Research Paper

Improved Simple Model of Steel Beam-Column Connection for Simultaneous Application of Shear and Flexure Behavior on It

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ABSTRACT

Beam-to-column connection in the Moment-resisting frame plays a significant role in the seismic responses of the structure, including its ductility. Panel zone is subjected to large asymmetric anchors under seismic loads, which causes shear ductility. Therefore, an accurate estimation of the panel zone behavior during the structure's design is essential. Existing relationships have problems in estimating the behavior of the panel zone. Some relationships, such as the Krawinkler model, have good accuracy but are highly complex in the modeling process, and some of the proposed models for estimating the behavior of the panel zone, such as the scissors model, are easy to use. However, they do not have good accuracy. For this reason, engineers typically abandon panel zone modeling when using software such as SAP and ETABS to design structures, and consider the panel zone to be rigid. This method of modeling the panel zone in short-rise buildings does not have much effect on the deformation of the frame. However, with increasing the height of the structure, the effect of accurate modeling of the behavior of the panel zone on the overall behavior of the structure will increase. Given this issue, it is necessary to provide an accurate and, simultaneously, simple numerical model to predict the behavior of the panel zone. For this purpose, in this research, first, the weaknesses and strengths of the existing models were carefully studied, and then by combining these models and performing various analyzes in OpenSees software, the final model was presented. In this numerical model, a torsional spring with trilinear behavior is used to simulate the behavior of the panel zone, considering both the bending and shear effects. The relationships used in the proposed model are derived from Krawinkler relationships that have been optimized for the conditions of the new model. Using this model, the independent degree of freedom of the panel zone has been reduced by 75% compared to the Krawinkler model, and the capacity of the panel zone in the non-linear region 7has been increased 6.8 times and is almost identical to the experimental results.

Keywords:
Steel moment frame system; Panel zone; Nonlinear analysis; Krawinkler model; Scissors model

1. Introduction

Seismic design of structures to minimize damage caused by earthquakes is important and necessary. In lateral loads, many asymmetric moments are applied to the beam to column joint, which will cause many bending and shear deformations at the panel zone. Previous research has shown that the panel zone has shear deformation, ductile behavior, and stability under reciprocating loads (Fielding & Huang, 1970; Krawinkler et al., 1971; Kato et al., 1988; FEMA 451, 2006; Ma & Tan, 2023)

These panel zone characteristics have caused it to be effective in the shear deformation of flexural frames (Lui & Wai-Fah, 1986; Schneider & Amidi, 1998). Therefore, the importance of the panel zone as one of the most important parts in the behavior of joints in the moment frame has long been considered by researchers.

In this case, a lot of research and experiments have been done to evaluate the panel zone's behavior. It should be noted that the effect of panel zone modelling on the maximum displacement experienced by short-rise moment frames in seismic loading is negligible, but in high-rise and mid-rise frames, it can have significant effects on the displacement and drift experienced. Therefore, panel zone modelling is required in high-rise mid-rise structures (Hatami et al., 2014). Corman et al. (2022) in their study examined the panel zone of welded beam to column joints under monotonic load. They performed a parametric study on (non-) axially loaded (un-) stiffened exterior and interior joints. As a result of these studies and the performance of multiple finite element analyzes, they proposed a model that offers a more coherent analysis of the plastic shear strength of panel zone (Figure 1).

In 1971, Krawinkler et al. (1971) studied the behavior of 10 full-scale beams to column connections. This analysis showed that brittle fractures usually occurred in the heat-affected zone in the joints, and the lower flange of the beam was broken. This study fully elucidated the effect of panel zone participation as a deformable

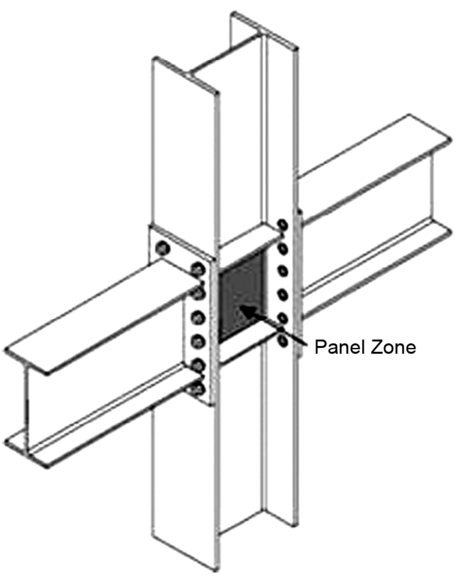

Figure 1. The panel zone.

member. Studies by Krawinkler et al. were continued in 1973 by Bertero et al. (1973). In his studies, Bertero evaluated the effect of panel zone shear deformations on the final deformation of the frame and its effect on the period of the structure in the frame design. In 1985, Popov et al. (1985) tested eight different connection specimens in their study. As a result of this research, the superiority of penetration weld with groove angle was more than the superiority of one-way fillet weld. In 1988, Tsai & Popov (1988) evaluated 10 full-scale connections in their study. This study showed that the placement and number of web bolts effectively connect ductility. In another study, Tsai et al. (1992), who tested the BOX shape column, showed that if the width of the beam flange was smaller, less hardening moment would be generated, which is why a joint in an earthquake is not enough for the required ductility. In the 1994 Northridge earthquake, many joints were broken by penetration welds, which in many cases extended to the column flange. FEMA research on the effects of this earthquake proved that direct connection failure occurred due to three-axis stress conditions in the heat-affected zone. In 1998, Englehardt et al. Engelhardt & Sabol (1998) tested 12 full-scale samples of cover plates in their experiments. Out of 12 specimens tested, 10 specimens showed appropriate seismic behavior.

In 2000, the FEMA-355D (2000) and FEMA-451 (2006) provided a comprehensive summary of connection performance. This study evaluated different details of the cover plate and flange plate implementation performed in previous experiments. Fracture modes in the cover plate and flange plate were also accurately expressed. Based on this, the research results conducted after the Northridge earthquake have been described in detail. Based on the test results, the cause of cracks in Northridge joints is divided into four general categories:

- The improper shape of the connection leads to stress concentration, strengthens the three-axis condition, and limits the flow strength.
- Use of low-strength weld filler metal, which leads to premature fracture.
- The use of materials with various mechanical properties (including yield strength of beams and

columns) leads to the production of poorly unpredictable areas in the joint area.

- Improper welding operations that do not meet the minimum requirements of the welding codes.

Studies of fracture modes of each type of connection are given separately in the FEMA instruction series.

In 2002, Kim et al. (2002) performed 10 full-scale tests of the cover plate and flange plate details. In these experiments, the superiority of the penetration weld proved the superiority of cover plate and joint plate joints over other joints. Also, in 2007, Chou et al. tested flange plate joints by reducing the cross-sectional area of the flange plate. This experiment showed that these joints are more efficient than flange plate joints.

According to research, the yielding of the panel zone usually starts from the middle of it and then spreads radially in other places, which minimizes these shear waves in the corners of the panel zone. The presence of axial stress in the column accelerates the shear yield of the column, which is also in line with the Von-Mises yield criterion. Of course, it is important to note that the presence of axial force does not have much effect on the final strength of the panel zone because when shear yield occurs, the column flanges withstand the axial stress of the column, and a small amount of it is transferred to the panel zone. If at large shear strains the column flanges reach their final plastic capacity, the column flanges will find large buckles that may cause the joint to fail. The reason for this is the creation of extensive strains in the beam to column connection welds or near them (Krawinkler et al., 1971; Becker, 1975). Once the panel zone is yielded, its shear capacity is drastically reduced, and the column flanges and web stiffeners surrounding the panel zone are resisted. According to the experimental results, the deformation that results in the full yield of the panel zone is four times the deformation that gave rise to the initial yield of the panel zone. Before the panel zone is affected by shear buckling damage, It can experience large plastic deformations. However, if the joint deformations are large, it will cause poor column performance and failure of the weld joints. Panel zone has an influential role in energy loss in the structure and can be effective in re-hardening. As mentioned, the panel zone can help to waste energy in the structure by its large deformations, but if these deformations are excessive, it will affect the overall performance of the structure. Therefore, in the designs, specific limits must be set for the plastic deformation of the panel zone Foutch & Yun (2002).

- The panel zone, once it reaches its initial yield, can withstand much larger plastic deformations with the help of column flanges and strain hardening.
- Panel zone deformations can be added to the overall steel moment frame deformation in extreme elastic and inelastic behavior ranges.
- To increase the strength of the panel zone, you can use a doubler plate or

Stiffener plate, but the extent of the effect of these cases depends on how these plates are connected to the column.

- If the panel zone is deformed too much, it can cause the failure in welds connecting the beam to the column.

Studies by Pan et al. (2016) and Del Carpio et al. (2016) Indicated that the panel zone may be slightly deteriorated due to shear forces. If shear buckling or doubler plate failure does not occur, this shear distortion will spread. To prevent shear buckling, you should follow the code requirements. It cannot be considered unless there is a clear cause for deterioration in the maximum considered earthquake (MCE). Brandonisio et al. (2012) in their study, which evaluated these European and American letters, found that American codes, unlike Europeans, provide a good evaluation of the shear strength of the panel zone. Sazmand and Aghakouchak (2012) studied the effect of panel zone on steel moment resistant frames composed of built-up columns and proposed an analytical model for the panel zone of connections of steel moment-resisting frames (SMRFs) composed of built-up columns with double sections and a vertical continuity plate. Tuna and Topkaya (2015) introduced a numerical model for evaluating the shear demand of different panel zones with different specifications. Saffari et al. (2016) conducted a parametric study using the finite element method considering the effective parameters in panel zone behavior such as column flange thickness, column web thickness, and thickness of continuity plates. They also presented a numerical model for measuring the panel zone strength of cruciform columns. Augusto et al. (2016) studied the effect of beam-to-column double extended end-plate steel joint, using finite element analysis. According to the results of this study, the behavior of the panel zone was completely ideal using a beam-to-column double extended end-plate steel joint. Pan et al. (2016) in their study examined the effect of vertical plates within the panel zone. As a result of this study, vertical plates inside the connection spring were introduced to reduce the potential for shear buckling in slender panel zones.

