An Innovative Method for the Seismic Retrofitting of Existing Steel Moment Frame Structures Using Side Plate Technology

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Abstract

The 1994 Northridge earthquake caused widespread damage to steel moment-resisting frames including brittle fractures of beam-to-column welded moment connections. The damage was in the form of brittle failures at the interface of the beam and column members and was detected in multi-story buildings. Improvements to these pre-Northridge existing connections are often required as part of an overall strengthening scheme, particularly when higher performance levels are desired. To improve the behavior of moment resisting connections, the SidePlate connection was designed and began to market as an alternative moment connection. The SidePlate connection technology uses a pair of parallel full-depth side plates to connect the beam to the column, thereby eliminating the traditional welded connection between the end of the beam and the face of the column flange.

While the SidePlate technology has been largely used in the construction of post-Northridge buildings, its application in the retrofitting of existing buildings has been limited to date. In this paper a new generation of SidePlate connections is proposed which is applicable for retrofitting purposes and can improve the performance of existing steel structures. It is believed that the new technology may lead to the best way for retrofitting existing building moment frame systems instead of adding concrete shear walls and/or bracing elements. The proposed detail aims to minimize the disturbances of existing building usage situation and at the same time improves the performance of the system by increasing the rigidity of panel zone region and the stiffness of the beam segment between the column face and plastic hinge location.

A case study including two multi-story steel moment frame structures was conducted to show that the proposed technology can largely upgrade the performance level of existing structures. Nonlinear analysis was performed to evaluate the vulnerability of the existing structures first, then the proposed detail was implemented to the connections, and new performance levels were obtained by means of nonlinear static analysis. The results showed that the plastic hinges are always formed in beams instead of the columns. It has also been shown that the application of the proposed details reduces the target displacement and improves the performance level of the structures. It can be concluded that depending on the performance level of existing buildings, the proposed detail might be considered as an efficient and optimized way of retrofitting those structures.

Keywords: SidePlate connections, nonlinear static analysis, performance levels, retrofitting

1- Introduction

The seismic design philosophy of moment resistance frames is based on the flexible behavior under earthquake forces and safe performance. Nevertheless, unexpected damages to steel moment-resisting frame connections occurred in the 1994 Northridge earthquake. The majority of damage occurred at the widely used welded unreinforced flange-bolted web (WUF-B) "traditional" connection, known as the "pre-Northridge" moment connection. After the 1994 Northridge earthquake, researchers carried out various studies to find the reasons for the unacceptable behavior of the connection. Investigations showed that the most common type of damage in the majority of steel buildings was the premature brittle fractures of groove weld and base metal due to the triaxial stress state in the connection.

After the damage evaluation in moment resistance frames, various practical solutions for each type of damages were suggested. One of these proprietary designs was the SidePlate moment connection system, invented by David L. Houghton. SidePlate steel frame connection technology provides beam-to-column moment connections for use in steel special moment frame (SMF) and steel intermediate moment frame (IMF) systems.

The SidePlate moment connection system is constructed of all welded fabrication, and features a physical separation, or "gap," between the face of the column flange and the end of the beam, by means of parallel full-depth side plates which sandwich and connect the beam(s) and the column together. Top and bottom beam flange cover plates are used, as necessary, to bridge any difference between flange widths of the beam(s) and of the column. Vertical and horizontal shear plates, as applicable, are provided at the beam and column webs, respectively.

Moment transfer from the beam to the side plates, and from the side plates to the column is achieved through the fillet welds. The side plates are designed with adequate strength and stiffness to force all significant plastic behavior of the connection system into the beam, in the form of flange and web local buckling centered at a distance of approximately 1/3 the depth of the beam away from the side plates (FEMA 350 [3]). SidePlate connection has no limit on the size of the connection members or the type of column – it can be a wide flange shape or built up box column or a cruciform column for biaxial applications (Davis [4]).

More than 20 full scale tests have been conducted on the SidePlate connection to prove that SidePlate connection is a strong solution for the connection of beam and column in special moment frame applications. Houghton [3] reported the result of experimental testing on three uniaxial test specimens. These specimens consisted of a W36×150 beam, which was connected with full-depth side plates to a W14×426 column. Also, another specimen that included of W36×170 beams was tested. In this specimen, beams were connected to a built-up cruciform column, fabricated using W36×230 sections in each principal direction, to form a three-sided connection. Study results were generally qualitative and implied that the desired behavior, intended by the connection design procedure, could be achieved easily.

While the SidePlate technology has been largely used in the construction of post-Northridge buildings, its application in the retrofitting of existing buildings has been limited to date. In this paper a new generation of SidePlate connections is proposed which is applicable for retrofitting purposes and can improve the performance of existing steel structures. In this study, two steel moment frame structures were selected to assess the vulnerability of the existing buildings. The Basic Safety Objective (BSO) is used for the seismic assessment of the buildings. If the structure under consideration does not achieve the required performance level, it has been retrofitted by utilizing the SidePlate solution. The inelastic structural response has been expressed in terms of plastic hinge status, story drifts derived by means of nonlinear static analyses.

2- SidePlate Connection Modeling

ETABS 2013 [6] was used for modeling and analyzing the structures. An illustration of the numerical model established in ETABS to represent the connection is depicted in Fig. 1.

As illustrated in the figure, the lumped plasticity method is used for representing the nonlinear behavior of beams utilizing plastic hinges whose location is determined according to experimental observations. The rotational behavior of plastic hinges is modeled using a zero-length spring, which connects two elastic beam segments to each other. Elastic beams are defined using actual section properties measured from the test specimen [7]. As observed in experiment [7], the location of the plastic hinge in a SidePlate connection is about one-third the beam height beyond the end of the side plates (Fig. 2).

