CONSIDERING PZ SEISMIC BEHAVIOR IN WUF-B AND COVER PLATE CONNECTIONS

Peyman Shadmanheidari
East Tehran branch, Islamic Azad University, Tehran, Iran

R. Ahmady
Structural Research Centre, International Institute of Earthquake Engineering and Seismology (IIEES), Tehran, Iran

H. Kayhani & Pouya Shadmanheidari
Science and Research Branch, Islamic Azad University, Tehran, Iran

SUMMARY:

This study aims at modeling and evaluating the cyclic behavior of panel zone (PZ) in SMRF (Special Moment Resisting Frame) steel structures, considering the connection types, such as post Northridge Welded Unreinforced Flange-Bolted web (WUF-B) connections and coverplate connections. In this article some specimen of experimental works conducted by SAC are modeled using Finite Element (FE) to investigate the behavior of PZ. In FE models, all component interactions, such as slippage of bolts and welding properties are considered using a surface contact module. The results of numerical modeling show good agreement in comparison with test results. Backbone curves of moment vs. shear strain of PZ, which could be obtained from analytical models and experimental models, were the basis of Neural Network training. All information in considered variation of connection type is utilized to present a linear formula for linear and nonlinear phases and also determination of partial ductility factors. Research shows that PZ in cover plate connections have the most plastic shear strain whereas PZs in WUF-B connections have less plastic deformation capacity, therefore; cover plate can increase energy absorption of PZ; also it provides more partial ductility.

Keywords: panel zone (PZ), Flange-Bolted web (WUF-B), coverplate connection, modeling

1. INTRODUCTION

Special Moment Resisting frames (SMRFs) are widely used in steel structures as lateral force resisting system due to superior ductile behavior and energy absorption. SMRFs behave in ductile manner through flexural yielding of beam and shear yielding of the panel zone. During of sever ground motion, a huge amount of plastic deformation was expected at each member of SMRFs. In Northridge earthquake, sever damage in welded connection of steel moment frames occurred. But no member or building collapse as a result of connection failures was reported and no lives were lost.

Nevertheless, the occurrence of these connection fractures has resulted in changes to design and construction of steel moment frames. The fracture of moment connection in the Northridge earthquake exhibits a variety of origins and path. 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 resulted in CJP (Complete Joint Penetration) weld line fracture in beam to column connections. Cyclic test were conducted on 12 specimens constructed by SAC joint venture and also other some experimental works has been done by popov, whittaker , blondet, Engelhardt. 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 weld joining of the beam flanges to the face of relatively thick column flange is highly restrained. This restraint would cause somewhat more brittle behavior. In the context of earthquake damage to

SMRF buildings, the term repair is used to address the restoration of strength, stiffness, and inelastic deformation capacity of structural elements to their original level. For example the plastic strain of the test specimen specimen simple with WUF-B connections rarely reaches 0.02 radians .Yet Engelhardt and hussain (1993) observed that pre-Northridge connection had little plastic rotation capacity. Chi and Deierilein (2000) reported that larger panel zone caused a larger ductility demand. EL-TAWIL (2000) studied the effect of panel zone distortion on plastic rotation capacity. Also most of pre-Northridge connections behaved similarly. Also it was shown that capacity and stiffness of such connections respectively have a little amount. Enhancement of the strength, stiffness, or deformation capacity of either damaged or undamaged structural elements, would lead to improvements in their seismic resistance and that of structure as a whole. Modification typically involves substantial changes to the connection geometry that affected the method. In addition modification may also involve stiffening by cover plate or haunches and also removal of existing weld. Considering PZ seismic behavior of each category exhibit plastic rotation of each specimen depends on ratio of capacity of PZ to capacity of beam, type of connection, and number of continuity plates provided.

First of all, Popov et al.(1996) have studied WUF-B connections and triangular haunches at the bottom of beam flange. In haunch systems, studies show more beam plastic rotation and this behavior indicated more plastic capacity, on the contrary; decreases in plastic rotation of PZ in this kind of connection was observed, Also Engelhardt and shuey (1996) show similar result in their research.

