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Effect of Converting Connections to Moment-Resisting on the Seismic Retrofit of Existing Steel Structures

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ABSTRACT

Converting beam-to-column connections to moment-resisting connections can be considered as a retrofit strategy for various steel structural systems, including moment frames, braced frames, and mixed (dual) systems. Welded Flange Plate (WFP) connections are rigid beam-to-column connections in which steel plates are welded to the beam and column flanges to provide flexural strength and stiffness. In this study, an existing five-story steel building was evaluated under a vertical expansion scenario (adding stories up to nine). First, the structure was modeled in ETABS as an intermediate moment-resisting frame (IMRF) in the X direction combined with a special concentric braced frame (SCBF) in the Y direction. Next, an alternative model with the same geometry and member sizes was developed, in which the connections in the moment-frame direction were replaced with WFP rigid connections, resulting in a dual system. Member checks and acceptance evaluations were performed using the Iranian guideline for seismic retrofit of existing steel buildings (Publication 1-363). The results indicate that, for the 7-, 8-, and 9-story configurations, stiffening the connections increases the number of braces that satisfy the acceptance criteria; approximately 50% of the braces that did not meet the acceptance criteria in the simple-frame system satisfy the criteria after connection stiffening in the dual system.

Keywords: Seismic retrofit, steel structures, WFP connections, adding stories, existing structures

1. INTRODUCTION

Seismic retrofit of existing steel structures is a specialized and important process aimed at improving seismic performance and increasing resistance against earthquake-induced demands. Older steel buildings may be vulnerable because they were designed according to earlier regulations and may not satisfy the requirements of modern seismic codes. Change of occupancy, increased gravity loads, and the need to extend the service life of a building are among the factors that highlight the necessity of seismic retrofit. Several retrofit approaches are available for existing steel structures, and the selection of a suitable approach depends on the structural system, the level of vulnerability, the applicable code requirements, and the available budget. Common approaches include:

a) Connection strengthening: Beam-to-column connections are often among the weak points of steel structures. Strengthening connections—by converting them to moment-resisting connections, adding cover plates, or using strengthening jackets—can increase their strength and ductility under seismic actions.

b) Increasing member sizes: Where architectural and clearance constraints allow, increasing the size of beams and columns can enhance seismic resistance.

c) Adding new lateral-force-resisting elements: Braces and bracing systems can be added to improve lateral strength, stiffness, and stability.

d) Using advanced materials: Innovative materials such as FRP composites, due to their low weight and high strength, can be used as alternatives or complements to conventional retrofit techniques in steel structures.

Selecting an appropriate retrofit strategy requires detailed engineering studies and an assessment of the building condition by experienced structural and earthquake engineering professionals. When retrofit is implemented properly, earthquake-induced life and economic losses can be reduced and the service life of older steel buildings can be extended.

In 2017, Fathalian et al. reviewed different retrofit methods for existing steel buildings. They discussed approaches such as increasing strength and load-carrying capacity, introducing new structural systems, using strengthening elements, upgrading connections, and designing with explicit consideration of seismic behavior [1].

In 2007, Zandi et al. conducted vulnerability assessment and retrofit of a 14-story building with a steel frame and a reinforced concrete central core, constructed in 1350 in central Tehran. Based on evaluations at the Immediate Occupancy (IO) and Life Safety (LS) performance levels, they found that the RC core did not have adequate strength and ductility to meet the target performance levels. Several retrofit alternatives were studied; ultimately, the focus was placed on enhancing the shear capacity and increasing confinement of the concrete core by adding shear reinforcement and increasing the concrete thickness. Nonlinear analyses confirmed the adequacy of the retrofit design and its consistency with existing-building retrofit guidance (FEMA 356) [2]. In 2015, Azizi et al. investigated seismic retrofit of a two-story steel building in Mashhad, located in a region with high seismic hazard. A detailed model was developed in ETABS, and the provisions of Publication 360, "Guideline for Seismic Retrofit of Existing Buildings," were applied. The study indicated that the steel frame was weak under seismic loads and required strengthening; adding steel bracing was proposed as a suitable retrofit option [3]. In 2010, Biglari and Zahedi evaluated the seismic performance of steel buildings with a simple building frame system and eccentric bracing. Three buildings (5, 10, and 15 stories) were studied. Their results showed that although the linear analyses suggested weak performance, nonlinear analyses indicated satisfactory behavior, and therefore seismic retrofit was not required under the provisions of Publication 360 [4].

