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ANALYTICAL MODELING OF PANEL ZONE SEISMIC BEHAVIOR WITH DIFFERENT CONTINUITY PLATE FORMATION BASED ON SAC96 EXPERIMENTS

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ABSTRACT

The 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 plate thicknesses and geometric properties of PZ on its seismic behavior. But there is not much investigation on the effects of number of provided continuity plates in case of presence of one haunch or two haunches (3 to 4 continuity plates) in comparison with no haunch case (2 continuity plates) for exterior columns. In this research first detailed finite element models of 14 tested connection of SAC96 were created and analyzed then obtained cyclic behavior backbone curves of these models besides other SAC96 results for similar tests, were used for neural network training. Then seismic behavior (back bone curve of hysteretic behavior) of these data will be categorized according to continuity plate's arrangements. As a result, for each case, an analytical model for estimation of PZ seismic behavior will be proposed.

Introduction

Moment frames (MFs) are widely used in steel structures as a lateral force resisting system due to their superior energy absorption and ductile behavior. SMFs (special resisting moment frame) behave in a ductile manner through flexural yielding of beam and shear yielding of the panel zone. During severe ground motions, a huge amount of plastic deformation is expected at each member in MFs. In Northridge earthquake, severe damage in welded connection of steel moment frames occurred but there were no member or building collapses because of the connection failure. This research will focus on PZ (Panel zone) nonlinear behavior with regard to

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type of connection. Coverplate connection, connection with one sided haunch and double sided haunches connection will be specially considered.

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 paths. In general, fracture was found to initiate at the root of beam flange CJP weld (groove weld) and propagated 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 12 specimens constructed by SAC joint venture and other experimental works have been done by Popov, Whittaker, Blonodet, Engelhardt. Fragile behavior of WUF-B (Welded Unreinforced Flange Bolted) observed in SAC experiments, resulted in change of formation and connection types, therefore some kind of modification were made to connections, such as coverplate, one-sided and double-sided haunch to improve the performance of connections.

Welding of the beam flanges to the face of relatively thick column flange could make a highly restrained in the area of HAZ (heat affected zone). This restraint would cause more brittle behavior. In the context of earthquake damage to WSMF (welded special moment frame) 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 rotation of simple WUFs specimen rarely reaches 0.02 radians. Yet Engelhardt and hussain [1] observed that pre-Northridge connection had little plastic rotation capacity. Chi et.al [2] reported that larger panel zone caused a larger ductility demand. EL-TAWIL [3] studied the effect of panel zone distortion on plastic rotation capacity. Most Pre-Northridge connections behaved similarly and it was shown that those connections had little plastic capacity in beam and PZ (panel zone). 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. In addition, modification may also involve stiffening by coverplate, haunches or even removal of existing weld. To study of the effect of PZ, some experiment of SAC joint venture and other researchers with variety of connection type in three groups (coverplate, one sided haunch and double sided haunches) are considered which are presented in table 1.

Table 1: properties of selected specimens for verification of analytical model

specimen number	specimen name	Beam			column			connection type
		section	Web Fy	Flange Fy	section	Web Fy	Flange Fy	
1	RFSAN1(Whittaker etal)	W30x99	55.7	50.3	W14x176	69.5	69	Coverplate
2	UCBAN1(Popov etal)	W36x150	40.3	40.3	W14x257	67.8	67.8	Coverplate
3	UTA-4(Engelhardt)	W36x150	45.5	39.5	W14x257	69	69	Coverplate
4	SAC NO09(SAC joint)	W24x76	39.1	38.3	W14x132	66.1	66.4	Coverplate
5	SAC NO12(SAC joint)	W24x76	50.2	44.2	W14x132	66.1	66.4	Coverplate
6	SAC NO13(SAC joint)	W24x76	50.2	44.2	W14x132	66.1	66.4	Coverplate
7	UCSDR1(Uang etal)	W30x99	57.1	46.6	W14x176	67.2	68.2	one sided haunch
8	UCSDR3(Uang etal)	W30x99	57.1	46.6	W14x176	67.2	68.2	one sided haunch
9	UCBR2(Popve etal)	W36x150	60.6	60.6	W14x257	67.8	67.8	one sided haunch