Moreover in double haunch cases, this parameters show limited non-linear behavior drastically. In other word PZ of this type of connections behaves, completely linear, And PZ of this type of connection has more stiffness than one sided haunch and WUF. This result was observed from Engelhardt and shuey, Another strength specifications observed from double haunches connection, was degradation of capacity in the experimental result, this characteristic was known as a defect point in these categories, while huge amount of beam plastic rotation was a power point .Another category of this research was cover plate on top and bottom flange. The plastic behaviour and energy absorption of PZ and beam was stable and significant, also there were not significant degradations of the capacity according to shuey and Englhardt studies (1996) and Uang and Bondad study (1996).

2. MODELLING PROCEDURE

As it was mentioned models selected for this study and for verifying and considering its numerical modeling are taken from Popov et al(1985), Krawinkler (1978),Popov et al.(1996), Engelhardt (1996) and Uang and Bondad (1996)and SAC joint test experiments (1996a). The objective of these was to investigate and to improve post Northridge connection performance such as modification of weld procedure, and geometries of connections. The test set up studies of these researches include one beam and a column like an exterior joint, in which beam web connects to column through shear tab and bolt. The beam end is simply supported and applied load on the center of end beam generate moment on connection. Also the end of column was simply supported in two ends which indicate inflection point. All of mechanical properties and geometries of section has been shown in table 3.1. Yield strength (F_y) and ultimate strength (F_u) has shown in this table which was obtained from coupon test average.

3. FINITE ELEMENT MODELLING

ANSYS (1998) multi-purpose finite element modeling code is used to perform the numerical modeling of connections. FE models are created using the ANSYS parametric design language. The geometrical and mechanical properties of the connection model were treated as parameters, for example root opening geometries and Yield strength (F_y) and ultimate strength (F_U). Numerical modeling of connection is done including following considerations. Using eight-node-first order SOLID45 elements, and bolt shanks are modeled using SOLID64 element. Bolt holes vary for each model group, for example in SAC phase bolt diameter was 1 inch.

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 are modeled using mentioned contact element. Bolt heads and nuts are modeled as hexagonal and similar to real shape to simulate the frictional forces. Coulomb coefficient is assumed as 0.3 which produce the best result. Nearly in most of literature for class A type steel surfaces, the coulomb coefficient is one third of the usual value of 0.33 which is proposed. It means 0.1 is proper amount for converging and yielding result.

Table 3.1. Geometrical properties of specimen

Test specimen number	specimen name	Beam section	Beam Web F_y	Beam Flange F_y	column section	column Web F_y	column Flange F_y	connection type
1	RFSPN1(Whittaker etal)	W30x99	55.7	50.3	W14x176	69.5	69	WUF-B
2	RFSPN2(Whittaker etal)	W30x00	57.4	48.6	W14x176	70.8	68.9	WUF-B
3	RFSPN3(Whittaker etal)	W30x99	53.4	47.2	W14x176	72.5	68.4	WUF-B
4	RFSRN1(Whittaker etal)	W30x99	55.7	50.3	W14x176	69.5	69	WUF-B
5	UCSDPN1(Uang etal)	W30x99	57.1	46.6	W14x176	67.2	68.2	WUF-B
6	UCSDPN2(Uang etal)	W30x99	57.1	46.6	W14x176	67.2	68.2	WUF-B
7	UCSDPN3(Uang etal)	W30x99	57.1	46.6	W14x176	67.2	68.2	WUF-B
8	UCSDR2(Uang etal)	W30x99	57.1	46.6	W14x176	67.2	68.2	WUF-B
9	SAC-N06(SAC joint)	W24x76	50.2	44.2	W14x132	66.1	66.4	WUF-B
10	SAC-N07(SAC joint)	W24x76	50.2	44.2	W14x132	66.1	66.4	WUF-B
11	RFSAN1(Whittaker etal)	W30x99	55.7	50.3	W14x176	69.5	69	Coverplate
12	UCBAN1(Popove etal)	W36x150	40.3	40.3	W14x257	67.8	67.8	Coverplate
13	UTA-4(Engelhardt)	W36x150	45.5	39.5	W14x257	69	69	Coverplate
14	SAC NO09(SAC joint)	W24x76	39.1	38.3	W14x132	66.1	66.4	Coverplate
15	SAC NO12(SAC joint)	W24x76	50.2	44.2	W14x132	66.1	66.4	Coverplate
16	SAC NO13(SAC joint)	W24x76	50.2	44.2	W14x132	66.1	66.4	Coverplate

Unit: Ksi

4. BOUNDARY CONDITION AND APPLIED LOAD

To satisfy boundary condition of analytical model, the end of beam is restrained for outward motion. Also because of existing lateral bracing system in the experiment on the flange of beam of specimen some points in the flange model due to distance from column face are restrained. Since there was no information about the situation of bolt regarding pre-tension or ordinary twisting of bolt, it is considered as ordinary condition which would not permit shear tab to slip outward the plane of web. The loading procedure was in displacement control manner, and the loading pattering was accordance with SAC test protocol (1996) like the experiment.

