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EFFECT OF CONTINUITY PLATE ARRANGEMENT ON SEISMIC BEHAVIOR OF PANEL ZONE WITH UNEQUAL BEAM DEPTH FOR INTERIOR COLUMNS IN SMRFS

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ABSTRACT

There is an ongoing research about seismic behavior of Panel Zone with equal beam depth on both sides, most of which consider exterior columns (one beam present) for experimental tasks. As a whole, behavior of panel zone is a function of its geometric properties surrounding boundary elements, such as beams, continuity plates' formation and doubler plate's thickness. In this study first FE model of SAC subassemblagement experiments and works carried out at UC. Berkeley (for interior columns) was verified using available results. Then analytical models were established for two arrangements of unequal beam depth (small and significant differences), four arrangements of continuity plates (inclined continuity plate, horizontal continuity plate with and without haunch, with and without cover plate usage on haunch) and various column depths. Obtained results indicate that in case of shallow columns, with smaller depth difference in beams and inclined continuity plates, panel zone has desirable seismic behavior. On the contrary, in case of deeper columns with various beam depth differences, horizontal continuity plates would lead to better seismic behavior of panel zone.

Introduction

There are several researches on PZ and connection before and especially after Northridge earthquake. Popov and his colleagues did some full-scale tests in 1988 at UC Berkeley, experiment included a subassemblagement of interior joint with equal beam depth with use of strength base design method to provide detailing of PZ such as PZ doubler plate. The tests conducted on CJP (groove weld) used in connecting beam flange to column flange interface

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which resulted in proper cyclic behavior of connection. Many researches show that the value of 0.8 to 1 of the plastic moment of beam could be a reliable amount for designing PZ doubler plate to produce some controlled plastic rotation on beam and PZ simultaneously [1].

The 1994 Northridge earthquake revealed serious damage to conventional bolted web-welded flange (BWWF) connections, which were formerly known as ductile moment connections. Since then, a great deal of research has been conducted on the existing moment connections to find deficiencies and to improve their cyclic behavior [2,3]. Experiments in these investigations generally performed on an exterior joint specimen (column with beam on both sides). Among various methods proposed for modifying the cyclic performance of the conventional connections, many modifications on connections were seen to possess remarkable superiority. Through extensive experimental studies [4,5], it is confirmed that one haunch system in moment connections can develop high inelastic deformations and attain acceptable plastic rotation. However, there is a major problem with this type of connection: degradation of the load-carrying capacity due to lateral and local buckling in beams. Cover plate is another type of modification and is greatly accepted in the terms of "repair". This type of modification indicates suitable plastic rotation and no remarkable degradation. Cyclic tests of ten post-Northridge WUF-B connections were reported in the connection database provided by SAC [2]. Unlike pre-Northridge WUF-B connections, those specimens were made using a notch-tough welding metal, an improved welding. Most specimens fractured on the beam flange, and cracks were initiated and passed through the end of the access hole cut. However, none of the specimens satisfied the criteria for the total rotation capacity of SMFs, which is 0.04 radians, specified by the AISC seismic provision [6]. Nevertheless, it was reported that the notch-tough welding metal and new welding procedure improved the performance of WUF-B connections. Based on research carried out after the Northridge earthquake (SAC phase 1 and 2 steel projects), Seismic Provisions were significantly revised. As it was stated, modification was successfully done and test result depicted significant improvement in cover plate and haunch connection.

The design criteria for the limit states applicable to continuity plate and doubler plate design for non-seismic conditions and doubler plate design for non-seismic conditions are provided in Section K1 of Chapter K of the American Institute of Steel Construction (AISC) Load and Resistance Factor Design (LRFD)[8] Specification for Structural Steel Buildings (AISC, 1999a). In addition, there are more stringent requirements for Special Moment Frames (SMF) in the AISC Seismic Provisions for Structural Steel Buildings (AISC, 1992)[9]. However, the 1997 and 2002 AISC Seismic Provisions (AISC, 1997a, 2000, and 2002) removed all design procedures related to continuity plates, requiring instead that they be proportioned to match those provided in the tests used the connection.