In 2017, Reisi Dehkordi et al. compared the seismic performance of existing steel moment frames retrofitted using concentric bracing, eccentric bracing, and buckling-restrained braces (BRBs). They noted that existing steel moment frames designed according to the first edition of Iranian Standard 2800 generally require retrofit in terms of stiffness and ductility. Concentric bracing increases stiffness and reduces lateral displacements; however, due to buckling of compression braces, it may lead to sudden strength degradation and limited ductility. In eccentric bracing, plastic deformations are concentrated in the link beams while the braces remain largely elastic, resulting in higher ductility compared to concentric bracing. BRB retrofit prevents compression buckling and allows stable yielding in tension and compression, which improves stiffness, strength, and ductility relative to conventional concentric braces and enhances the seismic performance level of the structure [5].

In 2018, Rezaei et al. evaluated the seismic reliability of steel moment frames before and after retrofit using concentric bracing. Probabilistic models were employed to estimate the probability of failure under earthquakes. They concluded that probabilistic modeling can more accurately account for uncertainties in key parameters, and that retrofit with concentric bracing can significantly reduce the probability of failure, providing useful information for retrofit decision-making [6].

In 2015, Seyed Kazemi et al. investigated retrofit methods for ordinary steel moment frame buildings. Representative buildings were designed according to Iranian Standard 2800 and then checked for critical frames using different evaluation procedures, including nonlinear static analysis, linear dynamic response-spectrum analysis, and time-history analysis. They concluded that different retrofit approaches can significantly improve seismic performance; the appropriate method depends on factors such as building height, structural system, geographical location, and seismicity. They also reported that the increase in structural weight would be limited to about 14% for the investigated retrofit schemes [7].

In 2023, Fahimi Farzam and Yousefi studied the seismic performance of steel moment frame buildings equipped with lead-rubber bearings (LRBs). Incremental Dynamic Analysis (IDA) was performed on 3- and 9-story steel models with and without LRBs under 22 pairs of far-field ground motions recommended in FEMA P695. They found that LRBs can substantially increase the fundamental period, thereby reducing base shear and stress demands in structural elements, and can significantly reduce the probability of damage and collapse. The beneficial effects were more pronounced in low-rise buildings [8].

In 2021, Mortazavi et al. investigated the performance of concrete-steel composite buildings with and without mid-story seismic isolation in near-fault ground motions. Three buildings (4, 7, and 10 stories) were modeled with LRB isolators of different damping ratios and periods and analyzed under near-fault earthquakes. They reported: (a) mid-story isolation can markedly reduce base shear, floor accelerations, and interstory drifts; (b) the effectiveness of isolation decreases with increasing building height; and (c) increasing isolator damping and period reduces story shear in isolated structures. They concluded that mid-story isolation can be an efficient retrofit strategy for near-fault regions, especially for low-rise composite buildings [9].

In 2021, Bahmani and Zohraei investigated the effect of viscous dampers on the seismic retrofit of steel frames using fragility analysis. A five-story steel frame was modeled and analyzed in OpenSees. Four earthquake records and three types of viscous dampers with different damping ratios were considered [10].

In 1402, Mohbi et al. evaluated the seismic performance of steel buildings retrofitted with elliptical supplemental dampers. The general design principles of the dampers were described, and a four-story steel frame was modeled in OpenSees and analyzed under four earthquake records and three damper configurations with different geometric parameters. The results showed that such dampers can significantly reduce base shear, floor accelerations, and interstory drift, and optimized geometric parameters can notably improve seismic performance and reduce collapse probability [11].