5. MATERIAL PROPERTIES

The material properties of these models had kinematic behavior with strain hardening in nonlinear phase to predict the reality of material precisely. The stress-strain relation for all connection components except for the bolts is represented using a three-linear constitutive model. An isotropic hardening rule with a Von-Mises yielding criterion is applied to simulate plastic deformations of the connections. ASTM A36 steel grade was used for the beam and ASTM A52 steel grade was utilized for the column and connection details. In the current study, the mechanical properties of beam column and connections are taken from Table 3.1. To get more information see ref SAC (1996a &b). The yield stress and ultimate strength of bolts mechanical properties are assumed to be based on nominal properties of A325. The yield stress and ultimate stress of weld are assumed to be based on nominal properties of E71T-8(AWS A5.20) (SAC 1996b). Modulus of elasticity and Poisson's ratio are considered respectively to be 29000 kips/in^2 and 0.3.

6. VERIFICATION OF FINITE ELEMENT MODEL

To evaluate the accuracy of finite element modeling (ANSYS 1998) approach, 16 finite element models are created according to actual test which was mentioned in Table 3.1 and results are compared with test result. Fig 6 shows one of the FE models of WUF-B specimen and Fig 7 shows Von-Mises stress for this model. Respectively Figs 12 show FE model of each category and Von-Mises stress contour. For example, analytical and experimental hysteric responses of beam plastic rotation versus applied moment are shown in Figs 1 to 4. From these figures, it can be seen the results obtained from finite element models have good agreement with test data.

The differences between the test data and the finite element modeling are obvious. 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 can be mentioned that the material properties, which are used in FE, are from average, but in reality steel is not a homogenous material and amount of every coupon test result could affect the actual result. The differences between the test data and the numerical models grow in nonlinear portion of curve. Therefore all analytical models have good agreement with test models. And the results could be reliable for evaluating of PZ.

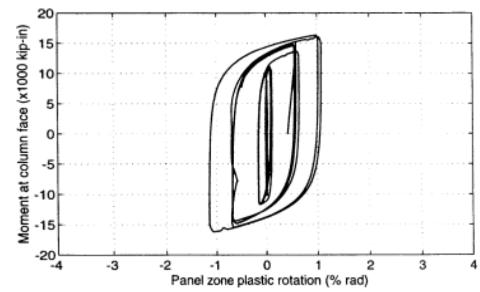


Figure 1. hysteretic behavior of test (UCSDPN3)(SAC 1996a)

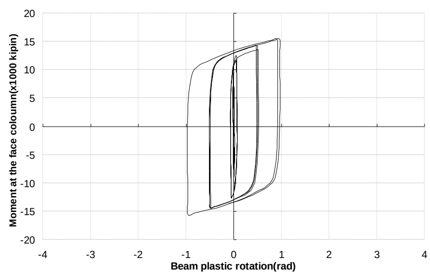


Figure 2. Hysteretic behavior of numerical model (UCSDPN3)

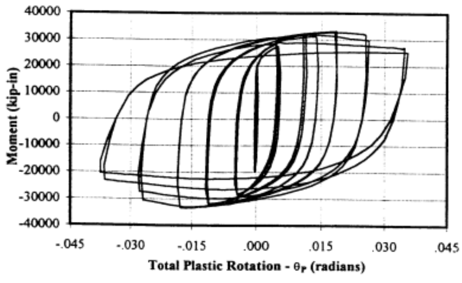


Figure 3. hystertic behavior of test (UTA-4) (SAC 1996a)

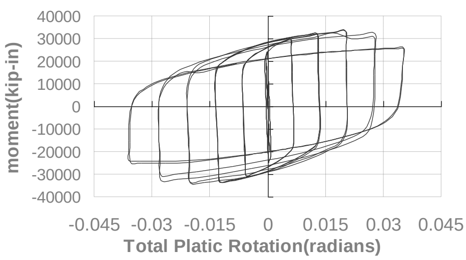


Figure 4.hysteretic behavior of numerical model (UTA-4)