10	UCB RN2(Popov etal)	W36x150	60.6	60.6	W14x257	67.8	67.8	one sided haunch
11	UTAR1(Engelhardt)	W36x150	45.5	39.5	W14x257	69	69	one sided haunch
12	UTAR1B(Engelhardt etal)	W36x150	45.5	39.5	W14x257	69	69	one sided haunch
12	UCBRN2(Popov etal)	W36x151	60.6	60.6	W14x258	67.8	67.8	double sided haunches
13	UCBRN3(Popov etal)	W36x152	60.6	60.6	W14x258	67.8	67.8	double sided haunches
14	UTA-3(Engelhardt etal)	W35x150	45.5	39.5	W14x257	69	69	double sided haunches

Units are in Kips and Inch.

The first category of this research was coverplate on top and bottom flange. The plastic behavior and energy absorption of PZ and beam was stable and significant, also there were not significant degradations of the capacity according to Shuy and Englhardt studies [5] and Uang study [6]. However, in double-sided haunch cases, plastic rotation show very limited non-linear behavior or even complete linear behavior may be observed. PZ of this type of connection has more stiffness than one sided-haunch. Another phenomenon which was observed from double-haunches connection, was degradation of capacity in the experimental results, this characteristic is known as a defect in this category while huge amount of beam plastic rotation is considered a power point.

Finite element modeling and material properties

Models used 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 welding procedures as well as connections geometries. The test set up of these researches consisted of a column and one beam, resembling an exterior joint in which beam web was connected to column through shear tab using bolts. The end of the beam was simply supported and the load applied on the center of beam end imposed moment on the connection. In addition, the end of column was simply supported at its two ends.

ANSYS [11] 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 were treated as parameters, for example Yield strength (F_y) and ultimate strength (F_U) were among the studied parameters. Numerical modeling of specimen including beam, coloumn and connection details such as coverplate and haunches and weld line was done 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 for bolt and hole. The interaction in adjacent surfaces 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.

To satisfy boundary condition of real model, the end of beam model was restrained from outward motion. Also because of existing of lateral bracing system on the flange of the beam of test specimen, some points on the flange analytical model due to distance from column face were restrained. The displacement-control loading procedure was employed in accordance with SAC test protocol [9] as it was considered in the actual test which was originated from ATC24.

Monotonic loading was applied to produce moment. The material properties of these models had kinematic behavior with strain hardening in nonlinear phase. An isotropic hardening rule with a Von-Mises yielding criterion was applied to simulate plastic deformations of the connections. To get more information on material properties see ref [9] and [10].

Verification of finite element model

To evaluate the accuracy of finite element modeling approach, 14 finite element models were created according to the actual test result which was mentioned in Table 1 and the results were compared with test result respectively. For example, analytical and experimental hysteretic behaviors 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. Verification of the models has been done for 14 tests for the overall sub-assemblagement behavior. Also verification of some PZ back bone curves of analytical models and the test results are shown in fig 5 which indicated good agreement in the obtained results.

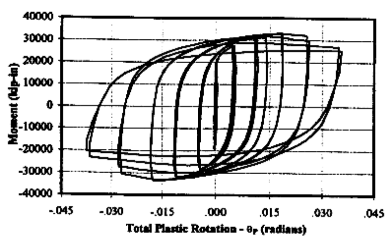


Figure 1. Hysteretic behavior of test (UTA-4) [10]

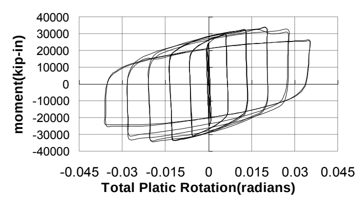


Figure 2. Hysteretic behavior of numerical model (UTA-4)