Recent research has revealed that excessively thick continuity plates are unnecessary. El-Tawil[10] and others (1999) performed parametric finite element analyses of girder-to-column joints. They found that continuity plates were effective as the thickness increased to approximately 60 percent of the girder flange. However, continuity plates more than 60 percent of the girder flange thickness caused diminishing returns. Furthermore, over-specification of column reinforcement may actually be disadvantageous to the performance of connections. As continuity plates were made thicker and attached with highly restrained CJP welds, they sometimes contributed to cracking during fabrication. (Tide, 2000)[11]. In this study all of the analytical models were designed according to FEMA273[12] which results in less thickness of continuity plate.

This study aims at considering the PZ behavior due to continuity plate configuration and details of the connection. In this study four categories of detailing including inclined continuity plate, straight continuity plate with haunch, straight continuity plate without haunch and straight continuity plate with haunch and cover plate which would be implemented on haunch, are considered for the case of unequal beams.

Tests overview

Popov [13] and Colleagues performed some tests to evaluate the behavior and role of PZ in MRFs. Some specimens resembling interior joints were made for these tests at UC Berkeley. In addition, cyclic tests were conducted on 12 specimens constructed by SAC joint venture and some experimental works have been done by Popov [2], Whittaker [14], Blondet [15] and Shuey [16]. Fragile behavior of WUF-B observed in SAC experiments caused changes in configuration and type of connections. As it was depicted, some modifications were made in some connections, such as one sided cover plate to improve the performance of these types of connections. Despite the first test ensemble of Popov experiment, the second group resembled exterior columns.

Because of variety of geometry and material, some specimens from interior and exterior joints were selected for verification in this research. Regarding some description in this chapter specimen No.8 from Popov experiments which indicates interior joint with WUF_B connection and from second group (exterior case), (UTA-4),(UCB-RN3) and(UTA-1R) specimens were selected to calibrate analytical model. Specimen (UTA-4) employed cover plate connection and in specimen (UTA-1R) haunch was used at the bottom and cover plate at the top. In addition in (UCB-RN3) double-sided haunches were employed. The main reason of considering these specimens in this study was the presence of these kinds of connections in the analytical phase of this research. Load sequence applied according to ATC and SAC protocol and material properties.

Modeling and verification

To evaluate the accuracy of finite element modeling approach, four finite element models are created based on the actual tests. Fig 1 and 2 show one of the Von-Mises stress distributions and the main model after conducting test. To satisfy boundary condition of analytical model, the end of beam is restrained for outward motion. Because of existing lateral bracing system in real model (on beam's flange) some points in the model are also restrained. Since there was no information about the situation of bolt regarding pre-tension or ordinary twisting, it was considered as ordinary which would not permit shear tab to slip outward the plane of web. The loading procedure was displacement control, and it was done in compliance with SAC test protocol [17,18] as it was considered in actual test. Monotonic loading was applied to produce moment.

The material properties of these models assumed to have kinematic behavior with strain hardening in nonlinear phase to produce a more realistic prediction. The stress-strain relation for all connection components except for the bolts is represented using a tri-linear constitutive model. An isotropic hardening rule with a Von-Mises yielding criterion is applied to simulate plastic deformations of the connections. ASTM 36 steel was used for the beams and ASTM 52 steel was utilized for the column and connection details.

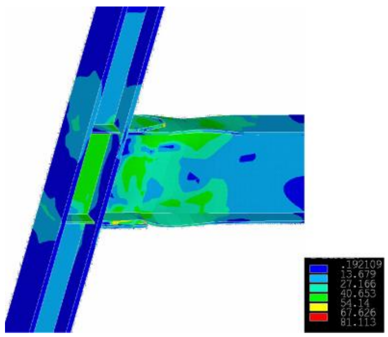

Figure1. Distribution stress of analytical model (UTA4R)