Also the back bone curve of analytical moment and experimental Model of moment versus PZ shear strain for some studied experimental and analytical models have been shown in Figs 5, which are in a good agreement. These curves are used for training neural network program, and new curve as Neural Network (N.N) output are reread from N.N. The N.N was consisted of three layered PERCEPTRON net .all of three group data was the basis of behavioral model in each group (Ghaboussi and WU 1998). It is mentioned that 11 models were used to train neural network and then 5 models were used to test the NN results. Finally, all of curves are reread from N.N as shown in the Fig 5, By considering the curves it is observed 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 (M_p) and type of connection.

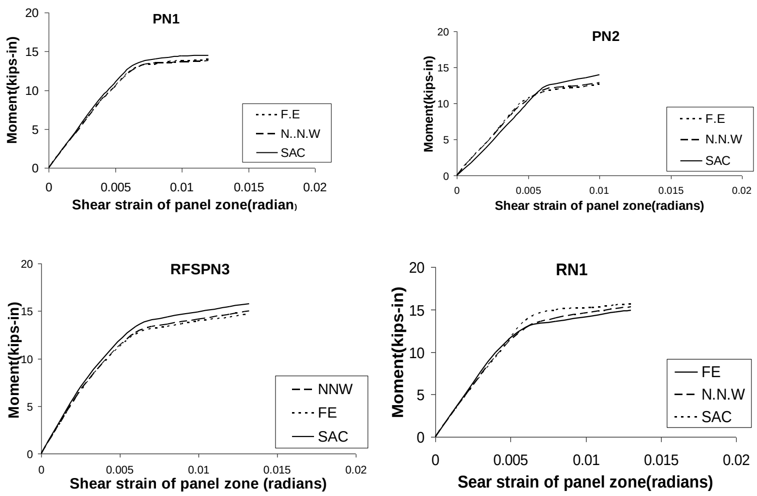
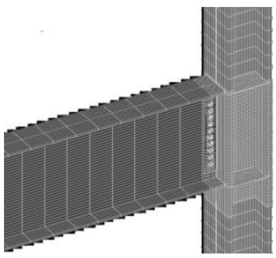


Figure 5. Comparison between result of FE and N.N.W and experimental result for (WUF-B) connection

7. EFFECT OF WUF-B CONNECTION ON PZ SEISMIC BEHAVIOUR

To evaluate the of WUF-B, hysteretic behavior of PZ which was obtained from previous section, must be converted to idealized curve based on FEMA 356 (2000),and from this idealized curve that lie in two lines, the yield moment and plastic rotation capacity of PZ could be determined. These two lines are tangents of end and start of back bone curve behavior of PZ, also yield moment is the intersection of these two lines. Figs 6 and 7 show the FE model and Von Mises stress distribution of the test specimen RFSPN1.



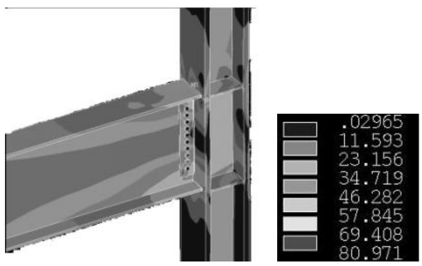


Figure 6. FE modeling of connection (RFSPN1) **Figure 7**. Von Misses distribution of connection (RFSPN1)

To obtain, the necessary data such as initial stiffness and V_y/V_{PZMy} ratio and M_m/M_p (M_m is the maximum moment at the column face and M_p is the plastic moment capacity of the connected beam) are presented in the fig. 14 .this amount are normalized by K_G , which is PZ initial stiffness is obtained by:

$$K_G = 0.95\, d_C.t_w.G \qquad (1)$$

G is shear of steel modulus and t_w is thickness of column web and dc is column depth. Also VpzMy is presented as follows:

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

In this equation $V_{pz}M_y$ is shear which is transfer from beam to PZ, L is beam length, h is column height and db is beam depth, also M_y and M_p are ultimate elastic and plastic moment of beam. Fig.8 in this paper shows moment ratio (Mm/MP) with respect to a panel zone strength ratio $V_y/V_{PZ}M_y$. The moment ratio is an indicator of the connected beam reaches its moment capacity before the connection fails; i.e. (M_m/M_p) less than 1 indicates the plastic moment capacity is not reached. According to fig. 8, (M_m/M_p) of this kind of connection exceeds 0.9 as $V_y/V_{PZ}M_y$ is greater than 1. This result according to Moon (2007) is respectively 1.0 and 0.9. The difference between two results may be originated from using deeper beam in this study which has lower plastic rotation capacity. In other words different variety of beam sections has been used in this study.