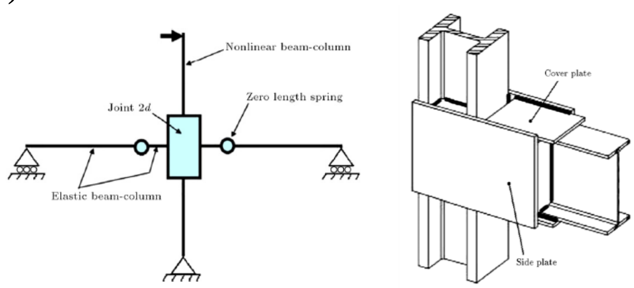


Fig. 1. Assembled model in ETABS software

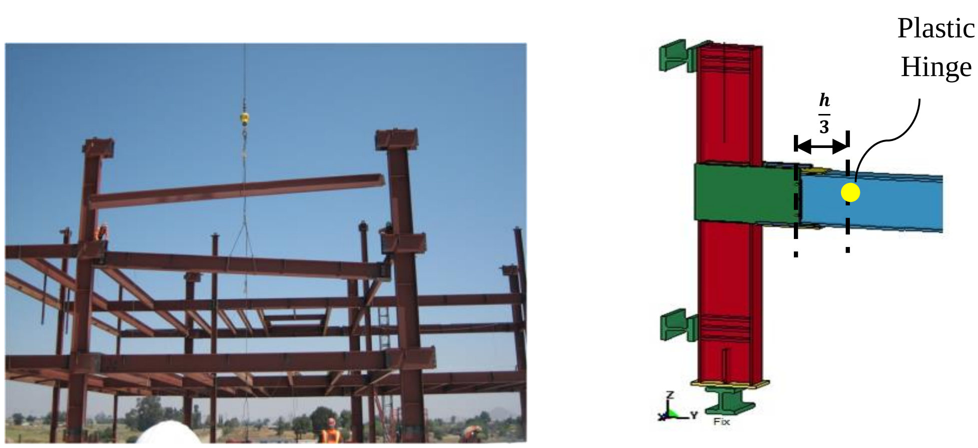


Fig.2. Location of plastic hinge in the SidePlate connection

Effective use and correct implementation of the inherent SidePlate connection stiffness on the global lateral frame of a structure is done as follows for ETABS 2013:

2-1- Method 1

The ETABS Built-in SidePlate Feature automatically creates a non-prismatic beam where each fixed beam end represents the appropriate SidePlate connection stiffness properties from the column face towards the beam centerline as follows:

• A section extends from column face to 77% of the nominal beam depth. The side plate section, which consists of the physical side plates {A}, cover plates {B} and beam, has an approximate moment of inertia (3) times that of the beam (Fig. 3). In this study, method 2 was used for modeling of the structures.

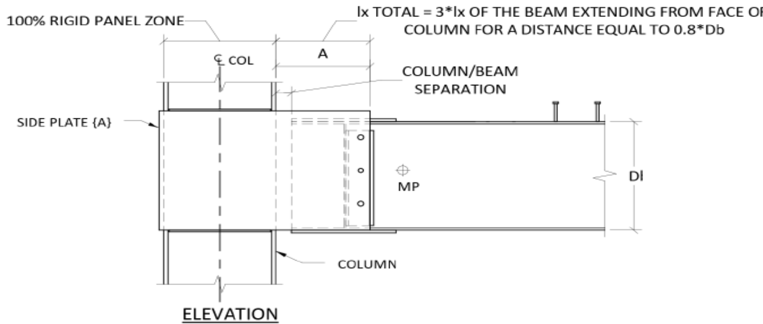


Fig. 3. Full length beam to side plate detail

2-2- Method 2

Using Non-Prismatic Beam Sections

Another way of implementing SidePlate connection properties is to use non-prismatic beam sections. This method can also be used for seismic or wind applications to help improve steel stress ratios if the current design is at or above the allowable code values due to present limitations with the built-in SidePlate feature.

3- Design and Analysis of Structures

3-1- Design of buildings

To investigate the performance of steel moment frames with SidePlate connections, a 2 and 6 story moment frame are designed, with reference to pertinent provisions AISC 360-05 [8] and AISC 341-05 [9]. Design loads are computed for a standard office building. The buildings are assumed to be located in Nashville, California. Figs. 4 and 5 show the two studied structures in this paper.

The first building consists of a 2 story shopping center which has 30 ft-long bays. The first and second story height are 17.5 ft and 19.5 ft. Nominal yield strength equal to 50 ksi was used for columns and girders. The second building is a 6-story residential building which has 32ft-long bays. The existing lateral force resisting system consists of steel moment frames. The moment frames are positioned at each of the two sides of the structure. The Site Class is D with very low potential for liquefaction. The seismic parameters S_s and S_1 per ASCE 7[10] are 1.85 and 0.84, respectively. The value of 0.40g was assumed for the peak ground acceleration at the bedrock.

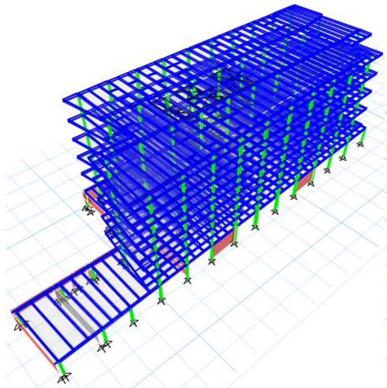


Fig. 4. Residential building

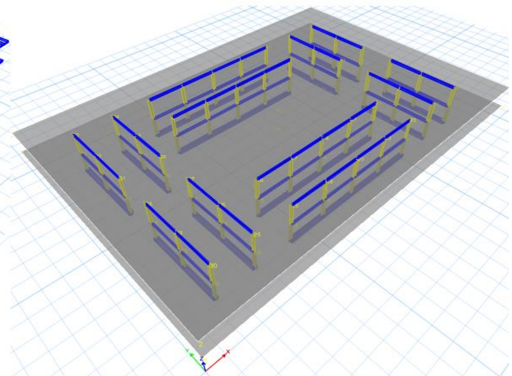


Fig. 5. Shopping center

4- Nonlinear Static Analysis

Static pushover analysis is becoming a popular tool for seismic performance evaluation of existing and new structures. The expectation is that the pushover analysis will provide adequate information on seismic demands imposed by the design ground motion on the structural system and its components. The pushover analysis of a structure is a static non-linear analysis under permanent vertical loads and gradually increasing lateral loads. The equivalent static lateral loads approximately represent earthquake induced forces. A plot of the total base shear versus top displacement in a structure is obtained by this analysis that would indicate any premature failure or weaknesses (Fig. 6). The analysis is carried out up to failure, thus it enables determination of collapse load and ductility capacity. On one building frame, plastic rotation is monitored, and lateral inelastic forces versus displacement response for the complete structure is analytically computed. This type of analysis enables weaknesses in the structure to be identified. The decision to retrofit can be taken in such studies.