2. Background of Design Criteria

In a rigid connection as shown below (Figure 2):

According to the FEMA 1976 code, the panel zone shear demand is calculated from the following formula:

$$V_{des} = \frac{M_l}{d_l - t_{fl}} + \frac{M_r}{d_r - t_{fr}} - V_{col} \quad (1)$$

$$V_{col} = \frac{M_l + M_r}{H} \quad (2)$$

M_l and M_r are the bending moments of the beam, d_l and d_r are the depth of the beams, t_{fl} and t_{fr} are the flange thickness of the left and right beams, V_{col} is the shear of the column and H is the height of the column.

In AISC 1978, the capacity of the panel zone is calculated by the following formula:

$$V_{max} = 0.4 F_y d_c \quad (3)$$

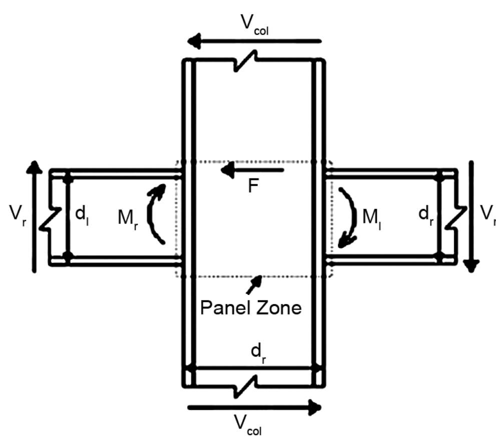

Figure 2. Forces and moments on the zone panel.

where F_y is the steel yield stress and dc is the column depth.

If seismic forces are applied, the capacity of the panel zone will be calculated as follows:

$$V_{max} = 0.53 F_y d_c t \quad (4)$$

Moreover, if we want to plastic design a panel zone, the capacity of the panel zone will be calculated as follows:

$$V_{max} = 0.55 F_y d_c t \quad (5)$$

In all of the above formulas t is the zone panel thickness.

A center-to-center linear model was used to draw a panel zone in the past. Figure (3) Comparison between the center-to-center linear model and the Experiment conducted by Krawinkler et al. (1971). This comparison clearly shows that the accuracy of the traditional panel zone drawing model is very low center-to-center, and new models have to replace it.

In recent years, researchers have introduced different methods for modelling panel zone behavior, and here are three commonly used methods for estimating panel zone behavior:

- Krawinkler model
- Scissors model
- Finite element based model

Each of these models will be described below.

2.1. Krawinkler Model

This method uses four rigid elements with joint connection and two rotational springs for

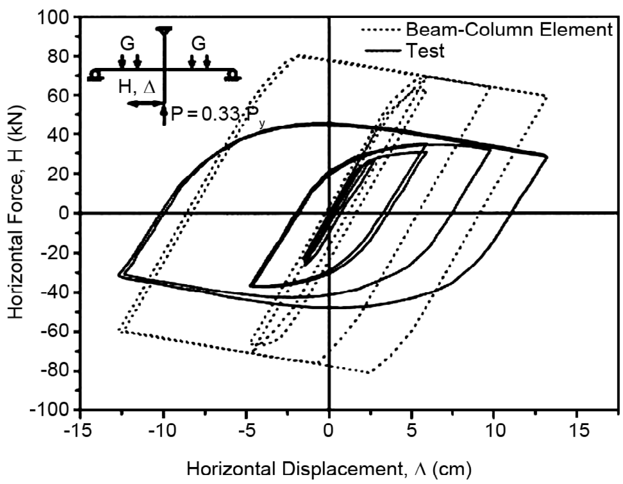

Figure 3. Comparison of center-to-center linear drawing model with experimental results (Krawinkler et al., 1971).

panel zone modelling.

As shown in Figure (4), four rigid elements are connected to each other. Two zero-length springs are used at one end of this quadrilateral, and the connection at the other three ends is simply a simple hinge connection. It should be noted that both springs used are zero length and are only responsible for transferring the moment between the rigid elements and have no other effect on the model behavior. The specifications of the springs used in this model are calculated according to FEMA-355C (2000).

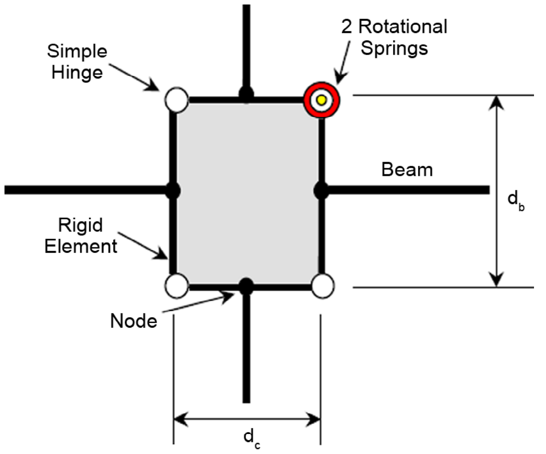

Figure 4. Krawinkler model.

It should be noted that as shown in Figure (5), due to the simplicity of using the Krawinkler model, the bilinear behavior of the springs is combined to become a spring with trilinear behavior. From now on, instead of using two springs, an equivalent spring is used.

The equivalent spring specifications are as follows:

$$V_y = \frac{F_y}{\sqrt{3}} A_{eff} = \frac{F_y}{\sqrt{3}}(0.95 d_c t_p) \approx 0.55 F_y d_c t_p \quad (6)$$

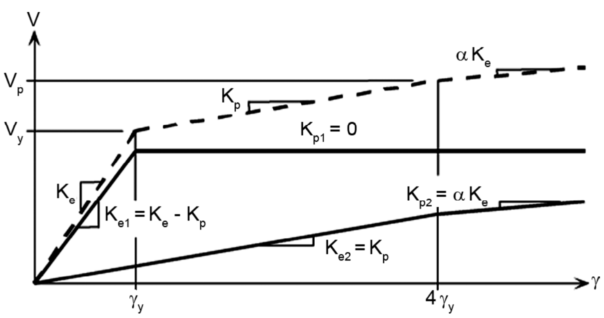

Figure 5. Combining the behavior of two springs of the Krawinkler model to the equivalent spring (FEMA-355E, 2000).

$$\gamma_y = \frac{F_y}{\sqrt{3} \times G} \quad (7)$$

$$K_e = \frac{V_y}{\gamma_y} = 0.95\, d_c t_p G \quad (8)$$

$$V_p = V_y \left(1 + \frac{3K_p}{K_e}\right) \approx 0.55 F_y d_c t_p \left(1 + \frac{3 b_c t_{cf}^{\,2}}{d_b d_c t_p}\right) \quad (9)$$

V_y is the panel zone shear yield strength, F_y is the yield strength of the panel zone, Aeff is the effective shear area, d_c is the depth of the column, t_p is the thickness of the web including any doubler plates, γ_y is the yield shear distortion, K_e is the Elastic stiffness of panel zone, G is Shear modulus of the column material, V_p is the full plastic shear resistance, K_p is the post-yield stiffness, b_c is the width of column flange and tcf is the thickness of Column flange.

In Figure (6), the modified shape of the connection and the beams and columns connected to it can be seen according to the Krawinkler model.

FEMA 451 (2006) identifies the Advantages and Disadvantages of the Krawinkler model (Figure 7).

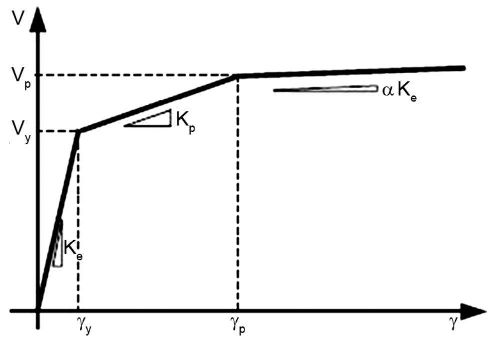

Figure 6. Trilinear panel zone behavior based on Krawinkler model (FEMA-355C, 2000).

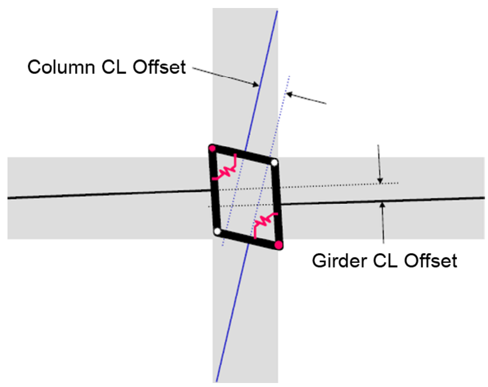

Figure 7. Connection deformation according to the Krawinkler model (FEMA 451, 2006).

Advantages of the Krawinkler Model

- Physically mimics actual panel zone distortion and thereby accurately portrays proper kinematic behavior
- Corner hinge rotation is the same as panel shear distortion
- Modeling parameters are independent of structure outside of panel zone region

Disadvantages of the Krawinkler Model

- Model is relatively complex
- Model does not include flexural deformations in panel zone region
- Requires 12 nodes, 12 elements, and 28 degrees of freedom (4 independent DOF).

2.2. Scissors Model

In this method, beams and columns are considered rigid elements and connected by a rotational spring (Figure 8).

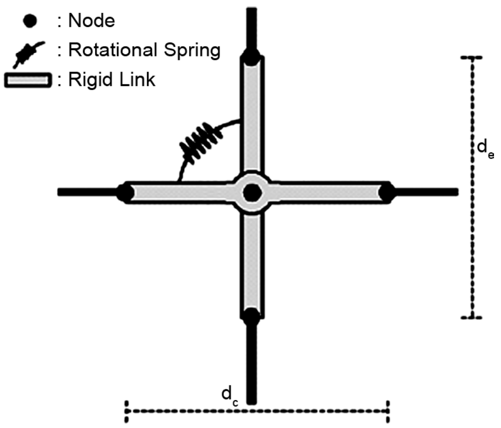

Figure 8. Scissors model.

As mentioned, in this method, panel zone modelling is performed by a rotational spring that transmits the moments entering the panel zone between the beams and columns connected to the connection. When modelling a panel zone in analytics software, no dimensions are considered for the panel zone element, and the beam and column are connected without any distance, but in this connection, a dimensionless spring is defined that can model the panel zone behavior. For this reason, the moment transmitted between the beam and the column by the panel zone changes the angle only to the extent of the relative rotation between the beam and the column (Figure 9).