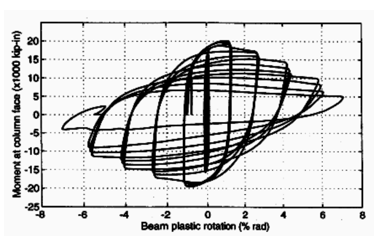


Figure 3. Hysteretic behavior of test (UCB-RN3) [9]

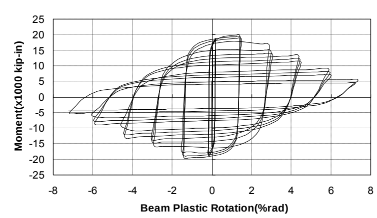
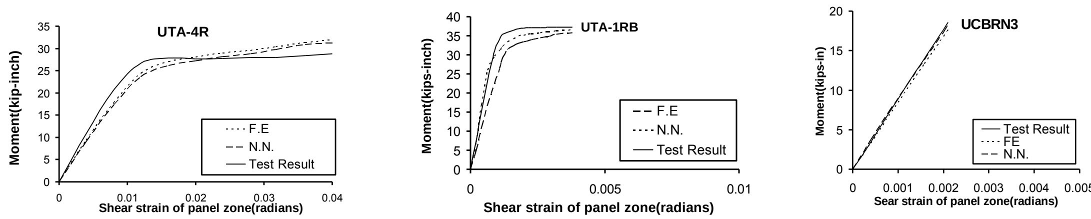


Figure 4. hysteretic behavior of numerical model (UCB-RN3)

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. 1, 2 or 3, 4 it is evident that for specimen UCB-RN3 or UTA-4, 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 support 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 moment versus PZ shear strain for analytical and experimental model are shown in Fig 5. These curves were used for training the neural network program, and new curve as Neural Network (N.N) output were reread from NN. NN consisted of three-layered PERCEPTRON net. All of the three data groups were the basis of behavioral model [12]. 14 models were used to train neural network and then 2 models were used to test the NN results. Finally, all of the curves were reread from N.N which can be seen in figs.5. By considering the curves, one can see that the N.N curves have good agreement with FE and test Results. Training data for this NN consisted of shear ratio V_y/V_{PZMy}, full plastic moment (Mp) and type of connection. From these results in every category, the plastic and elastic rotation of PZ were extracted and a linear regression analysis was performed which approximately had proper agreement and the correlation changes from 65 % to 90%.



(a) (b) (c)

Figure 5. Comparision between result of FE and N.N. and experimental result for cover plate connection a) PZ in cover plate connection b) PZ in one sided haunch connection c) PZ in double sided haunch connection

Effect of coverplate connection on PZ seismic behavior

This type of connection has more desirable behavior with respect to other connection types studied. According to SAC report, this type of connection has suitable plastic rotation, which could reach 0.03 radians. In addition, PZ plastic rotation of this group could reach 0.03 radians. The FE model of RFSAN6 is shown in Figure 6. Figure.7 shows moment ratio (M_m/M_P) with respect to a panel zone strength ratio V_y/V_{PZMy} . The moment ratio is an indicator if the connected beam reaches its moment capacity before the connection reaches to its maximum capacity or fails. Because of completion of data in this section other FE models (six models) were built to cover the gap between data and then all statistical tasks have been carried out using additional information. All of data is presented in figs. 7,8,9.