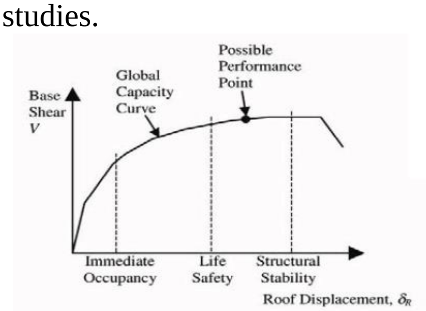


Fig. 6. Capacity curve of structure

According to the ASCE14-13 [11], for linear and nonlinear procedures, the following actions caused by gravity loads, Q_G, shall be considered for combination with actions caused by seismic forces. Where the effects or actions of gravity loads and seismic forces are additive, the action caused by gravity loads, Q_G , shall be obtained in accordance with Eq.(1):

$$Q_G = 1.1(Q_D + Q_L + Q_S) \tag{1}$$

Where the effects or actions of gravity loads and seismic forces are counteracting, the action caused by gravity loads, Q_G, shall be obtained in accordance with Eq.(2):

$$Q_G = 0.9Q_D \tag{2}$$

Several lateral load patterns have been suggested. They are: (1) inverted triangle distribution (modal pattern); (2) uniform distribution; (3) load distribution based on linear elastic dynamic analysis or response spectrum analysis of the building; (4) the adaptive distribution, which is varied as the inter story resistance changes in each load step; (5) distribution proportional to the product of the mass and fundamental mode shape, which is used initially until the first yielding takes place. Then the lateral forces are determined based on the product of the current floor displacement and mass at each step; (6) a distribution based on mode shapes derived from secant stiffness at each load step; The last three distributions are adaptive patterns, which try to establish equivalent lateral load distribution based on a certain theoretical basis. However, their superiority over the simple fixed load patterns has not been demonstrated. Due to the fact that the lateral force profiles in static pushover analyses will influence the structural response, two different load patterns have been utilized to represent the distribution of inertia forces imposed on the building. The first shape is triangular according to the first mode of structure vibration and second load pattern is uniform load pattern.

ASCE41-13 [11] document has developed the modeling procedures, acceptance criteria and analysis procedures for pushover analysis. This document defines force-deformation criteria for hinges used in pushover analysis. As shown in Fig. 7, five points labeled A, B, C, D, and E are used to define the force-deflection behavior of the hinge and three points labeled IO, LS and CP are used to define the acceptance criteria for the hinge. (IO, LS and CP stand for Immediate Occupancy, Life Safety and Collapse Prevention respectively.) The values assigned to each of these points vary depending on the type of member as well as many other parameters defined in ASCE41-13[11] document.

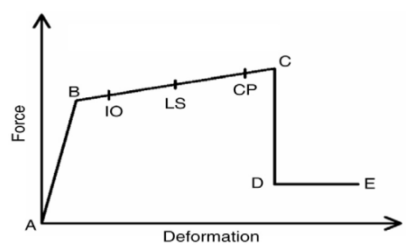


Fig. 7. Force–deformation relationship of a typical plastic hinge

As stated before, two existing steel structures were selected for retrofitting using SidePlate technology. Therefore, pushover analysis was done under two load patterns. In this study, the performance criteria for immediate occupancy (IO), life safety (LS) and collapse prevention (CP) were assigned to beams and columns according to ASCE 41-13. PMM hinges for columns and M3 hinges for beams were specified in this study. Plastic hinges are modeled at both ends of the beams and columns. The objective is to retrofit the structures if the performance level didn't satisfy the BSO level. The plastic hinges of SidePlate connections are located as shown in Fig. 8. Based upon past and current testing of the SidePlate connection system, the centerline of hinge development within the beam is located at approximately 1/3 of the beam depth ($0.33*d_{bm}$) from the ends of the side plates (Fig. 2). Similarly, the modeled hinge shall be placed at the same location. For this study, the extension of the side plates is set by design as 77% of the nominal depth of the beam ($0.77*d_{bm}$) from the face of the column (rounded to nearest whole inch), and the center of hinge is approximately 1/3 of the beam depth ($0.33*d_{bm}$) from the end of the side plates; the hinge shall be located at L_h from the centerline of column, where $L_h = ½d_{col} + 1.1d_{bm}$, ($0.77d_{bm} + 0.33d_{bm} = 1.1d_{bm}$).

5- Seismic Vulnerability Assessment of Structures

In this study, a seismic safety assessment was used for the evaluation of the existing buildings. The considered structures were retrofitted using the SidePlate technology when they did not satisfy BSO performance level. Nonlinear static analysis was conducted to evaluate the seismic vulnerability of the chosen structures. The results of pushover analyses are presented below.

6- Results of Pushover Analyses

Pushover analysis of structures was performed in the X and Y directions as shown in Figs. 8 and 9. Two lateral load pattern were used in the analysis. 1) The first mode shape of the structures in the X and Y directions; 2) uniform mode shape over the height. The monitored point of structures are shown in Figs. 8 and 9.

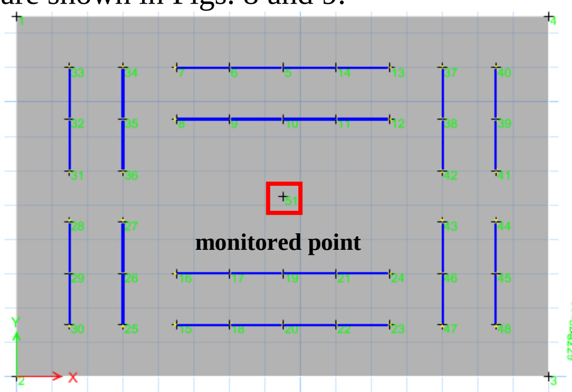


Fig. 8. Monitored point for Shopping center

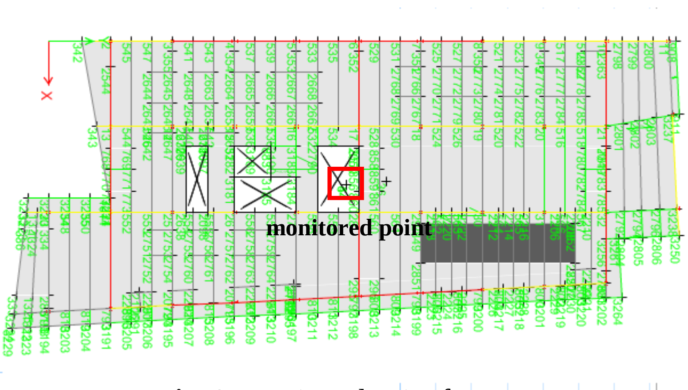


Fig. 9. Monitored point for Residential building

6.1- Pushover Curves

The pushover curves obtained for the structures are plotted in Figs. 10 to 13 (maximum top displacement at the center of mass (CM) versus the corresponding base shear).