Di Trapani et al. proposed a genetic algorithm-based framework for optimal seismic retrofit of reinforced concrete buildings using steel jacketing. Mathematical models were developed to represent the seismic behavior of jacketed structures, and a genetic algorithm was used to optimize jacket thickness and layout to maximize seismic performance. Their results indicate that steel jacketing can significantly improve the seismic response of reinforced concrete structures and that the proposed optimization framework can efficiently identify effective retrofit designs [12].

Zhai et al. investigated seismic retrofit of steel frames using buckling-restrained braces (BRBs) and compared the seismic performance of BRB-retrofitted frames with conventional steel frames. Their findings highlight the advantages of BRBs for retrofit and indicate that BRB-retrofitted frames can markedly improve the seismic performance of steel moment-resisting frames, providing an effective retrofit option [13].

2. METHODOLOGY

2.1. WFP (Welded Flange Plate) Connections

WFP (Welded Flange Plate) connections are rigid beam-to-column connections in steel frames. In this connection type, steel plates are used to connect the beam flanges to the column, providing flexural rigidity at the joint. The main components of a WFP connection are:

- Top flange plate: A plate placed on top of the beam top flange and welded to it.

- Bottom flange plate: A plate placed under the beam bottom flange and welded to it.
- Web plate: A plate placed between the top and bottom flange plates and welded to the column web.
- Web angle: An angle used to connect the web plate to the beam flange.

Design of this connection should comply with applicable standards, such as Part 10 of the Iranian National Building Regulations for steel structures [14]. The connection capacity should be determined based on the forces transmitted by the beam and column. Plate thicknesses, plate dimensions, web angle size, and weld type and length should be selected appropriately. Practical issues such as plate cutting, drilling, and fabrication tolerances should also be considered.

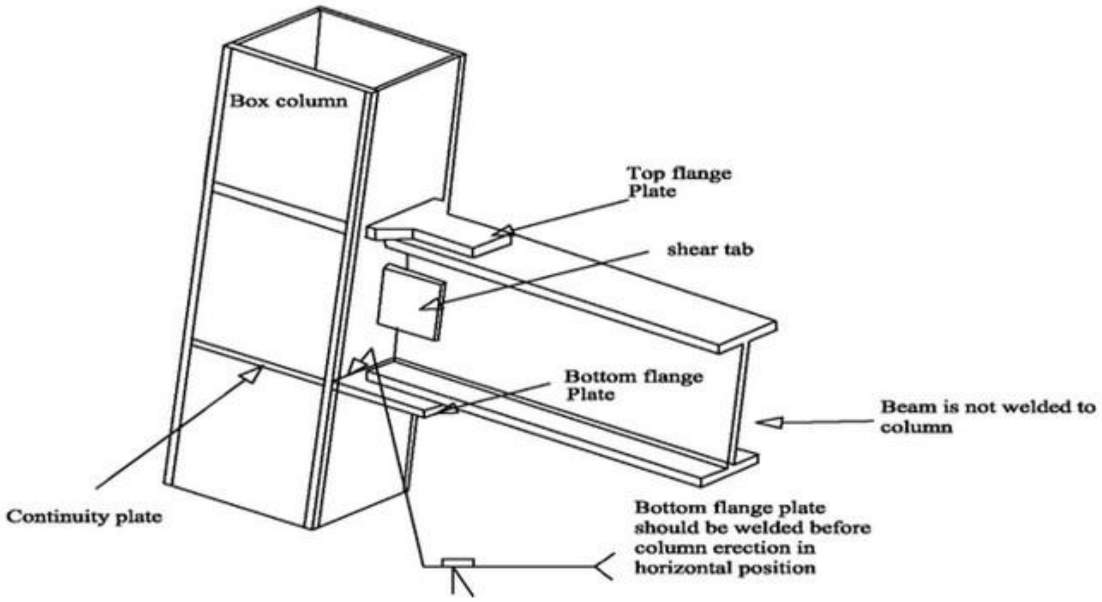

Figure 1. Components of the WFP connection [14]

