Seismic Vulnerability of Structures Designed in Accordance with the Allowable Stress Design and Load Resistant Factor Design Methods

Mohammadreza Vafaei, Amirali Moradi, Sophia C. Alih

***Abstract*—**The method selected for the design of structures not only can affect their seismic vulnerability but also can affect their construction cost. For the design of steel structures, two distinct methods have been introduced by existing codes, namely allowable stress design (ASD) and load resistant factor design (LRFD). This study investigates the effect of using the aforementioned design methods on the seismic vulnerability and construction cost of steel structures. Specifically, a 20-story building equipped with special moment resisting frame and an eccentrically braced system was selected for this study. The building was designed for three different intensities of peak ground acceleration including 0.2 g, 0.25 g, and 0.3 g using the ASD and LRFD methods. The required sizes of beams, columns, and braces were obtained using response spectrum analysis. Then, the designed frames were subjected to nine natural earthquake records which were scaled to the designed response spectrum. For each frame, the base shear, story shears, and inter-story drifts were calculated and then were compared. Results indicated that the LRFD method led to a more economical design for the frames. In addition, the LRFD method resulted in lower base shears and larger inter-story drifts when compared with the ASD method. It was concluded that the application of the LRFD method not only reduced the weights of structural elements but also provided a higher safety margin against seismic actions when compared with the ASD method.

***Keywords*—**Allowable stress design, load resistant factor design, nonlinear time history analysis, seismic vulnerability, steel structures.

I. INTRODUCTION

IN the design of structures, safety and economy aspects should be considered simultaneously. Structural safety against applied loads is specifically addressed by design codes; however, they often do not contribute to the economical aspect of the design guidelines and specifications. Moreover, change in the design specifications is often made because of improving the structural safety rather than making them more economical. For instance, ASD was the main design approach for steel structures for decades and it was widely employed by different design codes like AISC-ASD 89 [1]. However, after decades of researches, it was gradually replaced by a new design approach that nowadays is referred to as LRFD [2] or Limit State Design (LSD) [3]. The main purpose of the LRFD method is to take the uncertainties in the material and geometric properties as well as the ones in the modeling into account through a sophisticated approach. Therefore, the LRFD method aims to provide a higher safety margin for steel structures when compared with the ASD method. It is noteworthy that, because of difference in the design procedure, for a given structure, these two methods result in different sizes for beams, columns, and braces. In the other words, stiffness distribution and consequently the natural period of a building designed by these two methods can be different. Obviously, for the gravity, load design difference in the stiffness distribution is not as important as the design against lateral loads. This is because stiffness distribution plays a significant role in the dynamic behavior of structures especially when they are subjected to seismic actions. So far, less attention has been paid to the comparison between the seismic vulnerability of structures designed based on the ASD and LRFD methods. In addition to the different safety margin that these two design approaches may result in, a cost estimation can indicate which of these two methods provide a relatively more economical design. Considering this fact that in many countries design engineers are allowed to employ either of these two methods, this study was performed to compare the safety and economical aspects of steel structures designed based on the ASD and LRFD methods.

In this study, a 20-story tall steel structure is considered as the reference structure and it has been designed based on the ASD and LRFD methods. In the next section, at first, the reference structure is introduced. Then, the design criteria selected for gravity and lateral loads are explained. Next, the obtained weights for structural elements are presented and compared. Finally, the obtained seismic performances for the both design approaches are discussed.

II. REFERENCE STRUCTURE

As shown in Fig. 1, a typical architectural plan was considered for the reference structure. It was assumed that the building was a 20-story tall steel structure with a dual lateral load resisting system. The lateral load resisting system comprised of a special moment resisting frame together with an eccentrically braced frame. In order to reduce the computational time and effort, only one frame of this structure was investigated. Fig. 2 displays the analyzed frame in this study. As can be seen from this figure, the studied frame has

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four spans with an identical length of 5.6 m. All stories have also an identical height of 3 m which totally results in 60 m height for the frame. The length of link beam in eccentric braces is identical and it is equal to 0.5 m. This length was selected to make sure that a shear failure will occur for link elements. All beam-to-column and support connections were simulated as a fixed connection.