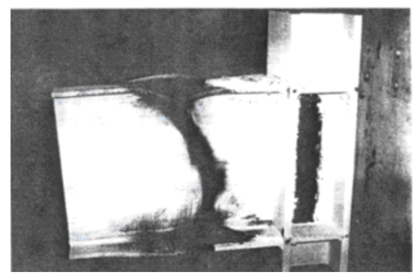

Figure 2. Deformed shape of specimen (UTA4R)[17]

In the current study, the mechanical properties of beams, columns and connections are taken from ref [17].The yield stress and ultimate strength of bolts 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) [18]. Modulus of elasticity and Poisson's ratio are considered respectively to be 29000 kips/in2 and 0.3.

Analytical and experimental hysteric behavior of beam plastic rotation versus applied moment are shown in figs3 to 6 for UTA-4 AND and UCB-RN3 specimen as a sample. From these figures, it can be seen the results obtained from finite element models have good agreement with test data.

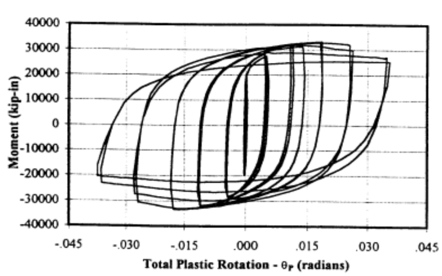

Figure 3. Hysteretic behavior of test (UTA-4) [17]

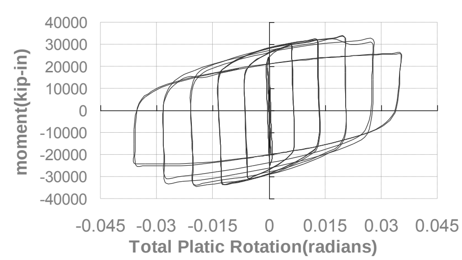

Figure 4. Hysteretic behavior of numerical model (UTA-4)

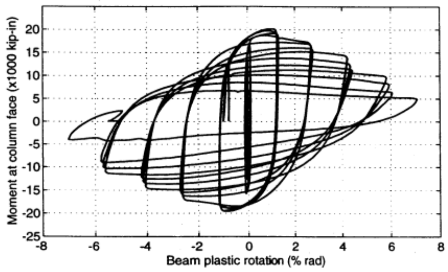

Figure 5. Hysteretic behavior of test (UCB-RN3) [17]

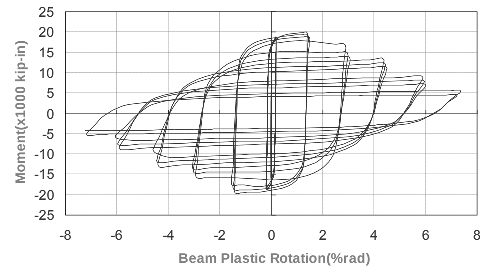

Figure 6. Hysteretic behavior of numerical model (UCB-RN3)

Differences between the numerical simulation and test results may be the result of several causes like numerical modeling simplification, test specimen defect or residual stress. In addition, the material properties, which are used in FE, are from average, but result. The differences between the test data and the numerical models grow in nonlinear portion of curve.

Analytical models description

Three groups of assembled models have been used in this article; their only difference was the depth of their columns. Three variable beam heights with definite height differences were considered in each group. In each definite height difference of beams, the panel zone was designed based on strength (IBC2000)[19]. Its thickness was found using the mentioned code, four arrangements of continuity plate formation and type of connection were considered: inclined continuity plate, straight continuity plate, straight continuity plate and one-sided haunch and finally straight continuity plate and one sided haunches with inclined cover plate on the haunch. Therefore, 24 models were created. The important point is the use of solid45 element, which was used with almost the same size for verification of analytical models in previous section.