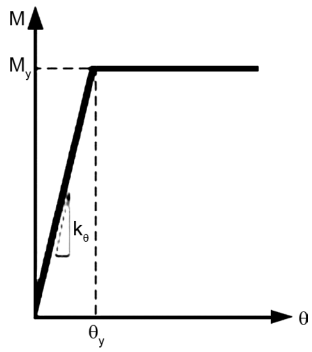

Figure 9. Scissors model behavior.

The relationships of the scissors model are:

$$\gamma_y = \frac{F_y}{\sqrt{3}G} = \theta_y \tag{10}$$

$$M_y = V_y \times d_b = 0.55 F_y d_c t . d_b \tag{11}$$

$$K_\theta = \frac{M_y}{\theta_y} = 0.95 G d_c t d_b \tag{12}$$

γ_y is the yield shear distortion, F_y is the yield strength of steel, G is Shear modulus, d_c is the depth of column, t is thickness of panel zone which is the thickness of the web of the column plus the thickness of the doubler plates if they are utilized, d_b is the depth of the beam (Figure 10).

According to FEMA 451 (2006) the advantages and disadvantages of this method are as follows:

Advantage of Scissors Model

- Relatively easy to model (compared to Krawinkler). Only four DOF per joint, and only two additional elements.

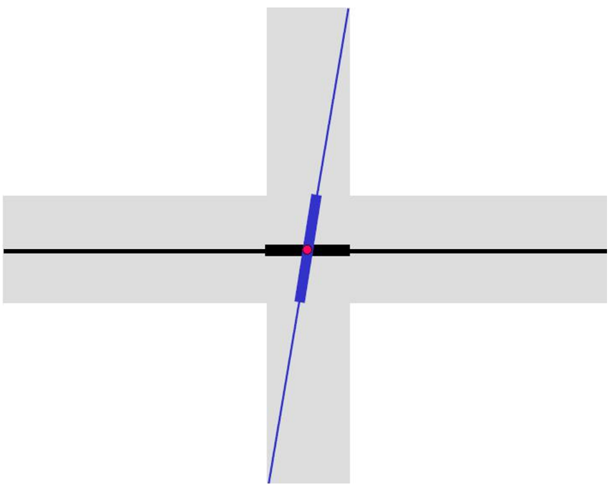

Figure 10. Connection deformation according to the scissors model (Foutch & Yun, 2002).

- Produces almost identical results as Krawinkler.

Disadvantages of Scissors Model

- Does not model proper behavior in the joint region.
- Does not include flexural deformations in panel zone region
- Not applicable to structures with unequal bay width (model parameters depend on α and β)

2.3. Finite Element Based Model

The finite element methods modelled the panel zone in this method (Figure 11).

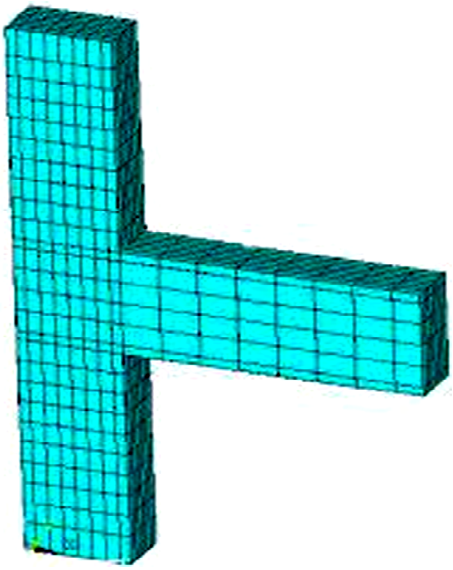

Figure 11. Connection modeling using finite element modeling, mesh size is 1/10 of column dimension.

Much research has been done by various researchers on the use of this method. These studies include Lui's research in 1986, Lui & Wai-Fah research in 1986, Bertero & Bertero research in 2002, and Mulas's research in 2004. This method is the most accurate method for modelling panel zone behavior, but its scope of application is very limited, and it is used in special cases due to its many complexities.

3. Methodology

This study aims to introduce a new model for predicting panel zone behavior that uses only one spring to model panel zone behavior. In the first step for verifying, we first select an experimental study as the baseline test. We then model this connection using the Krawinkler method in OpenSees software and compare the modelling results with the experimental results.

4. Experimental Test

A study conducted by Popov et al. (1985) was used for verification. This experiment studied eight specimens with different dimensions and executive details.

As shown in Figure (12), the lateral displacement at the top of the column is taken by a horizontal tool. Also, the whole set under test is connected to the ground by a diagonal tool with a slope of about

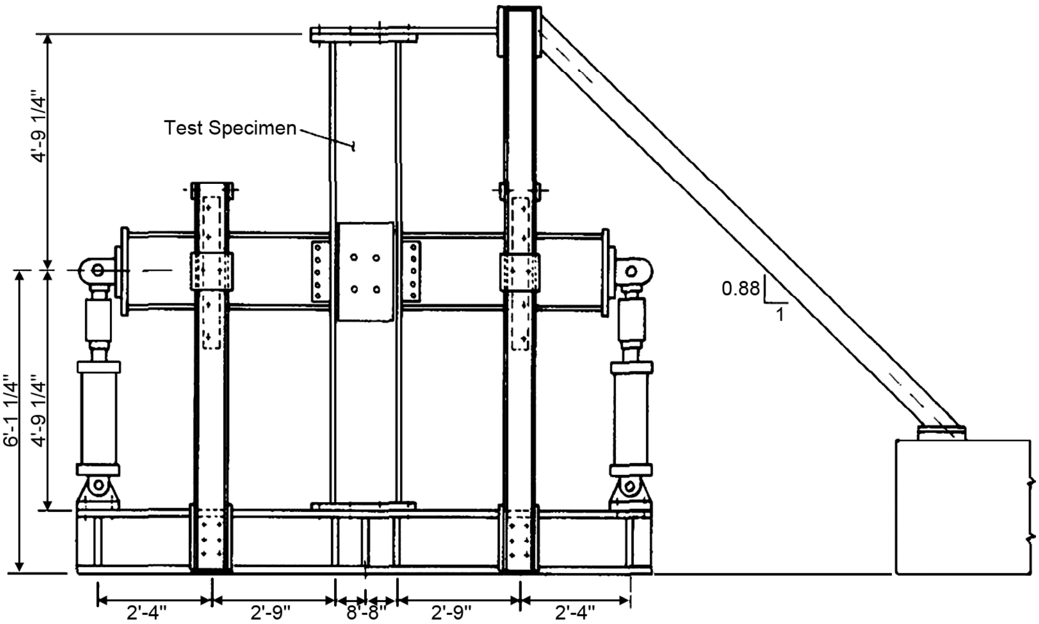

Figure 12. Test fixture for mounting specimen (Bertero et al., 1973).

41 degrees and a beam at the bottom of the set. Two vertical instruments controlled both beams' horizontal and off-plane displacement in the test specimen. In the case of loads, it should be noted that the loads applied to the beams are controlled by force and by two jacks, which can be seen in the figure, to the two ends of the beams by the endplates that are connected to the beams. The axial load applied to the column from the beginning and in a fixed manner is such that it creates a nominal stress of 21 ksi in the column, and this load will remain constant until the end of the test. As mentioned, the loading of the beams is force control, and the specimen is loaded cyclically. The load is applied so that one beam is loaded downwards, and the other beam is loaded upwards at the same time. In this load in the first two cycles, the load on the beam increases until the beams reach 50% of their nominal allowable bending stress, which is equivalent to 12 ksi. The other two cycles of the beams are then loaded to such an extent that the stress applied to them is equal to the allowable nominal bending stress of 24 ksi. In these elastic loading cycles, practical information will be obtained from the elastic behavior of the specimen. After these several loading stages, the amount of force increases to reach its nominal yield stress of 36 ksi, assuming the elastic behavior of the beam, which occurs once or twice. In the next stage, the force is so great that considering the behavior of the plastic, 36 ksi stress is created in the beam, which is also done once or twice. Up to this point, according to the obtained hysteresis curve, information about specimen ductility can be obtained. After this stage, the force applied to the beams in different cycles increases until the specimen is broken. The results are among the 8 specimens that were examined in this study. Because specimen number 8 hysteresis diagram was formed completely and stably, specimen number 8 was selected for verification (Figures 13 to 15)..

5. Model Verification

OpenSees software was selected to model the connection by Krawinkler method after reviewing the existing software in this field and comparing them.

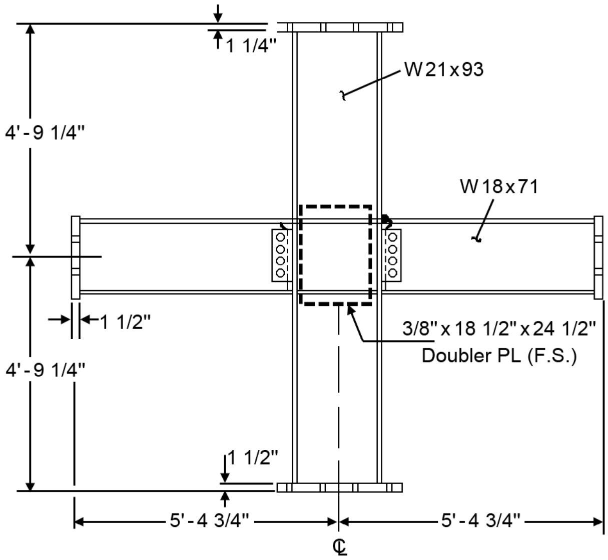

Figure 13. General configuration of specimen 8 (Popov et al., 1985).