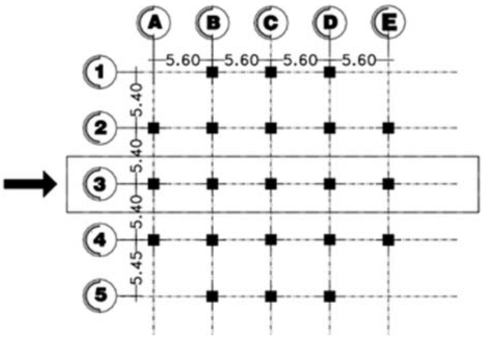


Fig. 1 Plan of the reference structure

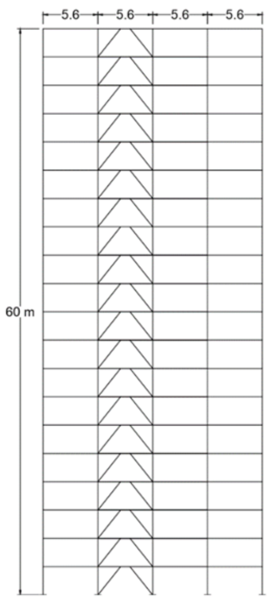


Fig. 2 The studied frame in this study

A. Loading of the Reference Structure

Assuming that the reference structure is a residential building, the dead and live loads for the design of structural elements for the both ASD and LRFD methods were considered to be 6 kN/m^2 and 2 kN/m^2, respectively. In addition, the seismic design of the reference structure was carried out by using the modal response spectrum analysis. It is noteworthy that, unlike gravity loads, the seismic forces calculated for the ASD and LRFD methods are different. The main reason for this difference is because the LRFD method makes use of structural elements' ultimate capacity, while the ASD method relies on a percentage of their yield capacity. Therefore, design codes suggest a lower response modification factor for the LRFD method in comparison to the ASD method. In this study, for the calculation of seismic loads, the response modification factors amounted to 7.5 for the LRFD method and 10.5 for the ASD method. This results in 40% higher base shear for the LRFD method when compared with the ASD method. In many seismic codes, the response modification factor of the ASD method is suggested to be 1.4 times more than that of the LRFD method [4], [5]. For the seismic analysis of the reference structure, the site class (i.e. soil type) was assumed to be similar to the site class C of ASCE/SEI 7-10 [6]. In addition, in the seismic design, the importance factor of the structure was considered to be 1. The reference structure was designed for three different intensities of peak ground acceleration (PGA) that included 0.2g, 0.25g, and 0.3g. Fig. 3 shows the design response spectra employed in this study. In the seismic analysis of the reference structure, all requirements by ASCE/SEI 7-10 [6] for dual lateral load resisting systems were satisfied for the ASD and LRFD design methods. Moreover, a displacement amplification factor of 4 was selected for the reference structure, and its inter-story drifts for the both design methods were limited to the code prescribed values [6]. Table I shows the seismic response coefficients (C_S) used for the calculations of seismic base shears.

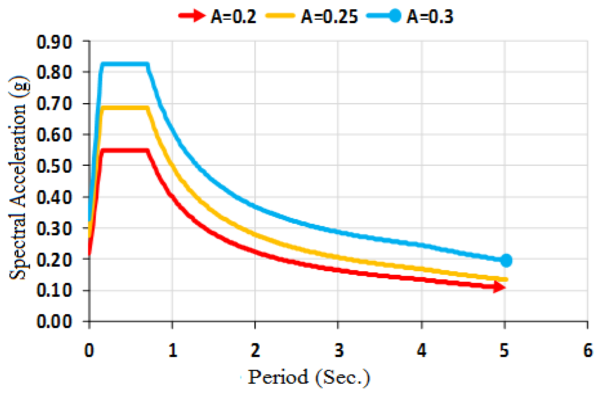


Fig. 3 Design response spectra

TABLE I
SEISMIC RESPONSE COEFFICIENTS

Design Method	PGA	C_S
	0.2	0.0297
ASD	0.25	0.0367
	0.3	0.0464
	0.2	0.0411
LRFD	0.25	0.0514
	0.3	0.065