Table1: Geometric properties of beams

Type of beams	web height (cm)	flange width (cm)	Thickness of web(cm)	Thickness of flanges(cm)
Beam 50	50	15	1	1
Beam 40	40	15	1	1
Beam 30	30	15	1	1

Table 2: Geometric properties of columns

Type of columns	web height (cm)	flange width (cm)	Thickness of web(cm)	Thickness of flanges(cm)
column35	35	25	1	1.5
column 45	45	20	1.2	1.5
Coloumn55	55	20	1.2	1.5

All models were designed according to IBC2000. Geometric properties in all series and models are included in Tables 1 and2. The naming convention of analytical model is presented in table 3: for naming the analytical models, 1 to 6 are assigned to first term of type numbers. Type 1 in comparative figure (in result chapter) is assigned to: column 35 and right beam height 50(cm) and left beam height 40(cm). The continuity plate arrangement is shown by the second number in the result chapter , No. 1 shows inclined continuity plate and so on. Table 3 shows complete naming convention reference. For example, 4-2, will be used for Type four with second continuity plate arrangement model. Applied load displacement controlled, also applied displacement on small beam tip is equal to reverse ratio of big beam depth to small beam depth multiple at applied displacement on big beam, for example, in type 1, applied loading displacement on beam30 (Δ_{Beam30}) is equal to $\frac{(height)_{Beam50}}{(height)_{Beam30}}.\Delta_{Beam50}$. This was originated from equality of stress level at the column flanges on both sides in linear phase and it can be proved empirically [20]. Strong column weak beam ratio in type1 to type 6 are 1.12, 1.05,1.3 , 1.21,1.6,1.45respectively.

Table 3: Naming convention

<table>
<tr><th rowspan="2">First term in Type naming</th><th rowspan="2">Column (cm)</th><th rowspan="2">Right Beam (cm)</th><th rowspan="2">Left Beam (cm)</th><th colspan="4">Continuity Plate Arrangement (second term in naming)</th></tr>
<tr><th>1</th><th>2</th><th>3</th><th>4</th></tr>
<tr><td>1</td><td>35</td><td>50</td><td>40</td><td rowspan="6">inclined continuity plate</td><td rowspan="6">Straight continuity plate</td><td rowspan="6">Straight continuity plate with one sided-haunch connection</td><td rowspan="6">Straight continuity plate with one sided haunch and inclined cover plate on haunch</td></tr>
<tr><td>2</td><td>35</td><td>50</td><td>30</td></tr>
<tr><td>3</td><td>45</td><td>50</td><td>40</td></tr>
<tr><td>4</td><td>45</td><td>50</td><td>30</td></tr>
<tr><td>5</td><td>55</td><td>50</td><td>40</td></tr>
<tr><td>6</td><td>55</td><td>50</td><td>30</td></tr>
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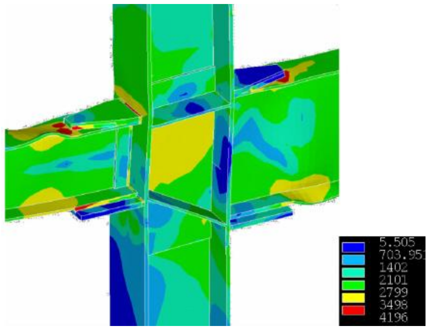


Figure 7. Von-Mises stress distribution (Type1-1)

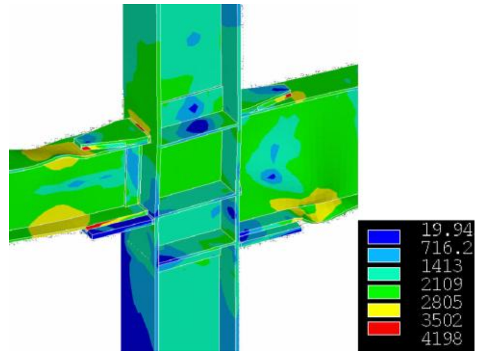


Figure 8. Von-Mises stress distribution (Type 1-2)