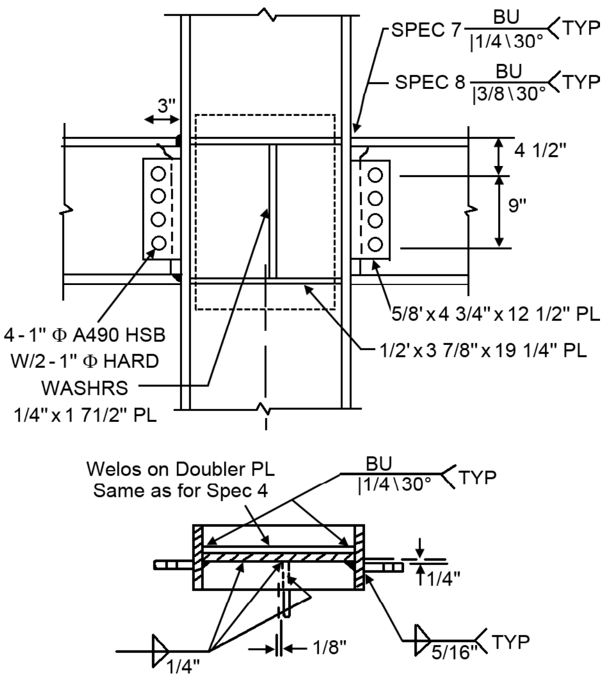

Figure 14. Details of specimen 8 (Popov et al., 1985).

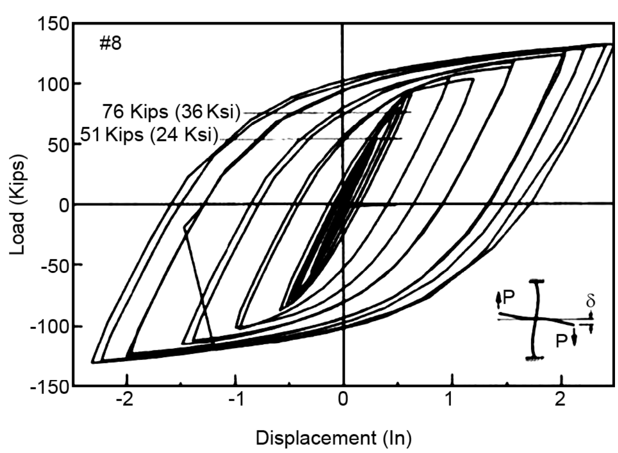

Figure 15. Average cantilever end load VS average beam end (Popov et al., 1985).

The capabilities of the OpenSees program can be described as:

- Modelling structures linearly and nonlinearly, as well as making high precision soil models
- Pushover nonlinear static analysis
- Incremental dynamic analysis (IDA)
- Cyclic analysis
- Multiple Support Excitation
- Dynamic time history analysis
- Uniform-support Excitation
- Multi support Excitation
- Apply relocations to supports
- Ability to apply accelerometer to the foot of the structure
- Ability to define accurate specifications of materials and diagrams of material hysteresis
- Define the isolator and operate the active and inactive controls
- High accuracy of the program compared to similar software
- High-speed program due to the possibility of error control and trial and error cycles
- Ability to follow the steps of analysis during the analysis
- Mathematical optimization of structures
- Ability to use in conjunction with other software to perform optimization
- Communication with Matlab and Simulink software for calculations

After selecting the software, the connection was modeled by Krawinkler in OpenSees. For Krawinkler modeling, four Nonlinear Beam-Column elements for modelling beams and columns connected to the panel zone, eight displacement-Based Beam-Column elements with axial and flexural rigidity for panel zone modelling, one dimensionless torsional spring torsion with trilinear behavior for shear deformation and flexural connection are used. It should be noted that for simplicity in modeling, instead of using two bilinear springs, one trilinear spring can be used to model the Krawinkler model. For this reason, a spring in the Krawinkler model is provided for ease of modelling (Figure 16).

After the connection is modelled according to the Krawinkler method in OpenSees, the boundary and abutment conditions are first applied, and it is done in two loading steps. First, the vertical load on the column is applied as constant and uniform stress, and in the second stage, the ends of the beams are loaded cyclically, and the structure is analyzed. After applying the loads and performing the analysis, the following diagram was extracted.

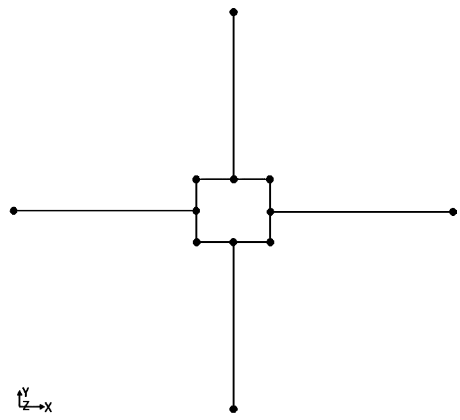

Figure 16. Krawinkler connection modeling in OpenSees.

According to the results, it is observed that there is a good agreement between the experimental results and the results of the analytical model. The difference in area under the curve diagram is less than 10% and the difference in the slope of the elastic region is less than 7%. The differences in the results are due to the lack of information in the experimental study. The Krawinkler model is also weak in flexural deformations in the panel zone region, as stated before. However, there is not much difference between the test results and the analytical model results.

After modeling the panel zone using the Krawinkler method in OpenSees, the specimen is also modeled in Abaqus software (Figure 17).

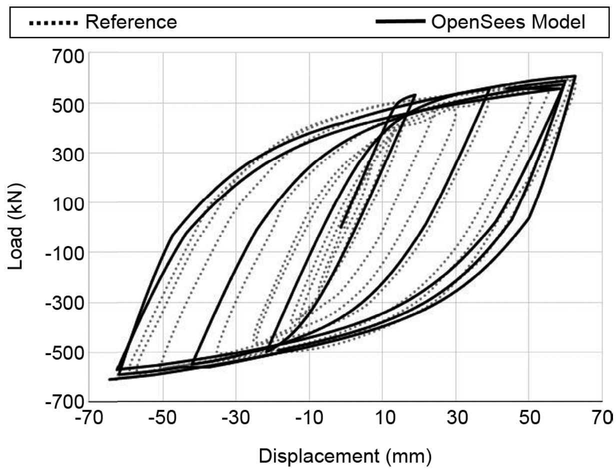

Figure 17. Comparison of OpenSees results with experimental results.

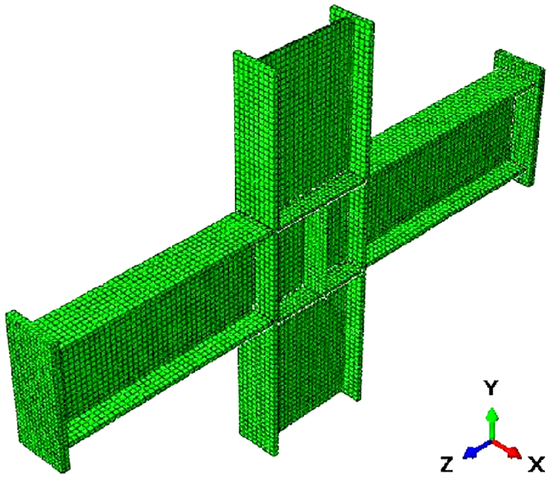

Figure 18. Modeling connection in Abaqus.

After modeling the connection in Abaqus by applying the boundary conditions and loading in accordance with the test and with the same conditions as modeling in OpenSees, the following results were extracted (Figure 18).

According to the above figure, there is a good agreement between the specimen finite element model and the experimental results. Of course, there is a slope difference of about 5% in the elastic range, which is negligible due to the test conditions (Figure 19).

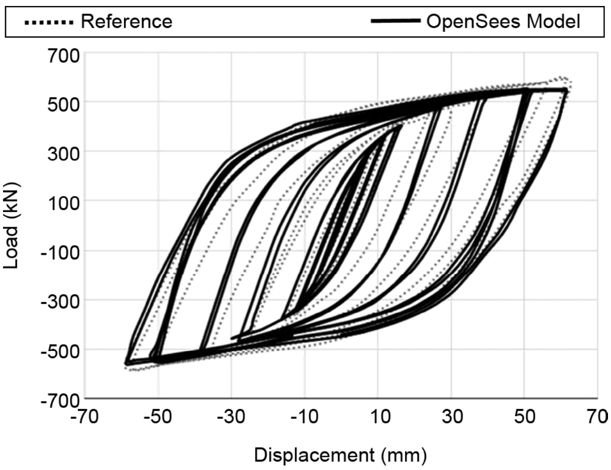

Figure 19. Comparison of Abaqus results with experimental results.

6. Introduce a New Model for Predicting Panel Zone Behavior

In determining the behavior of the spring used according to the Krawinkler model, a trilinear curve derived from the behavior of the Krawinkler model has been used. In defining the behavior of

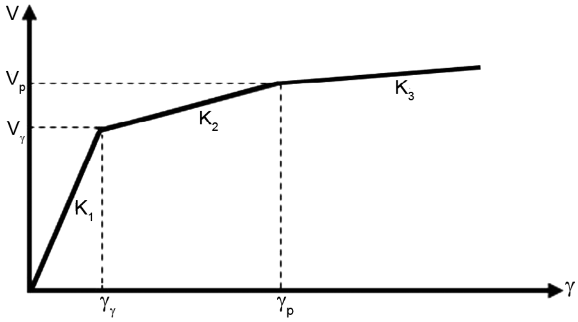

Figure 20. Trilinear spring behavior.

the proposed curve, the elastic stiffness (K_e) and panel zone shear yield strength (V_y) were selected according to the Krawinkler equation and K_2 and K_3 were extracted by trial and error after performing several analyzes. After determining the mentioned coefficients, various analyses were performed to determine the V_p coefficient to select the optimal coefficient for V_p relative to the Krawinkler relation (Figure 20).

According to the FEMA recommendation, the behavior of the trilinear panel zone in the Krawinkler model is defined as follows.

$$K_e = 0.95 d_c t_p G \tag{13}$$

$$V_y = 0.55 F_y d_c t_p \tag{14}$$

$$K_p = 0.95\, G\, \frac{b_c t_{cf}^2}{d_b} \tag{15}$$

$$V_p = 0.55 F_y\, d_c\, t_p \left(1 + \frac{3 b_c\, t_{cf}^2}{d_b\, d_c\, t_p}\right) \tag{16}$$

In this study, three coefficients α_1, α_2, α_3 are defined to optimize Krawinkler relationships.