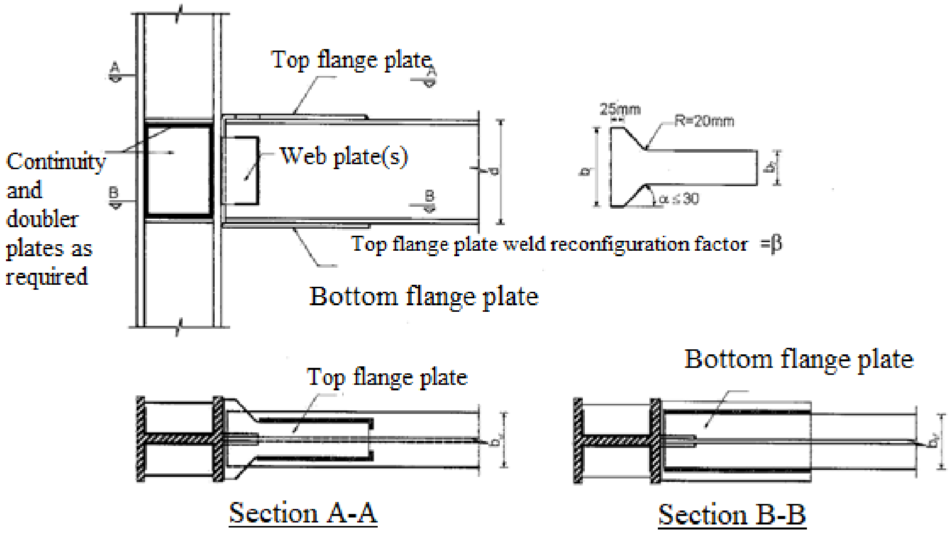

Figure 2. WFP connection details in Part 10 of the Iranian National Building Regulations [14]

2.2. Modeling Description

The structural models in this study are three-dimensional steel building models. Starting from an existing five-story structure, the number of stories was increased incrementally up to nine stories. First, the structure was modeled in ETABS as an intermediate moment-resisting frame (IMRF) in the X direction and a special concentric braced frame (SCBF) in the Y direction. Next, a second model was developed with the same geometry and member sections, in which all beam-to-column connections in the moment-frame direction were replaced with WFP rigid connections. In this second model, the structural system in the Y direction was considered as a dual system. The story height is 3 m, and the building has a rectangular plan as shown in Figures 3 and 4.