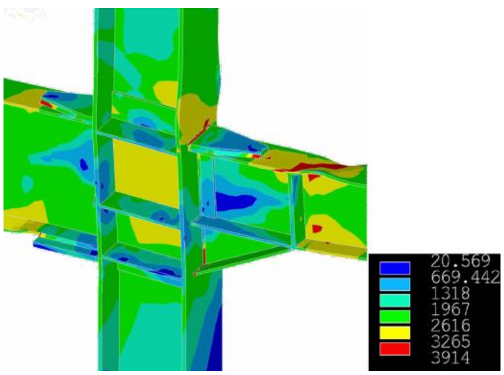


Figure 9. Von-Mises stress distribution (Type 1-3)

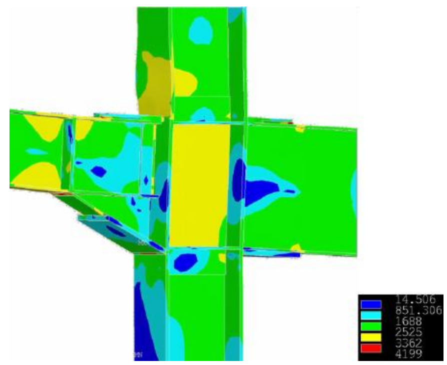


Figure 10. Von-Mises stress distribution (Type 1-4)

Numerical results

Analyses of defined models were performed considering and employing mentioned issues in the previous sections. Type of analysis was nonlinear static and effects of buckling and large displacements were also included. Vonmises failure criterion were used in the analysis which is correct for ductile materials [14]. Results of analyses consist of applied moment vs. Panel zone shear strain curves. Total moment versus shear strain of PZ for Type 1 models are shown in figs 11 to 14 as samples.

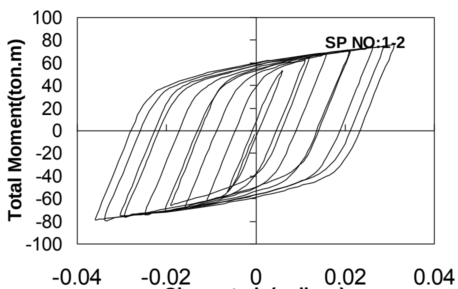


Figure 11. Total moment versus shear strain of PZ (model 1-1)

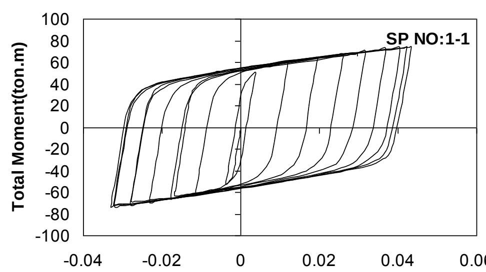
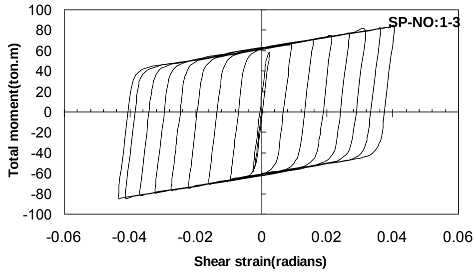


Figure 12. Total moment versus shear strain of PZ(model 1-2)



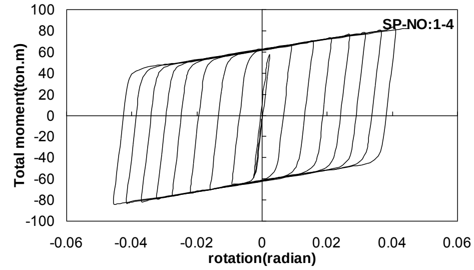
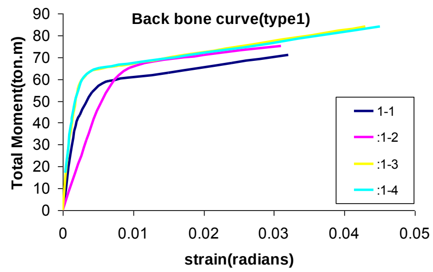


Figure 13. Total moment versus shear strain of PZ(model 1-3)

Figure 14. Total moment versus shear strain of PZ(model 1-4)