α_1 coefficient of K_2 and α_2 coefficient of K_3 are considered. In order to obtain the optimal values of α_1 and α_2, several analyzes were performed, with a change interval of α_1 1 to 10 and a change interval of α_2 0.01 to 0.1.

$$K_2 = \alpha_1 \times K_{2(krawinkler)} \tag{17}$$

$$K_3 = \alpha_2 \times K_{3(krawinkler)} \tag{18}$$

The connection modeled in the software was then analyzed with all combinations α_1 and α_2 to extract the best combination. Then, to calculate the

V_p coefficient, the variable α_3 was defined, the change range of which was 0.2 to 1.6, and the optimal value of α_3 was selected from these cases. It should be noted that, as mentioned earlier, the K_1 and V_y relationships are the same as those proposed by FEMA and have not been changed. The following figures are the best combinations of α_1 and α_2, the resulting behavioral curves of which correspond well to the actual behavior of the panel zone.

In FEMA, the value of K_3 is equal to $\alpha_1 K_e$, while the value of α is considered equal to 0.01 in various sources. It should also be noted that the coefficient α mentioned in FEMA about the relationship between K_3 and has no relationship with α_1, α_2, α_3, which are the variables of this study (Figures 21 and 22).

Finally, the proposed relationships of this research can be presented as follows:

$$K_1 = K_e = 0.95\,G\,t_p\,d_c \tag{19}$$

$$K_2 = \alpha_1 \times K_{2(krawinkler)} = 6 \times 0.95\,G\,\frac{b_{fc}t_{fc}^2}{d_b} = 5.7\frac{G\,b_{fc}\,t_{fc}^2}{d_b} \tag{20}$$

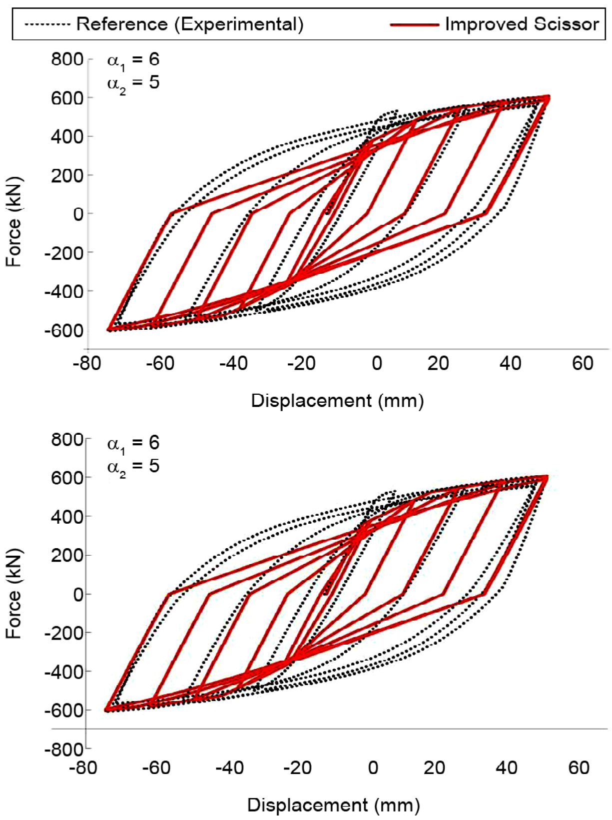

Figure 21. Comparison of the behavior caused by the impact of α_1 and α_2 with the actual behavior of the panel zone.

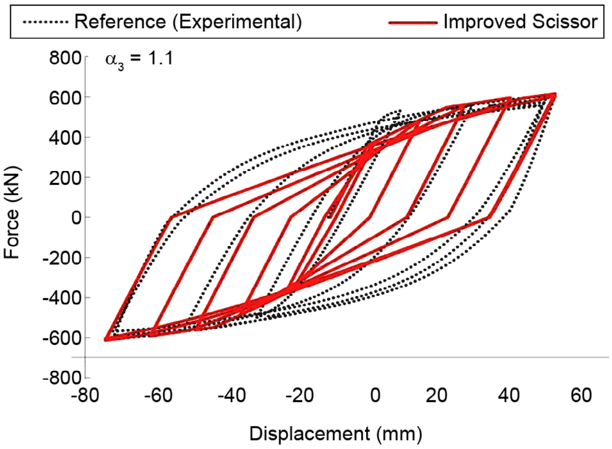

Figure 22. Comparison of the behavior caused by the impact of α_3 with the actual behavior of the connection.

$$K_3 = \alpha_2 K_{3(krawinkler)} = 5 \times 0.01 K_e = 0.05 K_e \tag{21}$$

$$V_y = 0.55 F_y d_c t_p \tag{22}$$

$$V_p = \alpha_3 \times V_{p(Krawinkler)} = 1.1 \times 0.55 F_y d_c t_p \times \left(1 + \frac{3b_c t_{cf}^2}{d_b d_c t_p}\right) = 0.605 F_y d_c t_p \left(1 + \frac{3b_c t_{cf}^2}{d_b d_c t_p}\right) \tag{23}$$

After comparing the behavior of the proposed model with the experimental behavior, the experimental specimen is modeled in Abaqus and the behavior of the model presented in this paper is also compared with the finite element analysis of the panel zone.

According to Figure (23), it can be seen that the area under curve of the improved scissors model is 77.5% of the connection modeling in Abaqus. The difference in the slope of the elastic

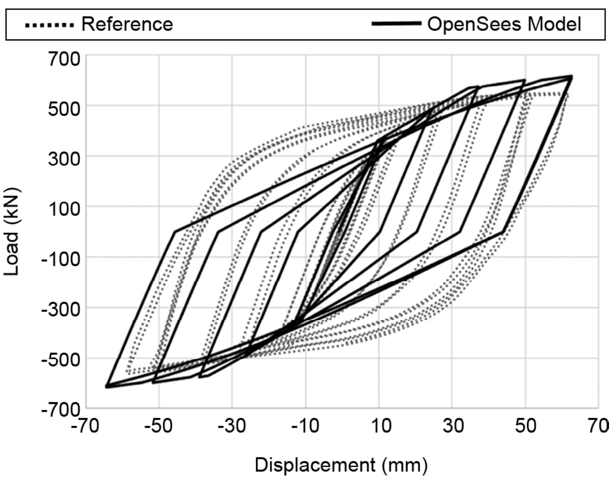

Figure 23. Comparison of Abaqus results with improved Scissors results.

region of these two diagrams is less than 3%, which is a good match between the experimental specimen finite element model and the results of the model presented in this paper.

If we use only shear or bending effect in panel zone modeling, the results will be as follows (Figure 24).

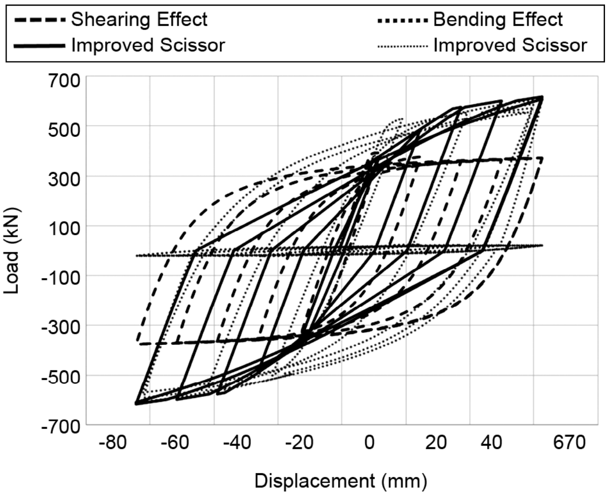

Figure 24. Panel zone modeling by considering the effect of bending and shear alone.

The effect of bending on the behavior of the panel zone is very small, and about 0.036% of the force applied to the panel zone is due to bending behavior (Figure 25). The shear effect alone models only 64.8% of the Force applied to the panel zone. According to the above figure, it is concluded that bending and shear alone are not a good representative for panel zone modeling and at best don't model more than 35% of the forces entering the panel zone. Also, the ratio of the area under curve in bending and shear modeling alone to the area under curve of the experimental test is 3% and 78.2%, respectively. The scissors model also does not provide an accurate estimate of the panel zone behavior because it merely uses the shear behavior of the panel zone to model its behavior.

Also, if we use the simple sum of bending and shear behavior to model the behavior of the panel zone, an accurate prediction of its behavior will not be obtained.

Also, the area under curve of the traditional scissors model is 73% of the area under curve of the experimental specimen (Figure 26).

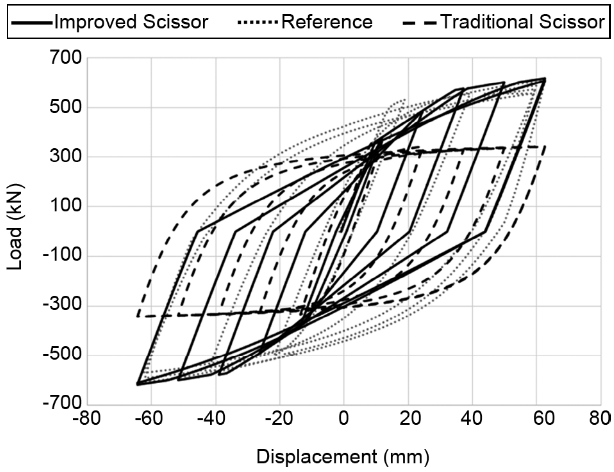

Figure 25. Comparison of panel zone modeling with traditional scissors model, improved scissors and actual panel zone behavior.

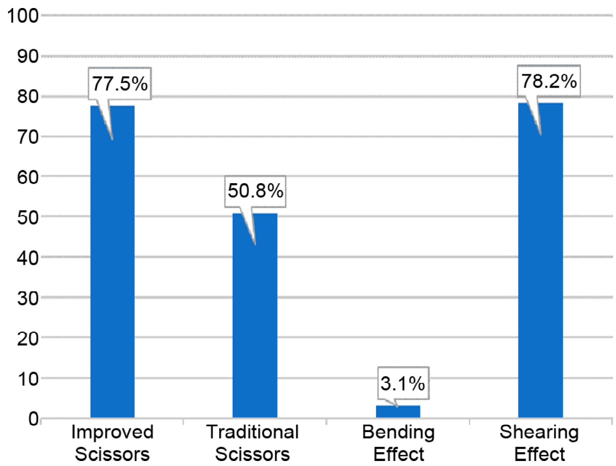

Figure 26. Statistical comparison of the area under curve of different connection modeling modes with the experimental specimen.