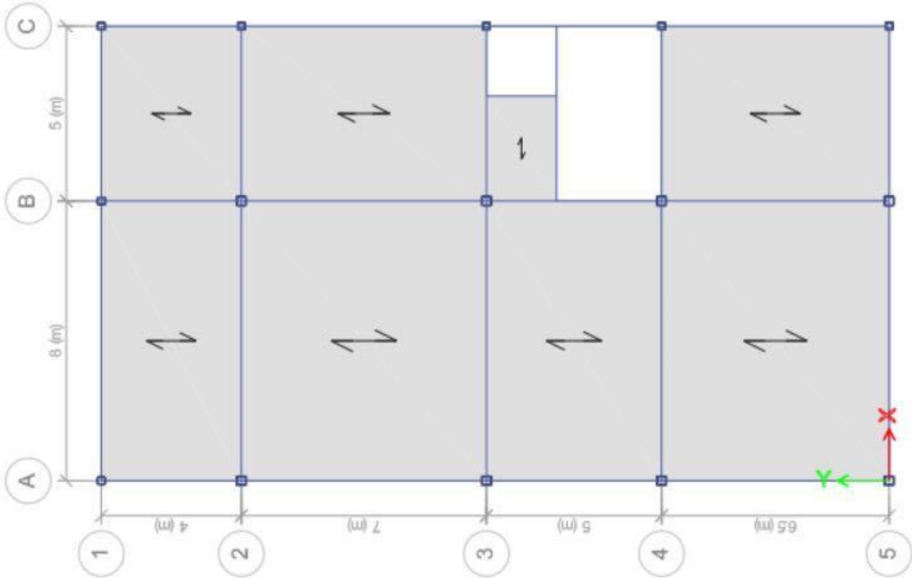

Figure 3. Plan view of the studied structures

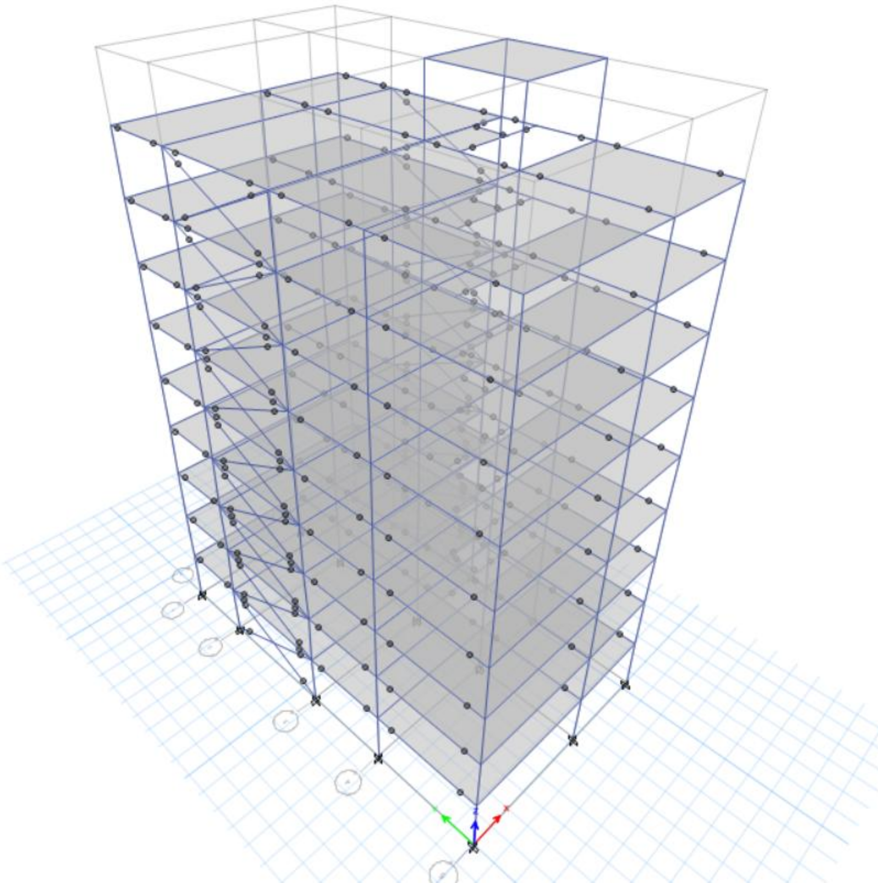

Figure 4. Three-dimensional ETABS model

The building location was assumed to be in a city with high seismic hazard and soil type II. The foundation dimensions for the five-story configuration were assumed as 1.5 m in width and 0.8 m in depth, which were considered adequate for the vertical expansion up to nine stories. For structural analysis, design, and lateral load definition, Part 10 of the Iranian National Building Regulations (1401 edition) [14] and Publication 1-363, "Practical Guide for Seismic Retrofit of Existing Buildings (Steel Buildings)," were used. Publication 363 has been officially issued since 1385 and has been widely used by practicing engineers and consultants. The purpose of this paper is to determine the number of members (columns, beams, and braces) that do not satisfy the acceptance criteria after adding stories up to nine and therefore require retrofit.