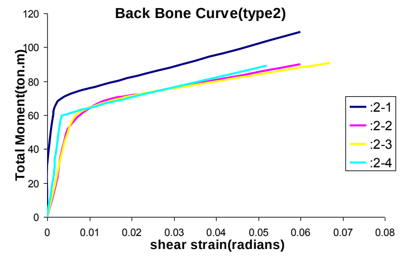
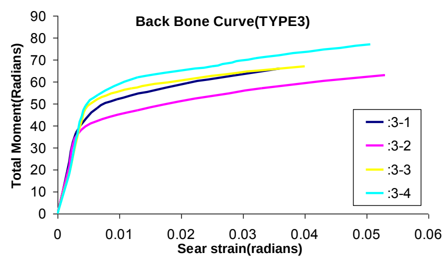


Figure 15. Backbone curve of PZ seismic behavior(type1)

Figure 16. Backbone curve of PZ seismic behavior(type2)



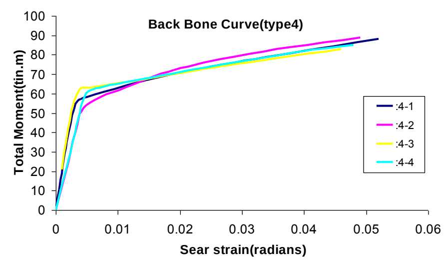
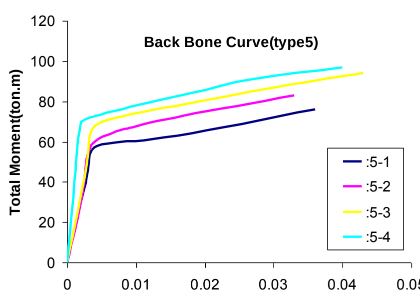


Figure 17. Backbone curve of PZ seismic behavior (type3)

Figure 18. Backbone curve of PZ seismic behavior(type4)



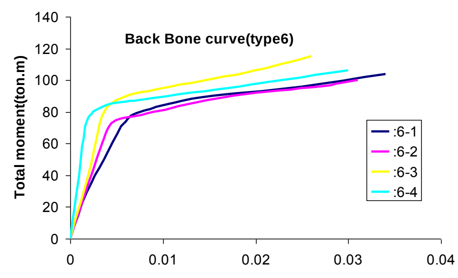


Figure 19. Backbone curve of PZ seismic behavior (type5)

Figure 20. Backbone curve of PZ seismic behavior(type6)

The backbone curves of PZ for analytical models are shown in figs 15 to 20. These figures show that generally, in the case of 50-30 side beams (height of left beam is 50 cm and, height of right beam is 30 cm), PZ seismic behavior for every column depth in continuity plate configurations 3 and 4 show better behavior, especially, configuration 4 in type 3 and type 5 shows more acceptable behavior. It means that straight continuity plate with one-sided haunch with use of cover plate on the haunch connection shows better response in linear and non-linear phase in these cases. In analytical model with shallowest column depth (35cm) and least differences in beams height (beam50-beam40), inclined continuity plate (continuity plate configuration1) PZ behaves stronger. It is worth mentioning that continuity plate configuration 4 with all beam height differences has the most appropriate behavior when column height deepens.

Conclusion

Regarding obtained results different arrangement of continuity plate in line with type of connection, such as haunch system and haunch system with cover plate connection would lead to different seismic behavior. This difference may reach thirty percent in some configurations and following conclusions can be made:

1. Type 2 series with fourth continuity plate arrangement behaves more stronger in the case of shallower column and small difference between beams' height (beam 50 and beam40).
2. The third and fourth continuity plate configurations are more resistant than the others are when the difference between beams' height are noticeable (type1, 3 and5).
3. Between third and fourth pattern, fourth pattern have more initial stiffness, when depth of column increases, overall, fourth pattern indicates more ultimate strength and initial stiffness.
4. Second continuity plate configuration, generally, does not have any advantage over other configurations and PZ of this arrangement shows below average behavior.

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