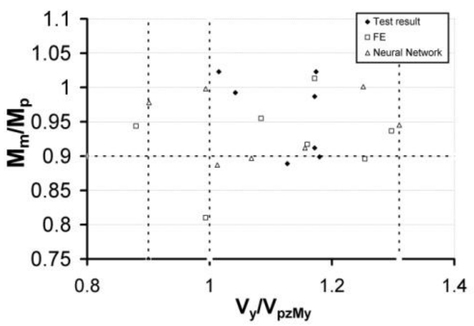


Figure 8. Moment ratio (M_m/M_P) versus V_y/V_{PZMy} for (WUF)

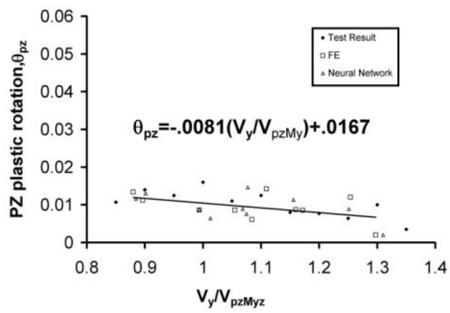


Figure 9. PZ plastic rotation versus V_y/V_{PZMy} for (WUF) connection connection

the comparison item in HANs work was (Mf/MP) which is Mf is moment at fracture point, but in Mm is maximum moment, but in this ensemble, because there is no sensible degradation in specimen,Mf is nearly equal with Mm .therefore the result of this study and HANs are comparable. According to this research, panel zone strength ratio, i.e. Vy/VPZMy, should be greater than 1.0, if ultimate moment of PZ is expected to reaches to 0.9 amount of beam plastic moment. In order to obtain more reliable information, some extra data were generated by the N.N program and applied for modeling of PZ properties. Considering FE, N.N and experiments data were used for analytical modeling of PZ and beam, Fig 9 shows PZ plastic rotation of WUF-B connection versus Vy/VPZMy, regarding these Figures, PZ plastic rotation does not reach 0.02 radians. Also the elastic limit of PZ shear strain versus strength ratio Vy/VPZMy has shown in the Fig.10.

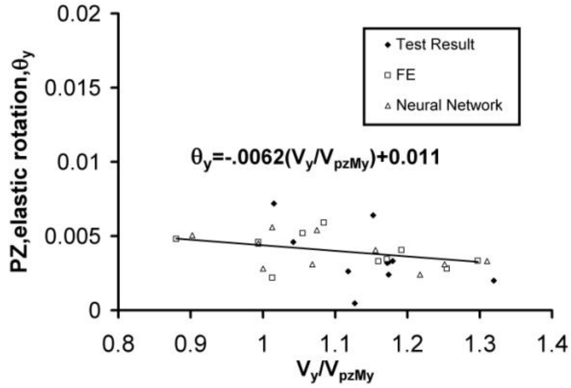
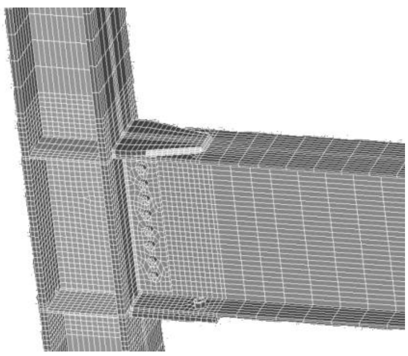