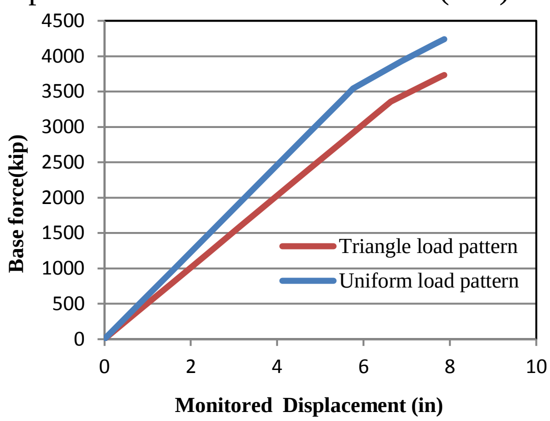


Fig. 10. Pushover curves for shopping center in the X direction

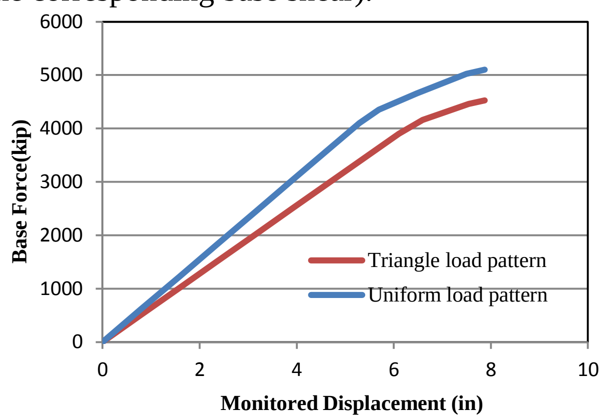


Fig. 11. Pushover curves for shopping center in the Y direction

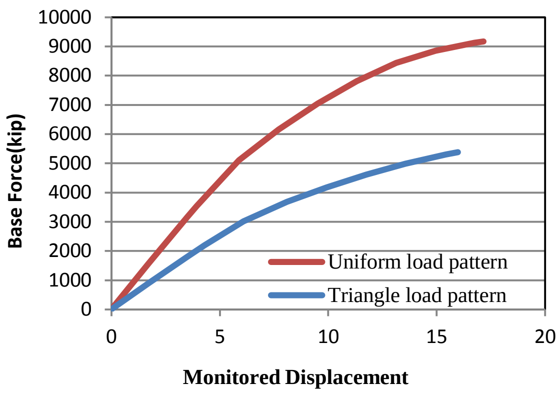


Fig. 12. Pushover curves for residential building in the X direction

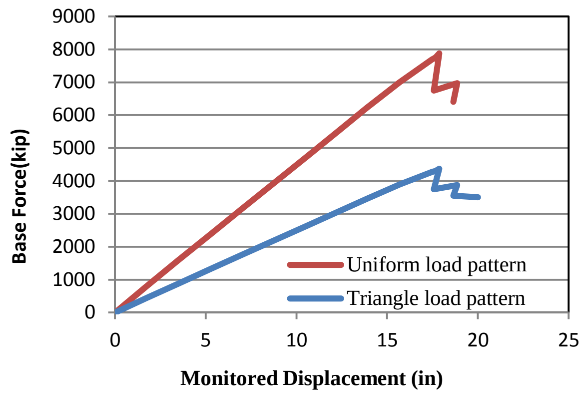


Fig. 13. Pushover curves for residential building in the Y direction

The plastic hinge distribution for the steel structures resulted from applying a lateral load to the building with both lateral load patterns (uniform (ULP) and triangle (TLP)) and the gravity load pattern of 0.9dead and 1.1(Q_D +Q_L +Q_S) at the performance point are shown in Figs. 14 through 17. The results show that all the plastic hinges are in the phase of LS-CP and some of beams and columns are in the phase of beyond E meaning performance level didn't satisfy the BSO objective; therefore, structures need to be retrofitted.

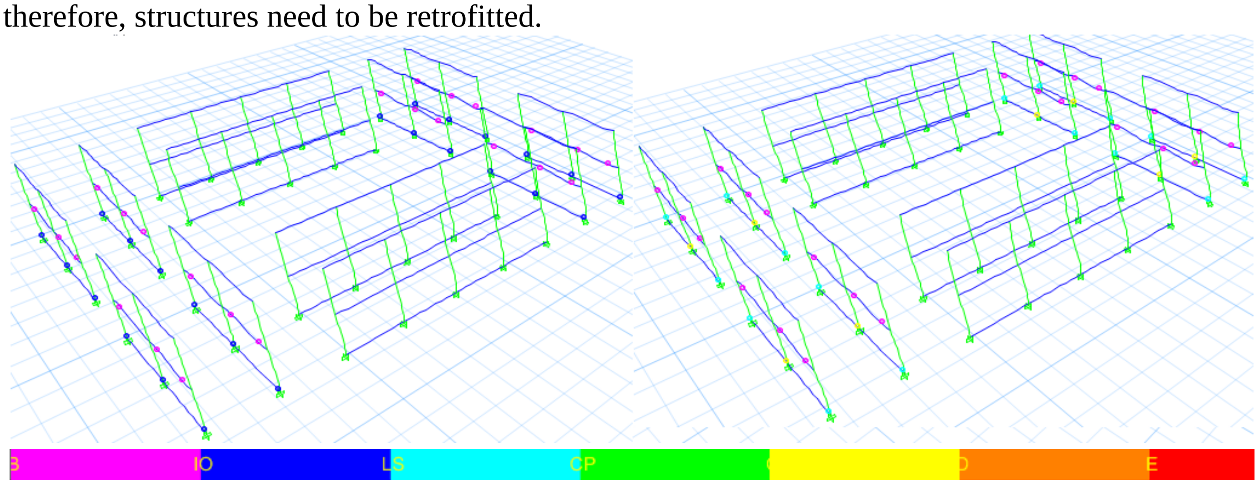
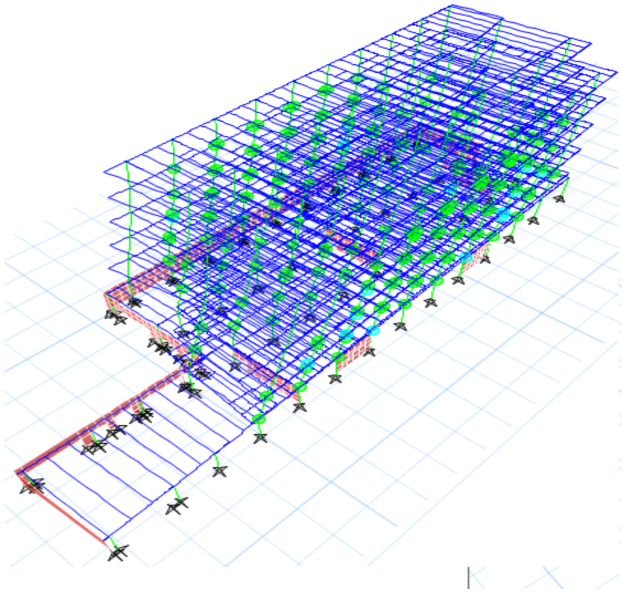