Table 1. Member sections used in modeling

Section	Object Type
IPE240	Beam
IPE270	Beam
PLG 01	Beam
PLG 02	Beam
PLG 06	Beam
PLG 08	Beam
PLG 09	Beam
BOX-30x30-2	Column
BOX-25x25-1.5	Column
BOX-20x20-1.5	Column
2UNP120	Brace
2UNP160	Brace

Table 2. Properties of beam sections used in modeling

Non-primary members (CP)	Non-primary members (LS)	Primary members (CP)	Primary members (LS)	All members (IO)	Component / Criteria	
IPE240	240	120	9.8	6.2	120	9.8
IPE270	270	135	10.2	6.6	135	10.2
PLG 01	320	150	10	8	150	10
PLG 02	324	150	12	8	150	12
PLG 06	330	200	20	8	200	20
PLG 08	415	200	20	10	200	20
PLG 09	415	250	20	10	250	20

3. RESULTS AND DISCUSSION

According to Sections 1.1.3.7 and 2.1.3.7 of ASCE/SEI 41-17, as well as the Iranian seismic rehabilitation guideline and its practical guide (Publications 360 and 1-363), the linear static analysis method is permitted when the building satisfies the limitations on height and structural regularity [15,16]. If the conditions for linear static analysis are not met, linear dynamic analysis may be used. In linear dynamic analysis, the structure is assumed to behave linearly and the maximum response is obtained from the contributions of vibration modes that significantly influence the global response. The peak response in each mode is computed from the standard design spectrum or a site-specific design spectrum based on the modal period, and the total structural response is estimated by statistical combination of the modal maxima. In this study, the required checks were performed and the corresponding analyses were carried out.

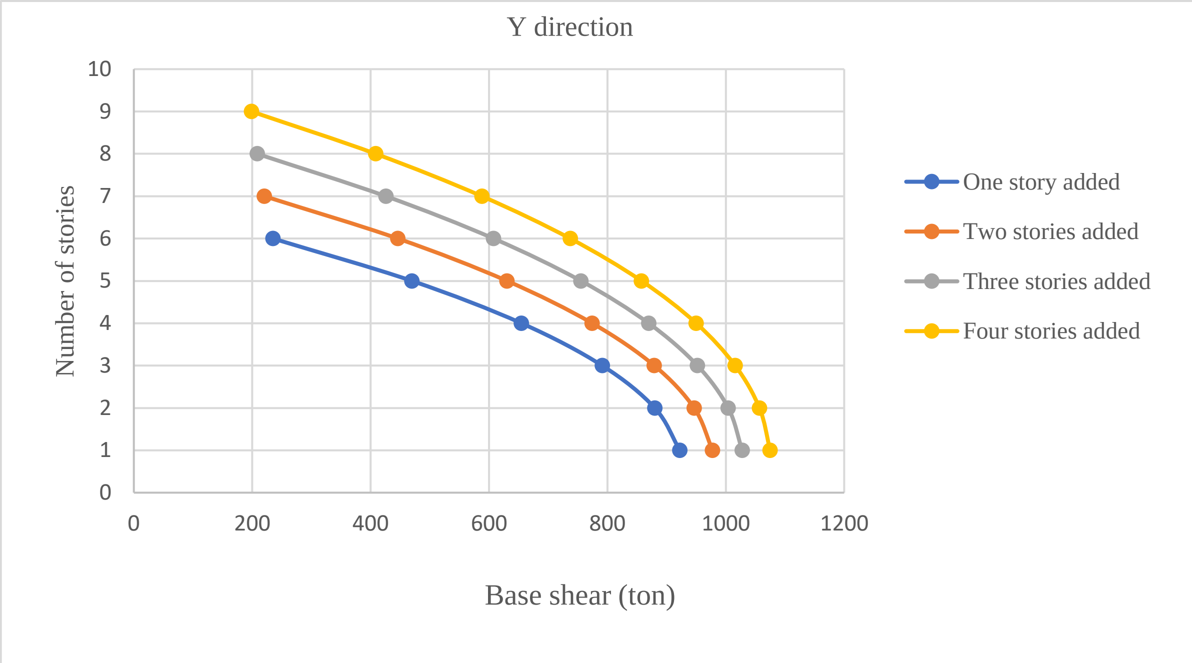

Figure 5. Base shear in the Y direction for the simple-frame system and the dual system

As shown in Figure 5, the base shear in the Y direction is the same for the two structural systems (simple frame and dual system). This is because, according to Publication 360, the fundamental periods of the two models are the same; therefore, the seismic coefficient and, consequently, the design base shear are equal for both systems.