8. Investigating the Effect of the Proposed Model on the Performance of the Structure

To investigate the effect of using the proposed model, a 2D frame, 5-story, 3 bay is designed with the following specifications and in 5 different modes, it is modeled and analyzed by pushover method and nonlinear time history in ETABS software (Figure 27).

1. Rigid panel zone with rigid zone factor=0
2. Rigid panel zone with rigid zone factor=0.5
3. Modeling the panel zone using the traditional scissors method
4. Modeling of panel zone by Krawinkler model method (including 8 elements and 4 degrees of freedom)
5. Modeling of panel zone by modified scissors method (proposed behavior of this research)

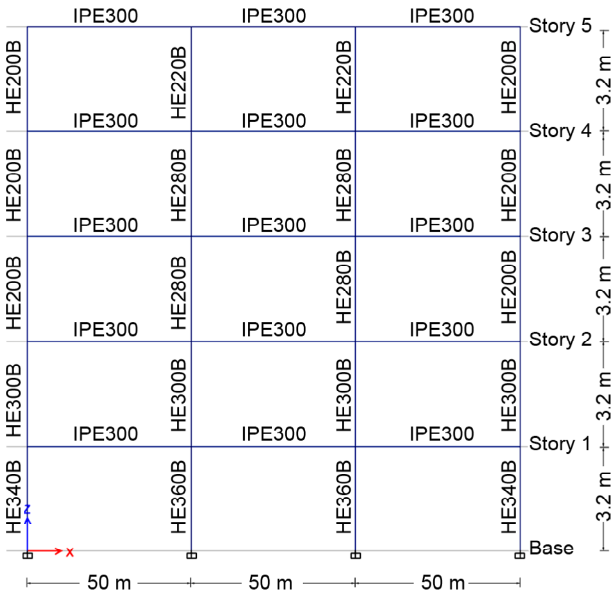

Figure 27. Analyzed frame specifications.

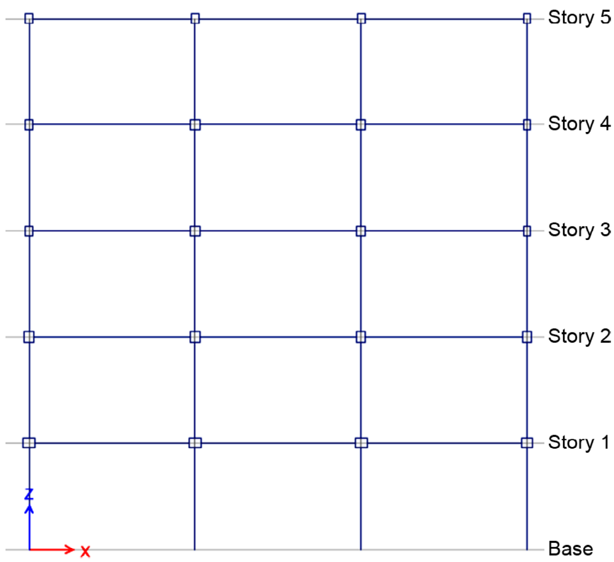

Figure 28. Frame modeling in ETABS software by panel zone modeling by Krawinkler method.

8.1. Structural Modeling Considering the Rigid Panel Zone

This is the usual method of engineers in designing structures and is also used in SAP and ETABS software. In this case, the dimensions of the panel zone are calculated according to the dimension of the beam and column, and the panel zone is modeled completely rigid. In this method, once the rigid zone factor is equal to 0 and once again equal to 0.5 in order to study the effect of relative rigidity that is considered for the panel zone in usual modeling.

8.2. Structural Modeling by Modeling the Panel Zone Using the Traditional Scissors Method

In this case, the behavior of the panel zone is modeled according to the behavior of traditional scissors according to relations (1-10) to (1-12).

8.3. Structural Modeling with Panel one Modeling by Krawinkler Method

In this case, the panel zone is modeled by the Krawinkler method. In this case, eight elements and 12 nodes have been used to model each panel zone (Figure 28).

8.4. Structural Modeling with Modeling of Connection Scissors by Improved Scissors Method

In this case, the panel zone is modeled according to the method proposed in this research. In this case, relations (1-19) to (1-23) have been used to model each panel zone.

8.4. Comparison of Pushover Analysis Results

After designing the mentioned frames, pushover analysis was performed up to the maximum capacity of the structure. The comparison of the results of this analysis for the above frames is as follows (Figure 29).

In Figures (30) to (34), the conditions of formation of the first and last plastic joints up to the collapse stage of the structure can be seen in the pushover analysis for each of the above structures.

9. Comparison of Nonlinear Time History Analysis Results

After designing the mentioned frames, nonlinear time history analysis was performed. The earthquake used in this analysis was Imperial Valley-06-Cerro Prieto, which was automatically scaled by ETABS software with a very high risk type 2 earth design spectrum of Iran seismic code. Figures (35) to (39) show the conditions for the first and last plastic joint in the nonlinear time history analysis for each of the above structures.

As mentioned earlier, a model with 4 degrees of freedom, 12 nodes and 12 elements is needed to predict the behavior of the panel zone using the Krawinkler method. But the model presented in this paper, use only 1 spring with trilinear behavior to predict the behavior of the panel zone with good accuracy. Therefore, it can be used by engineers in common structural design software such as SAP2000 and ETABS.

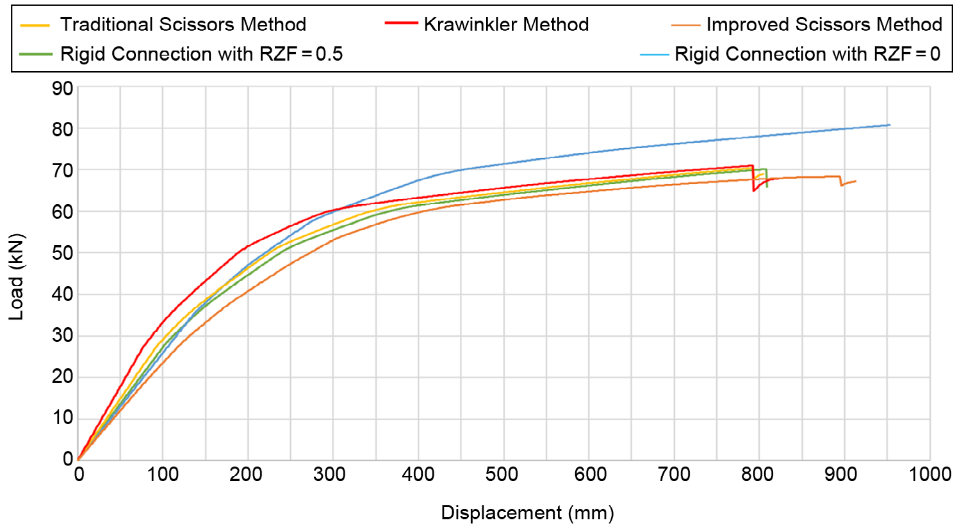

Figure 29. Comparison of pushover analysis results for different panel zone modeling methods.

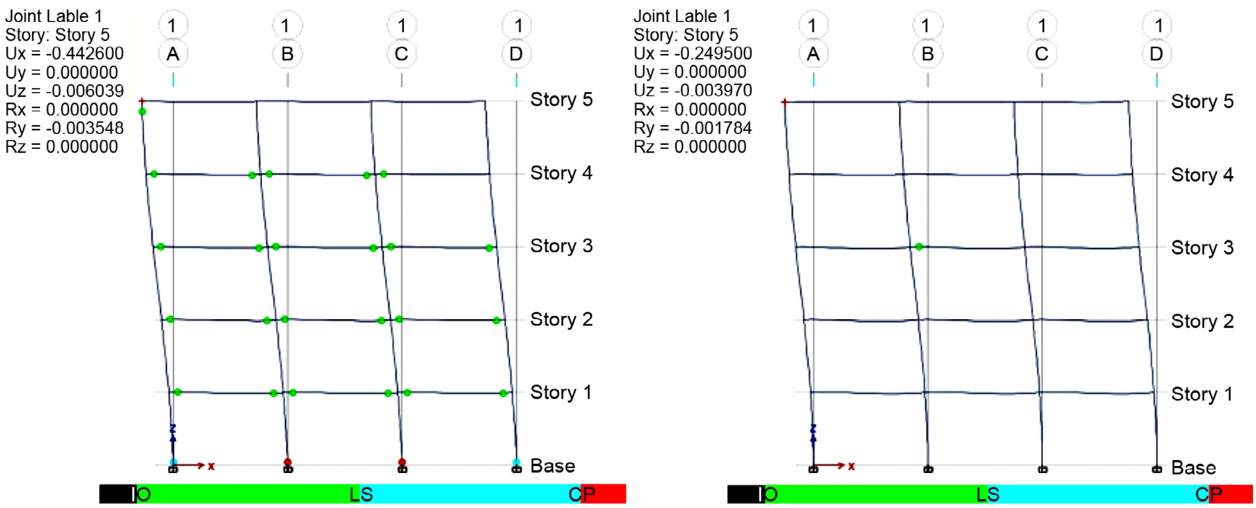

Figure 30. The first and last plastic joint in the structure with rigid panel zone and RZF = 0.