Fig. 14. Plastic hinge distribution for original shopping center under triangular load pattern in Y direction

Fig. 15. Plastic hinge distribution for original shopping center under uniform load pattern in Y direction



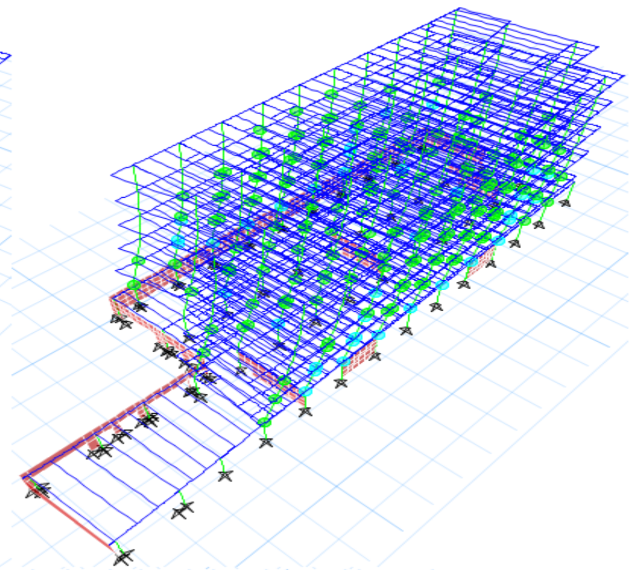


Fig. 16. Plastic hinge distribution for original residential building under triangular load pattern in Y direction

Fig. 17. Plastic hinge distribution for original residential building under uniform load pattern in Y direction

7- Retrofitting of the Buildings

The nonlinear static analyses showed the buildings' performance is in collapse prevention whereas, the expected performance for this level of earthquake must be life safety. Due to this reason, seismic retrofitting measure should be taken to increase the performance to target levels in terms of global strength and stiffness. In order to enhance the buildings' performances, SidePlate connections have been used for retrofitting of steel the structures. This method adds significant strength and stiffness to steel structures, in addition, this method leads to the best way for retrofitting of existing buildings instead of adding concrete shear walls and/or bracing elements and many conditions will be compatible with the original architectural plan of building.

7-1- Nonlinear Static Method (Pushover Analysis)

Nonlinear static analysis is used to understand the effect of the SidePlate connection in the behavior of structures. Inelastic static analysis, or pushover analysis, has been the preferred method for seismic performance evaluation due to its simplicity. The pushover analyses were performed by applying monotonically increasing lateral loads such as triangular and uniform load pattern to a structure. Nonlinear behavior of SidePlate connections defined using the experimental results composed by SidePlate Systems, Inc. was used in adjusting the acceptance criteria and the five points labeled A, B, C, D, and E define in the force–deformation behavior of the SidePlate connection. The moment-rotation ratio and acceptance criteria of beams with SidePlate are shown in Table 1 and Table 2. Also, a generalized force-displacement characteristics of hinge elements for SidePlate is shown in Fig. 18. It needs to be noted that the acceptance criteria used in this study need to be updated based on proposed retrofitting solution in real practical projects and the given numbers are only useful for explaining the design philosophy.

W36x150				
Total Hinge Rotation (rad)	Plastic Hinge Rotation (rad)	Moment (in-kip)	$Ø/Ø_y$	M/M_p
0.0000	0.0000	0	0.00	0.00
0.0088	0.0000	25200	0.88	0.88
0.0100	0.0000	28750	1.00	1.00
0.0175	0.0075	31875	1.75	1.11
0.0225	0.0125	32500	2.25	1.13
0.0310	0.0210	31375	3.10	1.09
0.0490	0.0390	25000	4.90	0.87
0.0490	0.0390	3750	4.90	0.13
0.0550	0.0450	3750	5.50	0.13

Table1. Test - moment rotation ratios

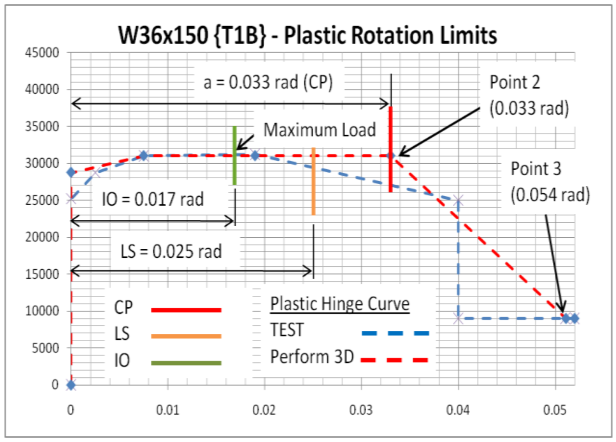


Fig. 18. SidePlate FRAME®W36x150 beam force-deformation modeling and acceptance criteria per ASCE 41-06 Supplement No. 1