Table 3. Acceptance criteria for beams (linear method) – steel structural components

Component / Criteria	**All members (IO)**	**Primary (LS)**	**Primary (CP)**	**Non-primary (LS)**	**Non-primary (CP)**
(a) Beams in flexure: h/tw ≤ 2.45√(E/Fye), bf/(2tf) ≤ 0.3√(E/Fye)	2	6	8	10	12
(b) Beams in flexure: 3.75√(E/Fye) ≤ h/tw ≤ 5.7√(E/Fye), 0.38√(E/Fye) ≤ bf/(2tf) ≤ 0.76√(E/Fye)	1/25	2	3	3	4
(c) For intermediate values between (a) and (b):	**use linear interpolation and adopt the smallest resulting value.**				
(d) h/tw ≥ 5.7√(E/Fye) or bf/(2tf) ≥ 0.76√(E/Fye).	**Force-controlled behavior for**				

Table 4. Acceptance criteria for columns (linear method) – steel structural components

Component / Criteria	**All members (IO)**	**Primary (LS)**	**Primary (CP)**	**Non-primary (LS)**	**Non-primary (CP)**
Columns in flexure (major-axis bending): PUF/PCL ≤ 0.2					
(a) h/tw ≤ 1.76√(E/Fye), bf/(2tf) ≤ 0.3√(E/Fye)	2	6	8	10	12
(b) 2.7√(E/Fye) ≤ h/tw ≤ 4.4√(E/Fye), 0.38√(E/Fye) ≤ bf/(2tf) ≤ 0.76√(E/Fye)	1/25	1/25	2	2	3
(c) For intermediate values between (a) and (b):	**use linear interpolation and adopt the smallest resulting value.**				
(d) h/tw ≥ 4.4√(E/Fye) or bf/(2tf) ≥ 0.76√(E/Fye).	**force-controlled behavior**				
Columns in flexure (major-axis bending): 0.2 < PUF/PCL ≤ 0.5					
(a) h/tw ≤ 1.52√÷(E/Fye), bf/(2tf) ≤ 0.3√(E/Fye)	1/25	9(1-1.7PUF/PCL)	12(1-1.7PUF/PCL)	15(1-1.7PUF/PCL)	18(1-1.7PUF/PCL)

(b) 2.34√(E/Fye) ≤ h/tw ≤ 3.8√(E/Fye), 0.38√(E/Fye) ≤ bf/(2tf) ≤ 0.76√(E/Fye)	1/25	1/25	1/5	2	2
(c) For intermediate values between (a) and (b):	**use linear interpolation and adopt the smallest resulting value.**				
(d) h/tw ≥ 3.8√(E/Fye) or bf/(2tf) ≥ 0.76√(E/Fye).	**Force-controlled behavior**				
For PUF/PCL > 0.5:	**force-controlled behavior**				

Table 5. Acceptance criteria for braces (linear method) – steel structural components

Compression bracing (except eccentric bracing)					
For KL/r ≥ 4.2√(E/Fye)					
(a) Double angles / double channels – in-plane buckling	1/25	6	8	7	9
(b) Double angles / double channels – out-of-plane buckling	1/25	5	7	6	8
(c) I or Z section	1/25	6	8	7	9
(d) Box section and tubular sections	1/25	5	7	6	8
For KL/r ≤ 2.1√(E/Fye)					
(a) Double angles / double channels – in-plane buckling	1/25	5	7	6	8
(b) Double angles / double channels – out-of-plane buckling	1/25	4	6	5	7
(c) I or Z section	1/25	5	7	6	8
(d) Box section and tubular sections	1/25	4	6	5	7
For 2.1√(E/Fye) < KL/r < 4.2√(E/Fye)	**use linear interpolation between stocky and slender brace values.**				
Tension bracing (except eccentric bracing)	1/25	6	8	8	10
Steel shear walls	1/5	8	12	12	14
Diaphragm components					
(a) Diaphragm shear yielding, or buckling in panel zone/plate	1/25	2	3	2	3
(b) Chord and collector elements – adequate lateral bracing	1/25	6	8	6	8
(c) Chord and collector elements – limited lateral bracing	1/25	2	3	2	3