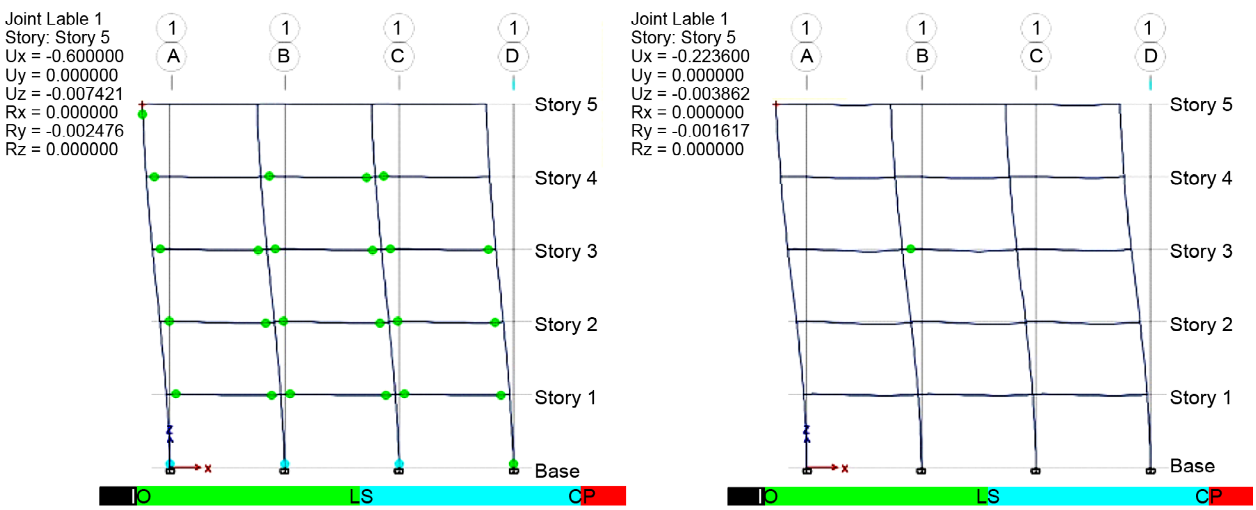

Figure 31. The first and last plastic joint in the structure with rigid panel zone and RZF = 0.5.

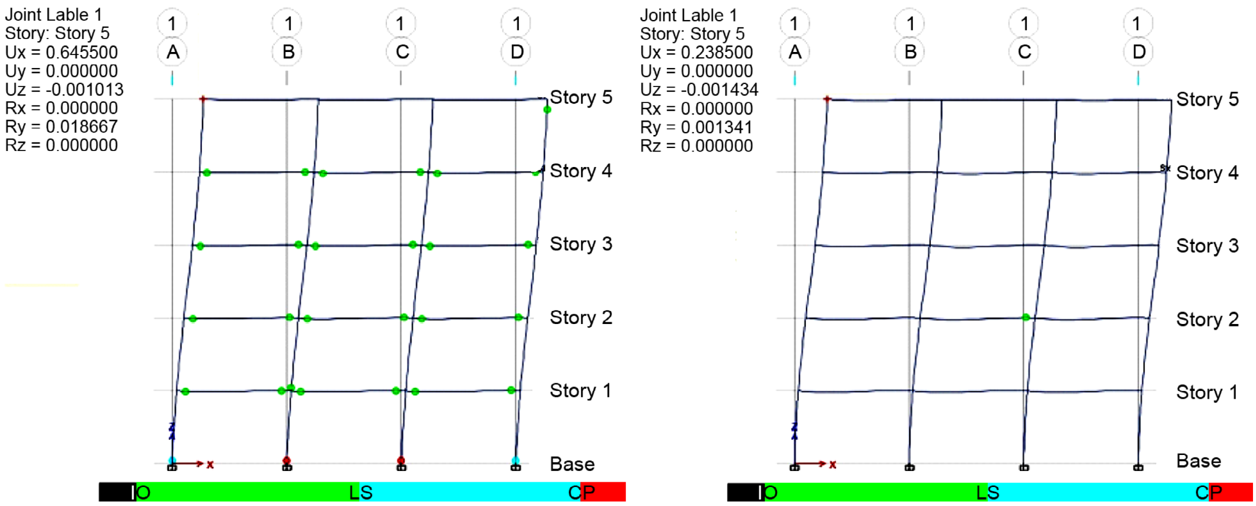

Figure 32. The first and last plastic joint in the structure by modeling the panel zone using the traditional scissors method.

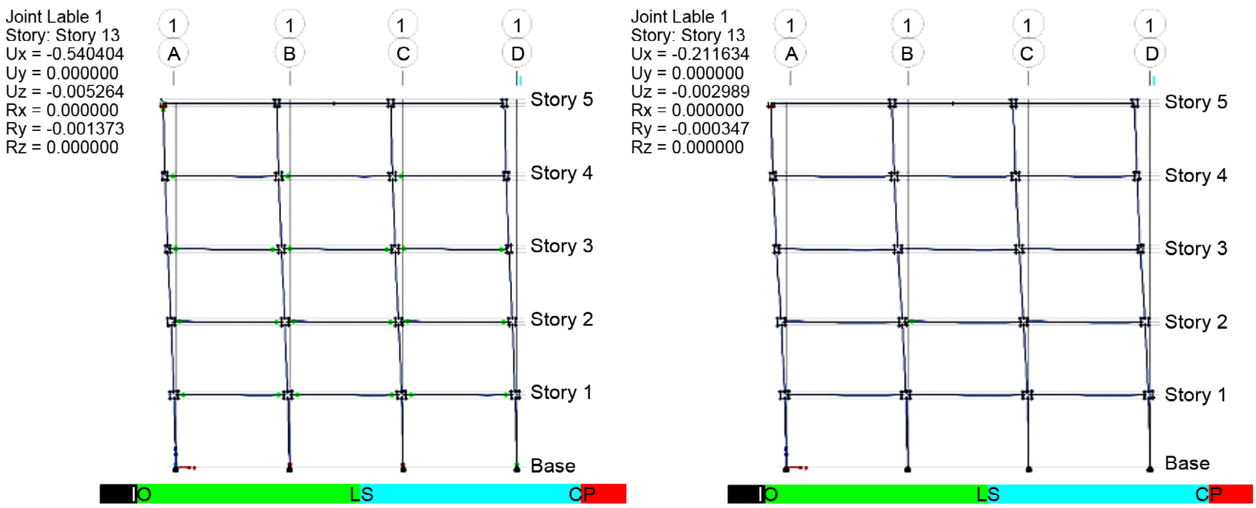

Figure 33. The first and last plastic joint in the structure by modeling the panel zone using the Krawinkler method.

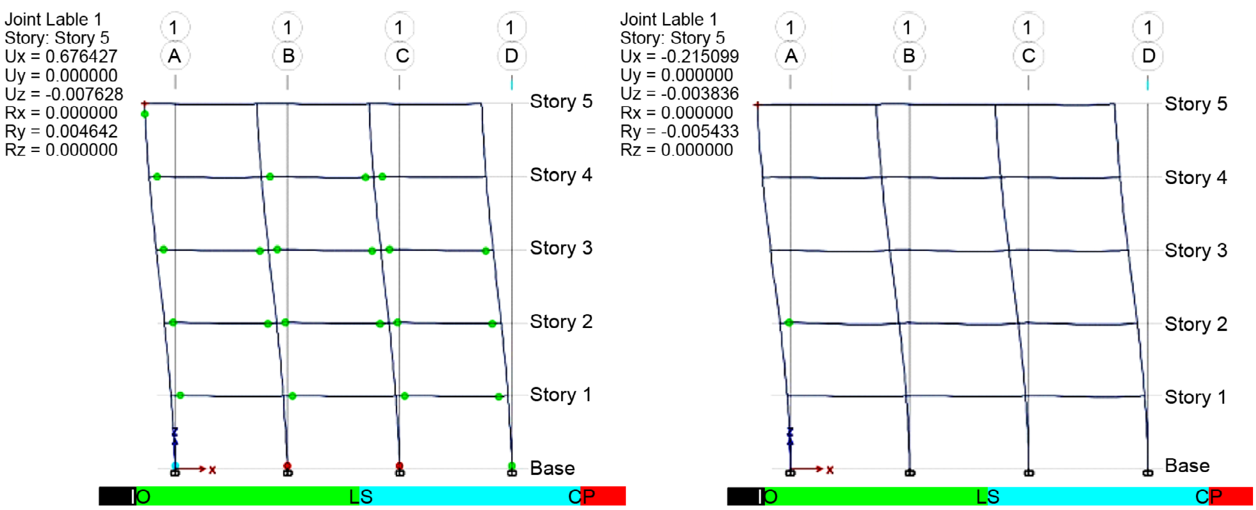

Figure 34. The first and last plastic joint in the structure by modeling the panel zone using the improved scissors method.

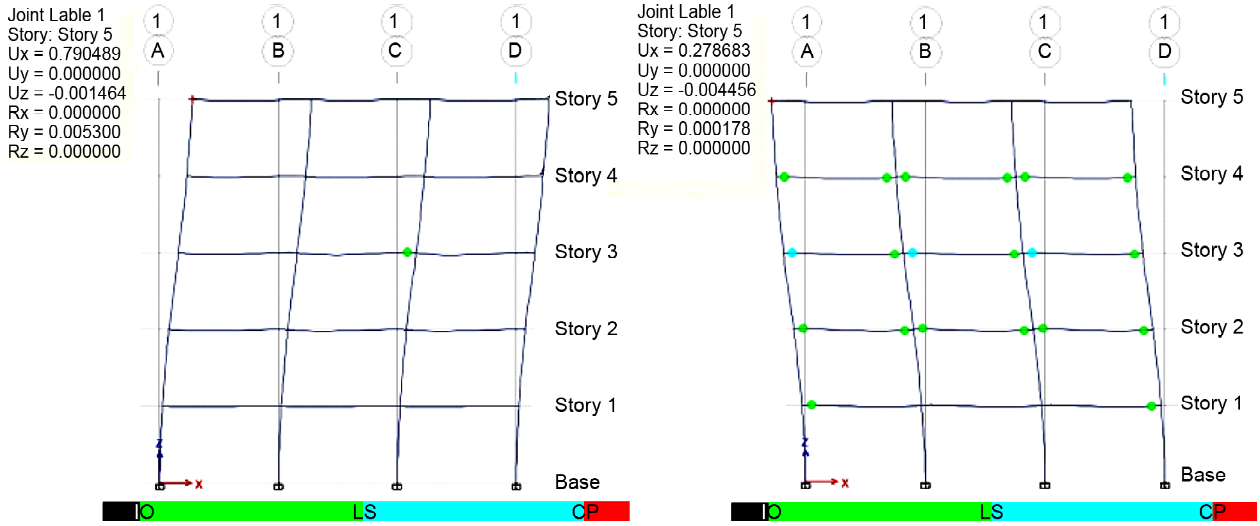

Figure 35. The first and the last plastic joint in the structure with rigid panel zone and RZF = 0.

