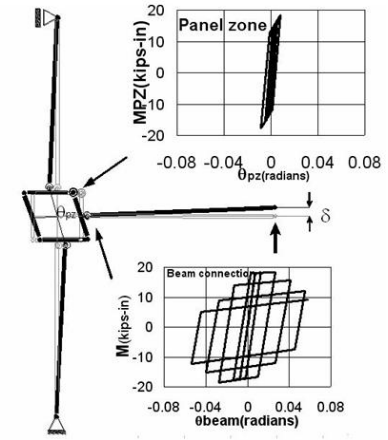


Fig. 24 Analytical model Specimen RN2

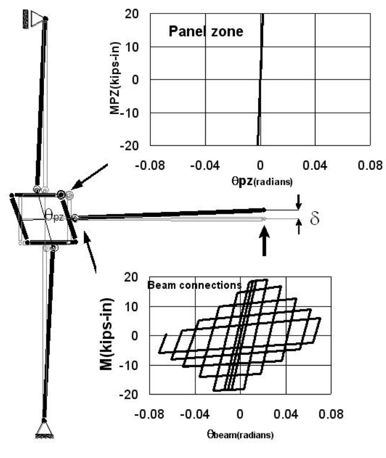


Fig. 25 Analytical model Specimen UCB-RN3

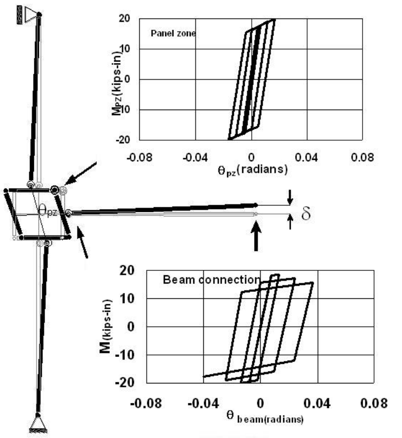


Fig26- Analytical model Specimen UCSD-6R

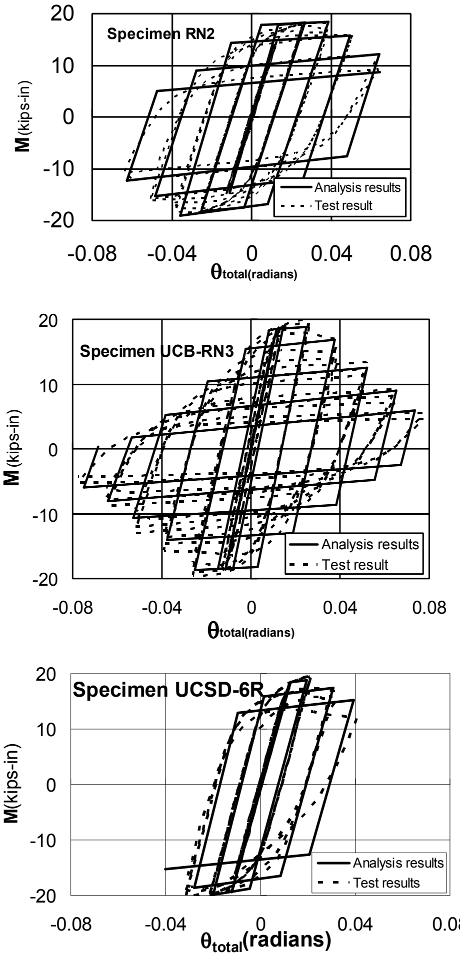


Fig. 27 Total rotation versus total moment of test and analysis result

VIII. Conclusions

This study investigated the cyclic behavior PZ according to connection type. In other words, this study emphasizes that the PZ nonlinear behavior, besides shear ratio (V_y/V_{pzMy}) and other parameters, is related to the type of connection. The following conclusions are made:

1-It is observed that plastic rotation of PZ was related to type of connection. PZ of specimen with T shape haunches has more plastic rotation rather than one sided haunches connection system.

2-as it is understood the PZ with double sided haunch has no plastic rotation. It may be due to existence of for continuity plate and strut action of haunches which don't permit the PZ to enter nonlinear phase. According Fig 17, as it is obvious the initial stiffness of this type of connection bigger than of all type of haunch systems.

3-According to the results of previous chapter, it is clear that ductility ratio and PZ plastic rotation, which equals $12\theta_y$ in FEMA 273, is not a constant value, and it seems to be dependent on type of connection, for all type of haunch system it does not reach to this amount, it seems that it overestimate

4-This research shows that all type of haunch connection system, beam could reach to their full plastic capacity, moreover some of them could provide extra capacity to use strain hardening. For example the minimum of M_{Max} for double sided haunch is 1.1 M_P and for T shape haunch is 1.05 M_P.

References

[1] M. D. Eenglehardt, A. S. Hussein "Cyclic loading performance of welded –bolted", *J. Struct Engrg, ASCE* 1993;119(12)

[2] W. M. Chi, A. R. Deierilein, "Fractures toughness demands in welded beam–column moment connections", *J Struct Engrg, ASCE* 2000;126(1).

[3] S. El-Tawil, "Panel zone yielding in steel moment connection", *J Struct Engrg, AISC* 2000;3qrt00;120(31).

[4] E.P. Popov, M. M. Blondet, L. Stepamov, and B. Stojadinvic, "Full-scale Steel Beam Column Connection Tests", SAC 96-01 Part 2. SAC-joint Venture, 1996.

[5] Engelhardt, Shuey, "Full-scale Steel Beam Column Connection Tests", SAC 96-01 Part 2. SAC-joint Venture, 1996

[6] S.C Goel, B. Stojadinvic.and K. H. Lee, "Truss Analogy for Steel Moment Connections", *Engineering Journal, AISC*, Vol.34,No. 2, ,pp. 42-53, 1997.

[7] K. Yu, S. Noel, and C. M. Uang, "Experimental studies on seismic rehabilitation of pre-northridge steel moment connections :RBS and Haunch approach", report NO. SSRP-1997/08 University of California Sandiego, 1997.

[8] Federal Emergency Management Agency (FEMA), *"Experimental investigations of beam-column sub assemblages*", Report SAC-96-01, Part 1, SAC Joint Venture, Sacramento (CA), 1996.

[9] Federal Emergency Management Agency (FEMA), *"Experimental investigations of beam-column sub assemblages*", Report SAC-96-02, Part 1, SAC Joint Venture, Sacramento (CA), 1996.

[10] Federal Emergency Management Agency (FEMA), *"Recommended seismic design criteria for new steel moment frame buildings*", FEMA-273, prepared by the SAC Joint Venture for FEMA, Washington, D.C., 2000.

[11] H. Gupta and H. Krawinkler, "Seismic for performance evaluation of steel moment resisting frame structures" John A. Blume earthquake Engrg.Ctr. Rep.No.132,Dept.of Civil and Evir. Engrg.,standford University, standford, Calif. 1999.

[12] Federal Emergency Management Agency (FEMA), "*Recommended seismic design criteria for new steel moment frame buildings"*, FEMA-355D, prepared by the SAC Joint Venture for FEMA, Washington, D.C. 2000.