Table 2. Acceptance criteria of beams with SidePlate connection

Section	h	t_w	b_f	t_f	h/t_w	$b_f/2t_f$	A	B	C	IO	LS	CP
W8x10	20.04	0.432	10.00	0.521	46.39	9.60	4	6	0.2	0.25	2	3
W10x45	25.65	0.889	20.37	1.575	28.85	6.46	9	11	0.6	1	6	8
W18x35	44.95	0.762	15.24	1.08	59.00	7.05	4	6	0.2	0.25	2	3
W18x50	45.72	0.902	19.05	1.448	50.68	6.57	4	6	0.2	0.25	2	3
W21x50	52.83	0.965	16.58	1.359	54.75	6.10	4	6	0.2	0.25	2	3
W21x57	53.59	1.029	16.66	1.651	52.08	5.04	4	6	0.2	0.25	2	3
W21x62	53.34	1.016	20.93	1.562	52.50	6.69	4	6	0.2	0.25	2	3
W24x55	59.94	1.003	17.80	1.283	59.76	6.93	4	6	0.2	0.25	2	3
W24x62	60.19	1.092	17.88	1.499	55.12	5.96	4	6	0.2	0.25	2	3
W24x76	60.70	1.118	22.83	1.727	54.29	6.61	4	6	0.2	0.25	2	3
W27x84	67.81	1.168	25.40	1.626	58.06	7.81	4	6	0.2	0.25	2	3
W27x94	68.32	1.245	25.40	1.892	54.88	6.71	4	6	0.2	0.25	2	3
W24x76	60.70	1.118	22.83	1.727	54.29	6.61	1	1.5	0.2	0.25	0.5	0.8
W24x103	62.23	1.397	22.86	2.489	44.54	4.59	1	1.5	0.2	0.25	0.5	0.8
W24x131	62.23	1.537	32.76	2.438	40.48	6.71	1	1.5	0.2	0.25	0.5	0.8
W24x146	62.73	1.651	32.76	2.769	38.00	5.91	1	1.5	0.2	0.25	0.5	0.8

7-2- Results of Pushover Analysis in Retrofitted Structures

The capacity curves of the structures from the nonlinear static analysis before and after retrofitting with the SidePlate connection are shown in Figs. 19 through 22. It can be seen that SidePlate connections increase the stiffness and strength and simultaneously and decrease the story drift.

The plastic hinge distribution for the steel moment resistance frame that resulted from applying a lateral load to the building with both lateral load patterns (uniform (ULP) and triangle (TLP)) and the gravity load pattern of 0.9dead at the different performance points are shown in Figs. 23 through 26. From the pushover analysis, it was found that the performance level of the retrofitted structures is acceptable because the yielding in beams and columns occur LS level; the amount of the damage to the building will be limited.

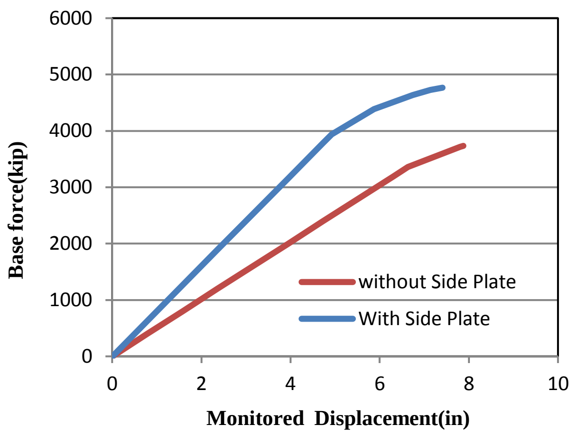


Fig. 19. Comparison of capacity curves for original and retrofitted shopping center in X direction under triangular load pattern

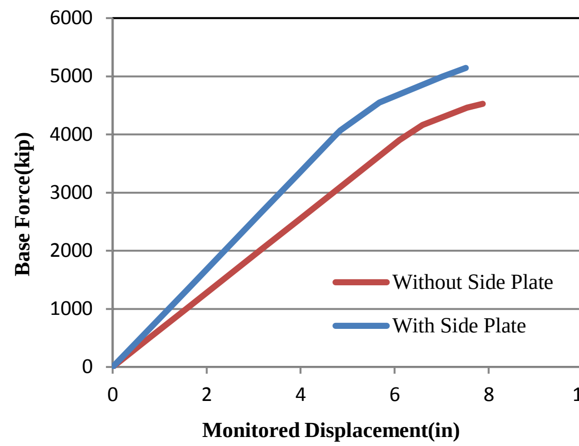


Fig. 20. Comparison of capacity curves for original and retrofitted shopping center in Y direction under triangular load pattern

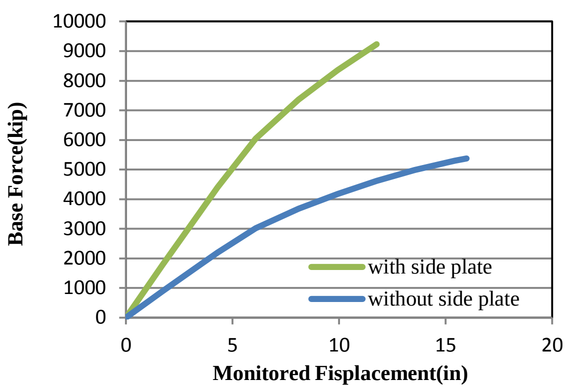


Fig. 21. Comparison of capacity curves for original and retrofitted residential building in X direction under triangular load pattern

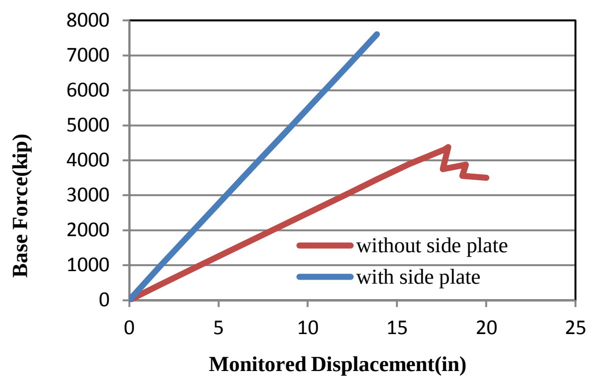
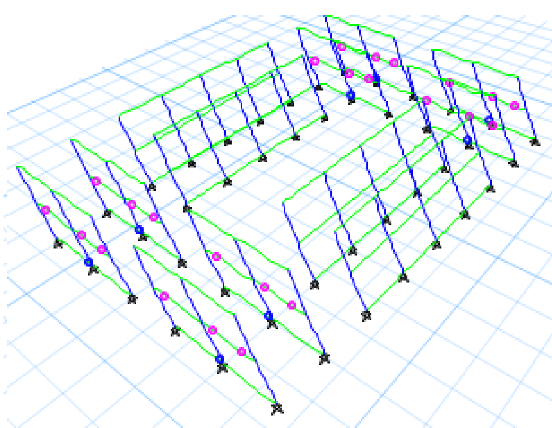