Using the acceptance criteria in Tables 3 to 5, the beams, columns, and braces were evaluated. Based on the results, all beams in both the simple-frame system and the dual system satisfy the acceptance criteria; therefore, increasing the number of stories does not necessitate retrofit of beams in the studied configurations.

Table 6. Number of members that did not satisfy the acceptance criteria in the modeled structural systems

Configuration	Simple frame with concentric bracing – Columns	Simple frame with concentric bracing – Braces	Dual system – Columns	Dual system – Braces
5-story + 1 added story	20	2	22	0
5-story + 2 added stories	26	6	27	2
5-story + 3 added stories	47	12	49	6
5-story + 4 added stories	76	12	75	6

The results summarized in Table 6 indicate that for the case of adding one story (six-story configuration), after converting the connections to moment-resisting connections, all braces satisfy the acceptance criteria. For the 7-, 8-, and 9-story configurations, connection stiffening increases the number of braces that satisfy the acceptance criteria, and in the best case the number of noncompliant braces decreases from 12 to 6 (about a 50% reduction) compared with the simple-frame system. However, it should be noted that, in general, the number of columns that satisfy the acceptance criteria in the simple-frame system is higher than in the dual system.

3.1. Detailed Retrofit Design

Based on the analysis results, it is observed that in both the simple-frame and dual-system configurations, the structures do not provide sufficient lateral stiffness to satisfy the allowable displacement limits. Therefore, the first step is to improve the lateral stiffness in both directions so that the identified deficiencies in member capacities—particularly in some columns—can be mitigated. Several alternatives can be considered for the studied building, given the existing structural system and the observed weaknesses.

3.2. Steel Jacketing of Columns

Using steel jacketing to strengthen columns and beams has the advantage that it prevents excessive increase in column dimensions, and the added weight of the structure is not expected to be significant. However, its effect on global lateral stiffness is limited, and a large number of columns and beams would need to be strengthened using this approach. Therefore, this option is not considered economical for the studied case.

3.3. Steel Jacketing Combined with Steel Bracing

The advantages of steel jacketing for strengthening beams and columns were noted above. In addition, the deficiency in lateral stiffness can be addressed by adding steel braces (concentric or eccentric), which makes this alternative attractive in terms of cost and construction speed. Moreover, with appropriate distribution of braces along building perimeters, the uplift phenomenon in the foundation can be reduced as much as possible. Considering these points, this option is judged to be acceptable for the studied structure.

4. CONCLUSION

In general, seismic retrofit of existing steel structures is an efficient and cost-effective approach to improve performance and extend service life. By selecting an appropriate retrofit strategy and implementing it properly, older buildings can be upgraded to safer structures with improved resistance to natural hazards. Based on the results of this study, the following conclusions can be drawn:

1) Since all beams satisfy the acceptance criteria in both systems, the connection type (moment-resisting versus simple) does not affect the retrofit requirements of the beam members in the studied cases.

2) According to Table 6, converting the connections in the Y direction to moment-resisting connections does not improve the acceptance of columns; the simple-connection configuration leads to better column performance.

3) After stiffening the connections, a larger number of braces satisfy the acceptance criteria, and in the best case (Table 6) the number of noncompliant braces decreases from 12 to 6.

Therefore, considering that connection stiffening has no effect on beam members and may adversely affect column members, this retrofit approach is not expected to be cost-effective for the studied building.

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AUTHORS CONTRIBUTION
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CONFLICT OF INTEREST
The author (s) declared no potential conflicts of interests with respect to the authorship and/or publication of this paper.

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