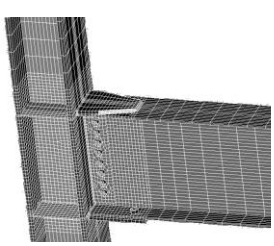


Figure 6. FE model of Specimen (RFSAN1)

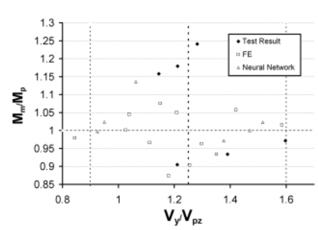
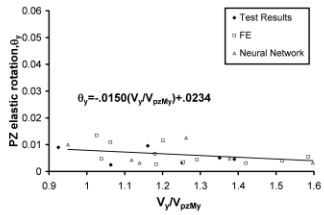


Figure 7. Moment ratio (M_m/M_p) versus V_y/V_{PZMy} for cover plate connection

PZ plastic rotation could reach 0.03 radians and the trend of regression line has negative slope as it is expected. PZ elastic rotation versus V_y/V_{PZM} is shown in fig 8. Fig. 9 shows PZ plastic rotation versus V_y/V_{PZM}. Moreover, the value of elastic rotation is more than the case of PZ with haunch connection. Regarding fig.7, for any value of strength ratio the data will be scattered but the average of moment ratio equals 1 and minimum will reach 0.9.



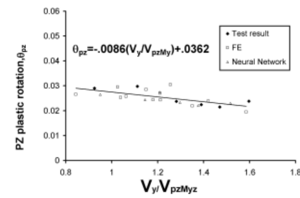


Figure 8. PZ elastic rotation versus V_y/V_{PZMy} Figure 9. PZ plastic rotation versus V_y/V_{PZMy}

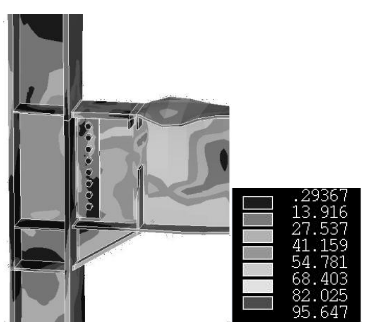
In conclusion presence of cover plate could enhance nonlinear properties of PZ. V_{PZMy} is the shear, which is transferred from column:

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

Where In this equation 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 cover plate length.

Effect of haunch connection on PZ seismic behavior

Welding a tapered haunch to the bottom of beam has been used for many years in steel structures which can be very effective for enhancing the cyclic performance of damaged moment connections or for new construction (Noel and Hang 1996)[13]. Reinforcing the beam with a welded haunch can be viewed as a means of increasing the section modulus of the beam at the face of the column. Most of the references [14] pointed out that a more appropriate approach is to treat the flange of the weld haunch as a diagonal strut. The strut action significantly changes the force transfer mechanism of this type of connection. In this study, some of experimental works including one sided-haunch connections have been considered. In most of the specimen 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 to prevent extension of the plastic area to the connection face at column. Von-Mises stress distribution of this model is shown in the fig. 10.



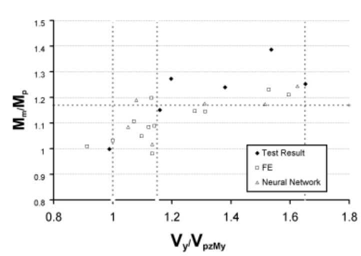


Figure 10.Von Misses distribution (ucsd-3R) Figure11. (M_m/M_P) versus V_y/V_{PZMy} for one sided haunch connection

Like the previous section, 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, to cover the data gap which is presented in figs 11 to13.

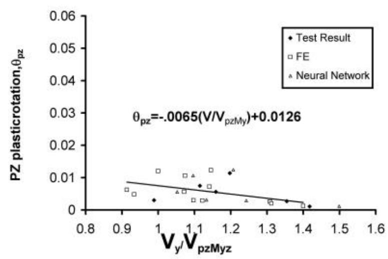


Figure 12.PZ plastic rotation versus V_y/V_{PZMy} for one sided haunch

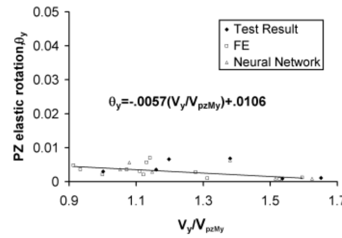


Figure13. PZ elastic rotation versus V_y/V_{PZMy} for one sided haunch

The strength ratio (M_m/M_P) for this group is shown in the fig.11. 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, reach to full plastic moment before fracture. The average of strength ratio (M_m/M_P) is 1.15. It means that this type of connection establishes a position for the beams to enter strain hardening before occurrence of collapse, according to the following formula:

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

Where Z is plastic modulus of beam, F_{tb} is expected tensile stress of steel, F_{yb} is yield strength of steel. and $M_{\max}$ is maximum moment before fracture. Further studies [15,16] show that this over strength could reach 40 or 50%. PZ plastic rotation versus V_y/V_{PZMy} for one sided-haunch connection is shown in fig 12. Plastic rotation does not reach 0.02 radians, even it is less than 0.015 and also PZ elastic rotation versus V_y/V_{PZMy} is shown in fig 13. In conclusions, PZ in this type of connection does not significantly enter the plastic phase. The most important causes of this phenomenon are three continuity plates and the strut action of the haunch flange that have restrained the boundary of PZ. Another type of haunch is double-sided haunch. Von-Mises stress distribution is shown in Fig.14. The cyclic performance of beams, which was reinforced by this type of haunch, as can be seen in fig 5-c, is essentially linear.

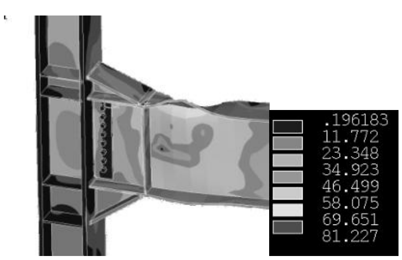


Figure 14. Von Misses distribution (UCB RN3)

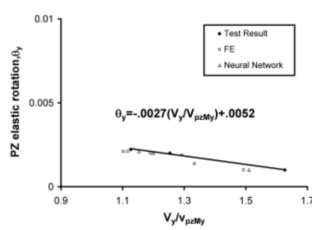


Figure 15. PZ elastic rotation versus V_y/V_{PZMy} for double sided haunch

To get reliable results, some models with various strengths according to 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.

The average value of strength ratio (M_m/M_P) of PZ in this field is about 1.17 and minimum value is 1.10. It means that beam moment in this type of connection could reach up to a 10% more than full plastic capacity. Considering fig.15, it can be realized that the 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 the presence of four continuity plates and strut action of two flange haunches which would considerably stiffen the PZ and leads to no energy absorption situation.

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 studied connection groups were used to create a PZ model. This study attempts to predict the PZ behavior with respect to connection type. 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.

Details of the previous bilinear PZ model can be found in Gupta and Krawinkler [17]. Because of focusing on the PZ seismic behavior in this study for different types of connections (coverplate, one-sided haunch, and two-sided haunch) and the number of continuity plate implemented in PZ, modeling of beam was ignored, and the backbone curve of beam and column, obtained from test results, was considered. The strain hardening for PZ could reach 6% according to FEMA355D [18] (M_m/M_P) which will be implemented for the behavioral model of PZ, the plastic and elastic specification of PZ model in this research were derived from regression lines which were extracted in previous sections depending on type of connection and V_y/V_{PZMy} ratio and the average amount of (M_m/M_P) ratio for every categories will be used.

The connection model proposed in this study was verified by comparing hysteretic curves obtained from the analyses with those obtained from test of a specimen for each group. Drain-2DX program [19] was used for conducting the analyses. As observed in Fig.17, the panel zone with coverplate connection (UCB-AN1) has significant plastic rotation while PZ with haunch system connection has less plastic rotation, and also PZ of specimen with one-sided haunch has minimum plastic rotation, in this ensemble (fig 18). However, PZ in double-sided haunch specimen behaves in elastic range (fig19). The hysteretic behavior of each component of the connection agrees well with that observed from experimental test.