Fig. 22. Comparison of capacity curves for original and retrofitted residential building in Y direction under triangular load pattern



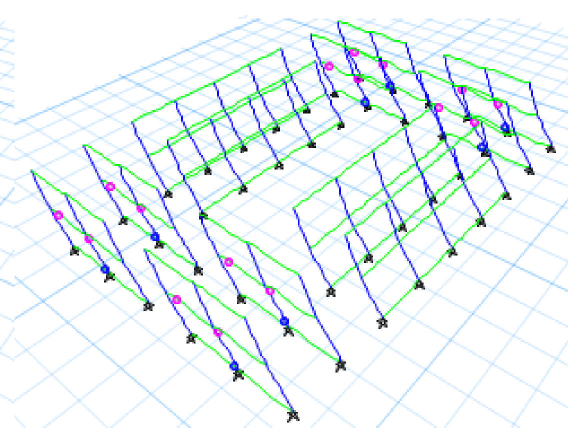
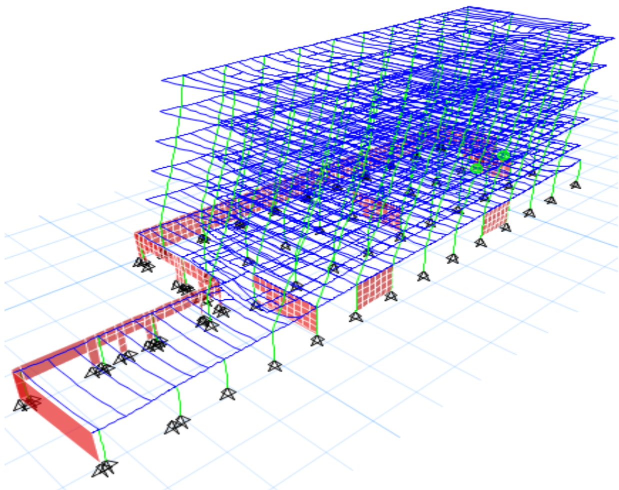


Fig. 23. Plastic hinge distribution for retrofitted shopping center under triangular load pattern in X direction

Fig. 24. Plastic hinge distribution for retrofitted shopping center under uniform load pattern in X direction



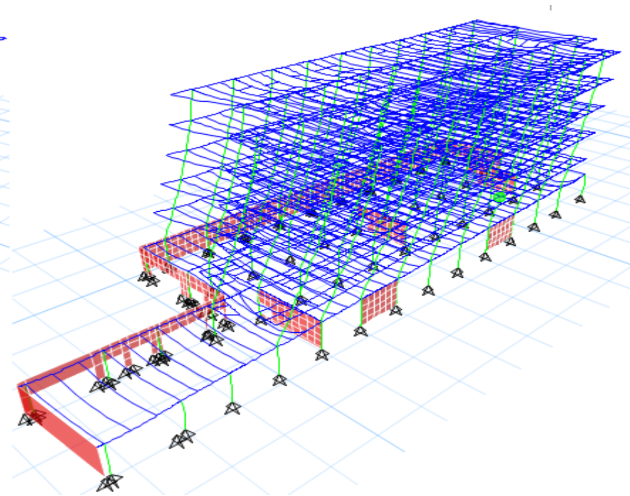


Fig. 25. Plastic hinge distribution for retrofitted residential building under triangular load pattern in the X direction

Fig. 26. Plastic hinge distribution for retrofitted residential building under uniform load pattern in the X direction

8- Finite Element Analysis

Nonlinear finite element analysis (FEA) with 3-dimensional solid brick elements was used to analyze the SidePlate retrofitted connection region. Fig. 27 shows a model of an interior moment connection where the beam-to-column assembly were cut at the points of inflection which occurs at column mid height and beam mid spans. The column is then loaded cyclically at cycles at 4% story drift at the top.

Fig. 28 and Fig. 29 show the details of the connection. From the details, the process of reinforcing an existing WUF-B connection, which includes field welding and bolting, can be done with minimal damage to the building. The concrete slab above can be left intact since all the welding will be under the slab. The SidePlate retrofit solution presents an economical solution to reinforcing existing structures and meeting modern building code requirement for a safe structure.