Figure 10. PZ plastic rotation versus V_y/V_{PZMy} for (WUF) connection

8. EFFECT OF COVER PLATE CONNECTION ON PZ SEISMIC BEHAVIOR

According to SAC report, this type of connection has significant effects on total and PZ plastic rotation the PZ plastic rotation reaches 0.03 radians for this type of connections. The FE model of RFSAN1 has shown in Fig 11.Fig.12 shows moment ratio (Mm/MP) with respect to a panel zone strength ratio Vy/VPZMy, The moment ratio is called as strength ratio, and it is an important indicator if the connected beam reaches its moment capacity before the connection reaches to maximum capacity or it fails. Because of completion of data in this section, some extra FE models were built to cover the gap between data and then all of statistical tasks have been done upon additional information. After training NN some data which was in accordance to test data average, have been reread and new output were extracted. Fig 13 and 14 show elastic and plastic PZ rotation versus Vy/VPZMy for the collected data from FE modeling ,N.N and the experiment . PZ plastic rotation could reach 0.03 radians and the trend of regression line is reducing, moreover value of elastic rotation is more than PZ of the specimen with WUF-B connection. Also paying attention to Fig.12 for all value of strength ratio Vy/VPZM data will be scattered but the average of (Mm/Mp) is 1 and minimum range of data reach 0.9 according to test results.



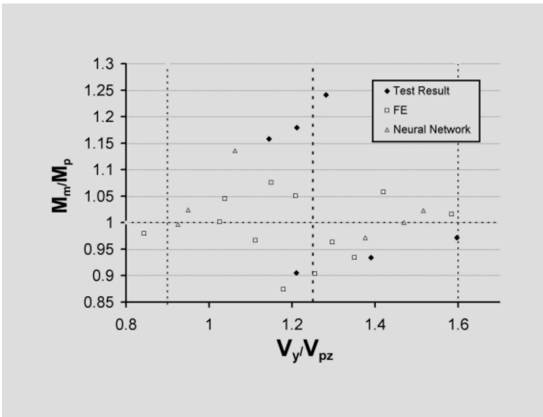
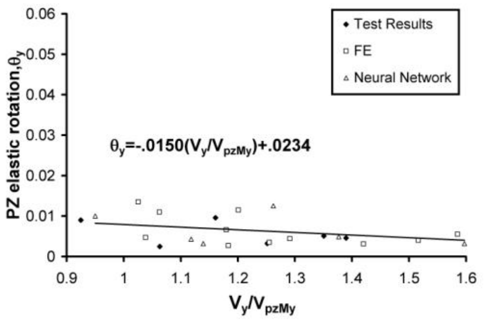


Figure 11. FE model of Specimen (RFSAN1) **Figure 12**. Moment ratio (M_m/M_p) versus V_y/V_{PZMy}



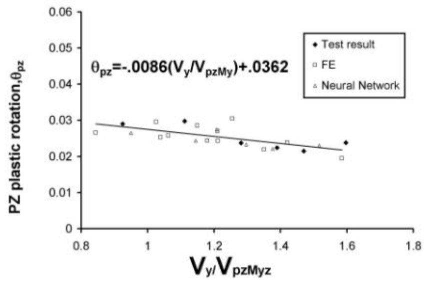
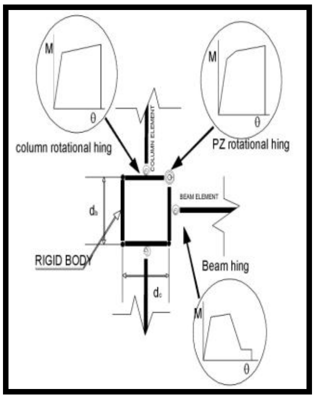


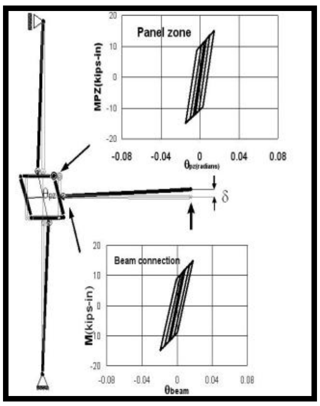
Figure 13. PZ elastic rotation versus V_y/V_{PZMy} **Figure 14**. PZ plastic rotation versus V_y/V_{PZMy}

9. ANALYTICAL MODELS PROPOSED FOR EACH CATEGORY

In order to simulate the hysteretic of the post Northridge behavior for each category, the regression line for two mentioned groups are used to create PZ model. This study attempts to predict the PZ behavior pertain to connection type therefore the main aim of this research is presenting a model for PZ according to connection, thus the proposed connection can account for inelastic panel zone as well as connection fractures.