Fig. 20 shows the total rotation of subassemblagement versus total moment. As it was shown experimental results had good agreement with analytical results. It means that proposed model of PZ has good agreement with experimental result and it has suitable accuracy to estimate PZ behavior.

FEMA 273[20] suggests a constant partial ductility factor (μ) for PZ. In addition, despite connection type, value of four times of elastic distortion of PZ ($4\theta_{yPZ}$) as a stable plastic shear strain domain is proposed by Krawinkler[8]. However, according to this study, it depends on the type of connection and geometry of beam and column for example these assumptions are justifiable for PZ with cover plate connection, but it is not acceptable for haunch connection consisting of one-sided or double-sided haunch.

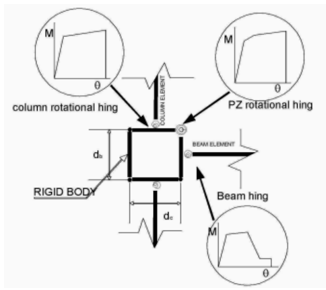


Figure 16. PZ modeling used in this study

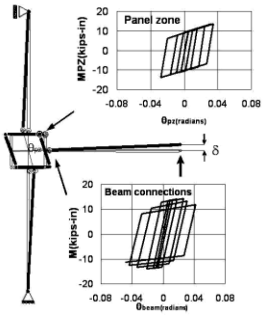


Figure 17-specimen UCB-AN1

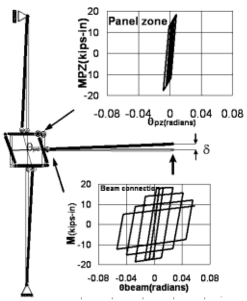


Figure 18. Specimen UCSD-R3

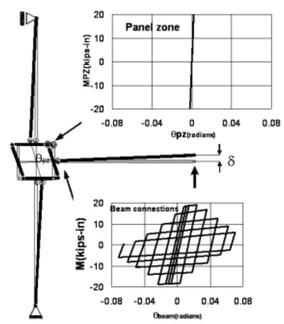


Figure 19. Specimen UCB-RN3

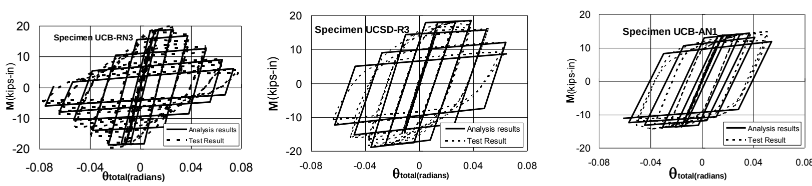


Figure 20. Total rotation versus total moment of test and analysis result.

Conclusion

This study investigated the cyclic behavior of PZ according to the 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 can be concluded that plastic rotation of PZ is related to type of connection. PZ specimen with coverplate has more plastic rotation in comparison with haunch connection system.
2. In the haunch system, PZ of one-sided haunch has a little plastic rotation capacity. In the two-sided haunch, there is no plastic rotation of PZ and it behaves linearly. In addition, the slope of regression line in this type of connection is less in comparison with others. It shows that dependency on shear ratio V_y/V_{PZMy} in this type of connection is less than

others, and the presence of three or four continuity plates respectively for one sided haunch and two sided haunch, in front of haunch flange and beam flange, can reduce plastic behavior of PZ.

3. In coverplate connection system, data was scattered but it can provide at least 0.9 M_p at fracture point for all values of shear ratio V_y/V_{PZMy}. However, in one-sided haunch it can provide full plastic moment for any value of shear ratio. Besides, double-sided haunches could reach 1.1 full plastic moments.
4. 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 the type of connection, for example in the case of coverplate, it is greater than haunch system.

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