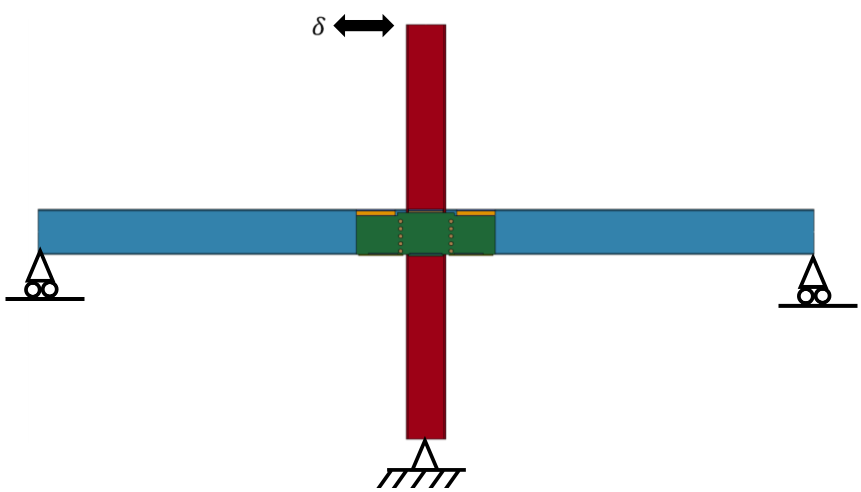


Fig. 27. Side elevation

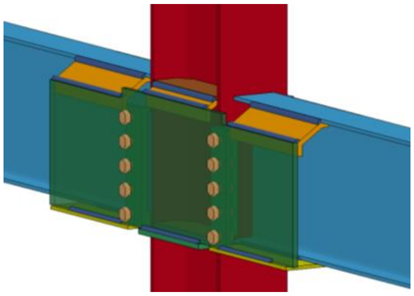


Fig. 28. 3-D isometric view

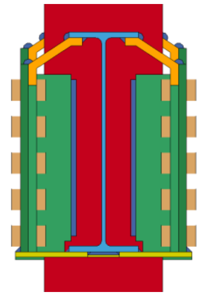


Fig. 29. Front elevation

Fig. 30 shows the average triaxial stress distribution in the connection as the column is loaded laterally at 4% drift. Triaxial stress is a well-known parameter that causes early fracture. Many of the pre-Northridge moment frame connections suffered from this phenomenon. When comparing a WUF-B connection to a SidePlate retrofitted solution, it can be shown that triaxial stress in the connection is significantly reduced, which will significantly reduce the change of early fracture. Fig. 31 shows that plasticity as a form of energy dissipation is pushed into the beam. The plastic hinge formation occurred in the beam, which aligns with modern structural design principles. In addition to significantly lowering the chance of early fracture, Fig. 32 also shows that with the SidePlate retrofit solution, the connection is stiffer and will dissipate more energy as the structure goes under high seismic loading, which is likely to give the connection the required stiffness to meet modern building code requirements.

a) WUF-B

b) SidePlate Retrofit Solution

Fig. 30. Average triaxial stress contour plot at 4% drift ($\sigma = -50\ ksi$)

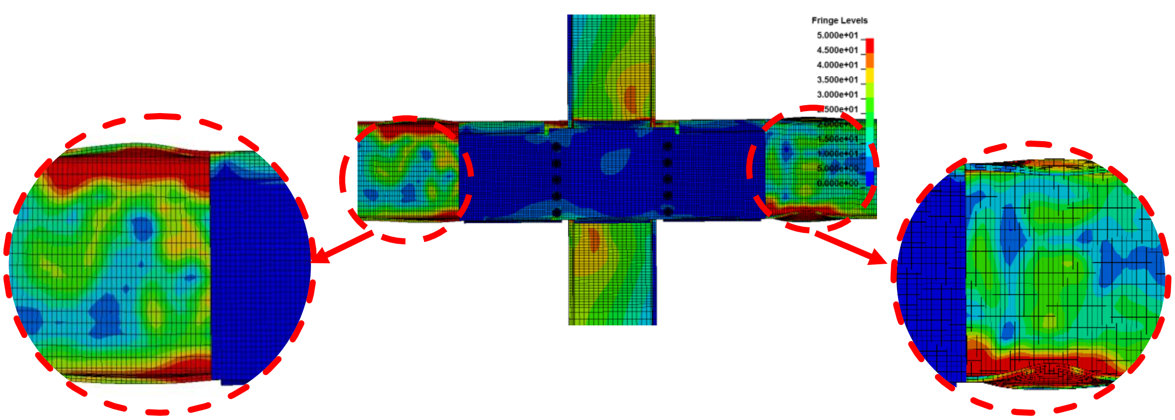


Fig. 31. Beam plastic hinge formation (von Mises Stress, $\sigma_y = 50\ ksi$)

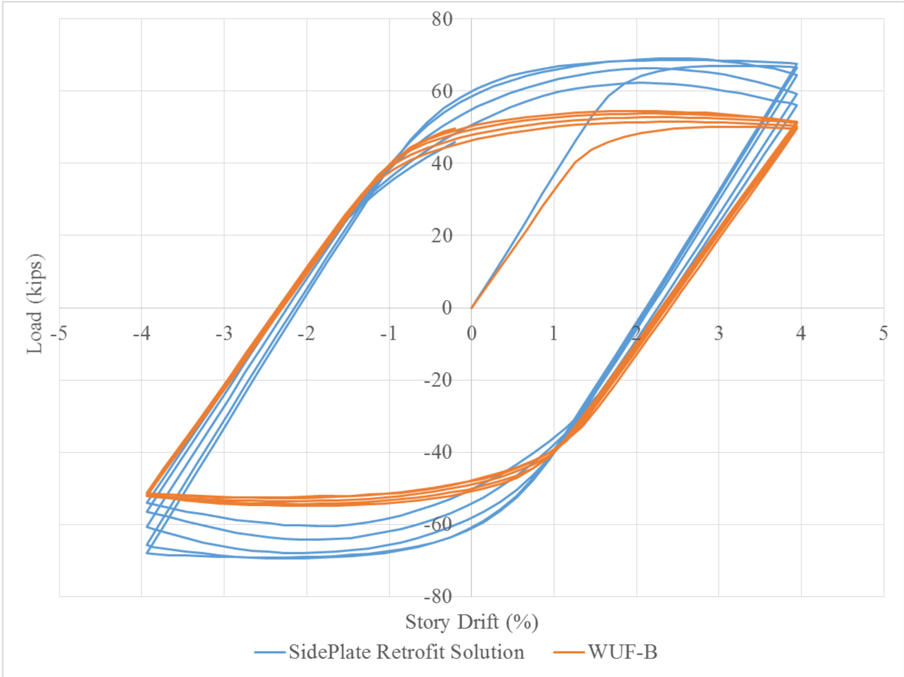


Fig. 32. Cyclic load response of WUF-B vs SidePlate retrofitted solution

9- Conclusion

In this study, two steel moment frame buildings were assessed in terms of seismic performance through the application of nonlinear static analysis. Analysis results showed that the structures did not meet life safety performance level; therefore, they should be retrofitted. SidePlate connections were used for retrofitting of steel structures. Comparison of results obtained from nonlinear static analyses confirms that the load bearing capacity and stiffness of structures is increased significantly by the SidePlate connections; in addition, this pattern of strengthening also prevents from undesirable weak column-strong beam mechanism by formation of plastic hinges in beams instead of columns. In short, it can be concluded that the proposed upgrading method is an applicable cost-saving method which considers all possible deficiencies, and improves the seismic behavior of typical steel buildings significantly.

10- Disclaimer

10-1- It needs to be noted that the acceptance criteria and other parameters used in section 7-1 need to be revised based on the proposed retrofitting solution in real practical situations. The primary objective of this paper is to show how the application of the SidePlate technology can improve the performance level of structures. Extra studies need to be performed for deriving the acceptance criteria and other parameters in practical situations.

10-2- The two example buildings in this study have purely been used to explain the proposed design methodology and they are not representative of any real structures.

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