Boundary elements for PZ modeling are rigid elements with very high axial rigidity. The panel zone shear strain and stiffness can be modeled by two bilinear springs in one of the four corners as shown in the fig.15.the elastic and plastic specifications of these two spring obtained by mentioned regression lines in last parts for each connection type . The two superimposed springs in the PZ are used to simulate trilinear behavior of PZ. Detail of the two bilinear can be found in Gupta and Krawinkler (Krawinkler 1978).





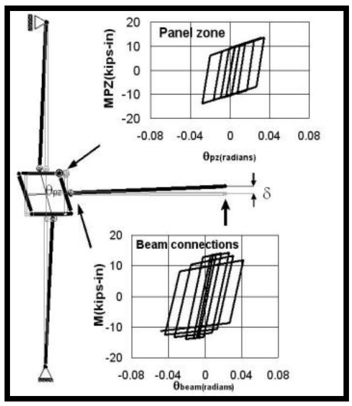


Figure 15. PZ modeling detailing

Figure 16. Results for the analytical model (UCB-PN3)

Figure 17. Results for the analytical model (UCB- AN1)

Because of focusing on the PZ seismic behavior in this study under different types of connection (wuf-b, cover plate, modeling of beam was ignored, and the backbone curve of beam and column which were obtained from test results are considered. The strain hardening for PZ reach to 6% which is proposed by FEMA 355D (2000) and FEMA273 (2000).

10. VERIFICATION OF PROPOSED PZ CONNECTION MODEL

The connection model proposed in this study verified by comparing hysteretic curves obtained from analyses with those from test of a specimen for each categories .i.e. connection type. Drain -2DX software (Parakash and Powell (1998)) was used for conducting analyses. Fig.18 shows hysteretic curve for beam plastic rotation and panel zone rotation versus moment obtained from analytical model and the experiments. As observed from Fig. 18, the test specimen with cover plate connection (UCB-AN1) has more plastic rotation, and it is significant while PZ with WUF-B connection has less plastic rotation, The hysteretic behavior of each component of the connection agrees well with that observed from experimental test.

Fig18 shows the total rotation of subassemblagement 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.

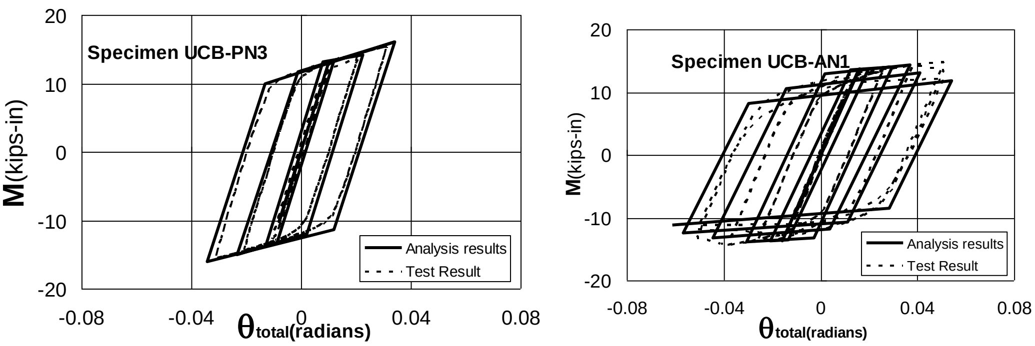


Figure 18. Total rotation versus total moment of test and analysis result for test specimen UCB-PN3 (test specimen with WUF-B) and UCB-AN1 (test specimen with cover plate connection)

11. CONCLOUSION AND ANALYSIS IMPLICATION

This study investigated the cyclic behavior PZ according to WUF-B and Coverplate connections. In other words, this study emphasizes that the PZ nonlinear behavior is related to the type of connection including shear ratio (Vy/VpzMy) and other parameters. The following conclusions are made:

1. It is observed that plastic rotation of PZ was related to type of connection. PZ specimen with cover plate has more plastic rotation rather than WUF-B and haunch connection system. Also in WUF-B connection system, PZ has more plastic rotation capacity rather than PZ of haunch system.
2. In cover plate connection system, data were scattered but it can provide at least 0.9 M_p at fracture point for all values of shear ratio V_y/V_{PZMy} But in one sided haunch it can provide full plastic moment for any value of shear ratio.
3. According to the results of previous chapter, it is clear that ductility ratio and pz plastic rotation, which in FEMA 273 is equal 12 θ_y , is not a constant value ,and it seems to be dependent on type of connection, for example in the case of cover plate

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