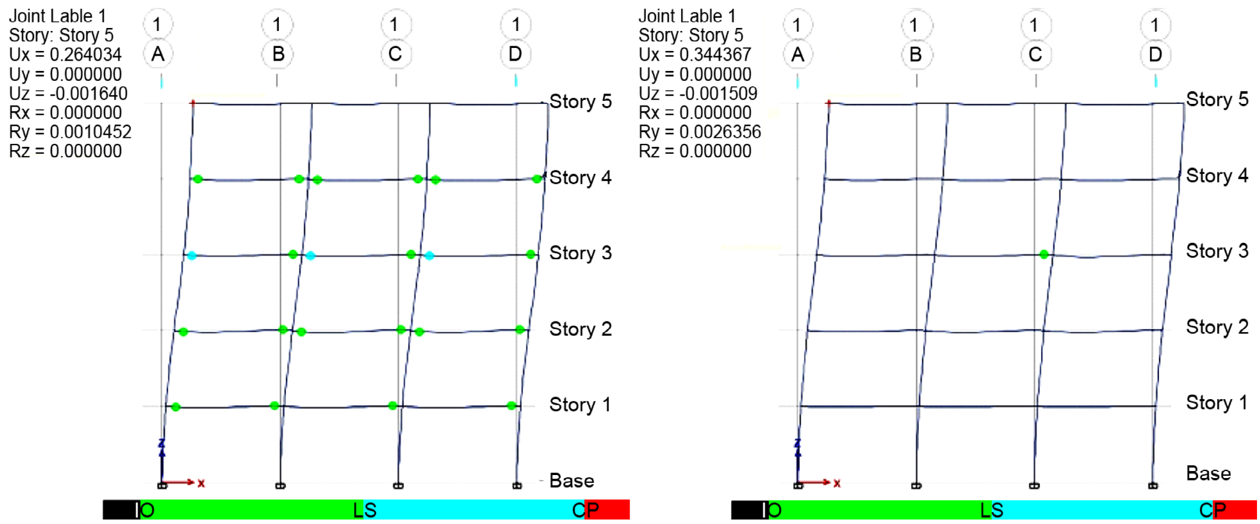

Figure 36. The first and last plastic joint in the structure with rigid panel zone and RZF = 0.5.

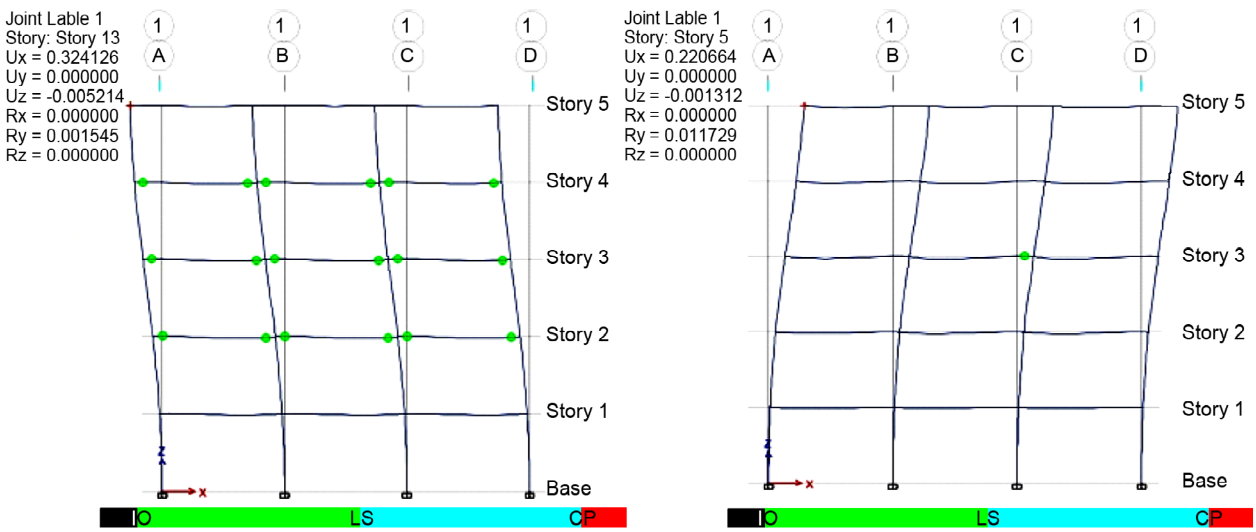

Figure 37. The first and last plastic joint in the structure by modeling the panel zone using the traditional scissors method.

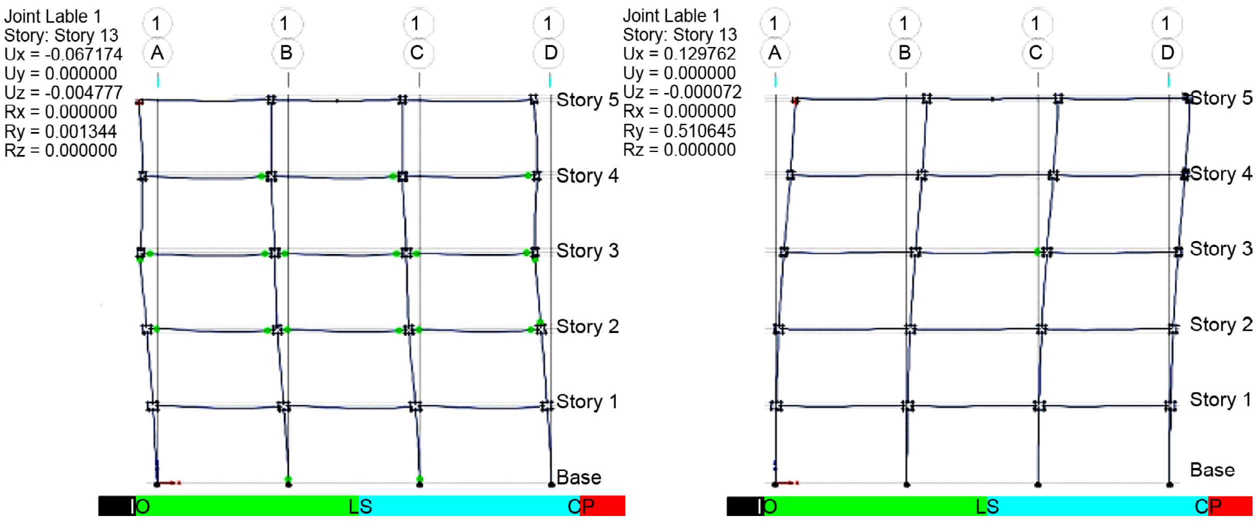

Figure 38. The first and last plastic joint in the structure by modeling the panel zone using the Krawinkler method.

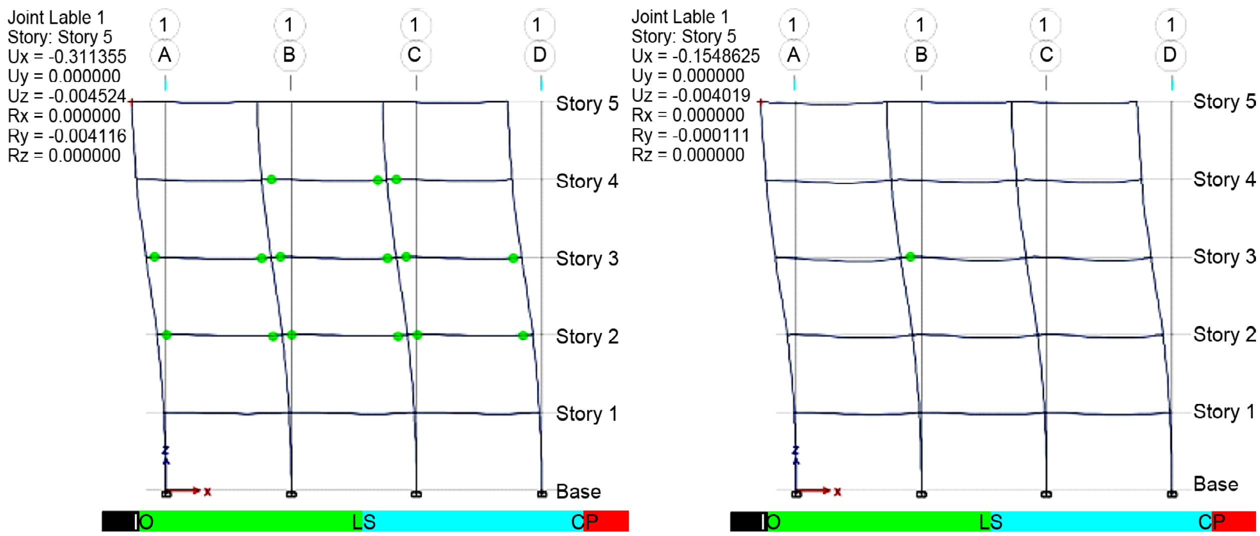

Figure 39. The first and last plastic joint in the structure by modeling the panel zone using the improved scissors method.

9. Conclusion

The improved proposed model has 1 degree of freedom instead of the combination shear and flexure behavior both, which is why it can be used simply in common design software such as SAP and ETABS. In this model, one spring with trilinear behavior is used to model the total behavior of the panel zone. By using this model, the degree of freedoms is reduced by 75% compared to the Krawinkler model, and the load capacity of the panel zone in the nonlinear region is increased by about 58% compared to the scissors model, which is almost equal to the result of an experimental test (approximately error with the test has 8% error conservatively).

Other advantages of the proposed model include:

- High accuracy in predicting panel zone behavior
- High accuracy in modeling panel zone deformations
- High accuracy in estimating panel zone shear rotation
- High speed in calculating connection behavior
- Ease of use due to having a degree of freedom
- Can be used simply in structural design software such as SAP and ETABS
- The independence of the parameters from the structure, and the more dependence on the panel zone profile
- Independence Modeling parameters from structure outside of the panel zone region

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