B. Design of Structural Elements

The design of structural elements for the ASD and LRFD methods was carried out using AISC-ASD 89 [1] and ANSI/AISC 360-10 [2] codes, respectively. The cross sections of columns were selected to be a box-shaped. Beams had I-shaped cross sections and for braces, double channel cross sections were employed. The size of flanges and webs was selected such that all structural elements had the compact cross section as per requirements of seismic codes [6]. The yield stress and tensile strength of structural elements were assumed to be 235 Mpa and 360 Mpa, respectively. In the LRFD design method, the Direct Analysis Method was employed to check on the stability of steel members. In addition, the stiffness reduction of structural elements in the LRFD method was considered through the Tau-b variable approach. During the design process, sizes of beams, columns, and braces were selected such that all demand to capacity ratios (D/C ratios) were less than one and greater than 0.8. By this way, for both ASD and LRFD methods, structural elements were loaded in a safe range and the obtained weights for structural elements became comparable. For the analysis and design of the reference structure, ETABS [7] software was employed.

III. Results of Structural Design

After analyzing the reference structure for the described gravity and seismic loads, structural elements were designed based on the specifications of the selected codes. Tables II -IV show the total weights obtained for beams, columns, and braces of the reference structure considering the different design methods and design PGAs. Table V summarizes the total weights obtained for all structural elements.

TABLE II
WEIGHTS OF BEAMS

Design Method	PGA(g)	Weight (kN)
ASD	0.2	318.7
	0.25	337
	0.3	366.8
LRFD	0.2	224.6
	0.25	239.05
	0.3	267.2

TABLE III
WEIGHT OF COLUMNS

Design Method	PGA(g)	Weight (kN)
ASD	0.2	779.6
	0.25	803.7
	0.3	860.9
LRFD	0.2	492.3
	0.25	525.03
	0.3	572.2

TABLE IV
WEIGHT OF BRACES

Design Method	PGA(g)	Weight (kN)
ASD	0.2	32.8
	0.25	35.5
	0.3	40.8
LRFD	0.2	34.4
	0.25	38.3
	0.3	45.3

As can be seen from Table V, for all design PGAs, the total weights of structural elements obtained from the LRFD method are significantly lower than those of the ASD method. Considering this fact that the D/C ratios of all structural elements for the both design methods have been in the range of 0.8 to 1, it can be concluded that the LRFD method has led to a more economical design. A comparison between the obtained weights for beams, columns, and braces shows that, regardless of the design PGA, the LRFD method has resulted in a higher weight for braces when compared with the ASD method. On the other hand, the ASD method has significantly increased the weights of columns and beams when compared with the LRFD method.

TABLE V
TOTAL WEIGHTS OF STRUCTURAL ELEMENTS

Design Method	PGA(g)	Weight (kN)
ASD	0.2	1131.08
	0.25	1176.09
	0.3	1268.5
LRFD	0.2	751.5
	0.25	802.4
	0.3	884.7

Fig. 4 compares the relative increase in the weights of structural elements when the design PGA increases from 0.2 g to 0.25 g and 0.3 g. It is evident from this figure that, compared to the ASD method, the LRFD method has a higher increase in the weights of beams, columns, and braces as the design PGA increases. This can also be seen that there is no linear relationship between the increase in the design PGA and increase in structural elements' weights. Results also indicate that, as the design PGA increases, for the both design methods, increase in the weights of braces are more than beams and columns.

In short, for the reference structure, the weights obtained from the ASD and LRFD methods imply the superiority of the LRFD method over the ASD method when it comes to an economical design.

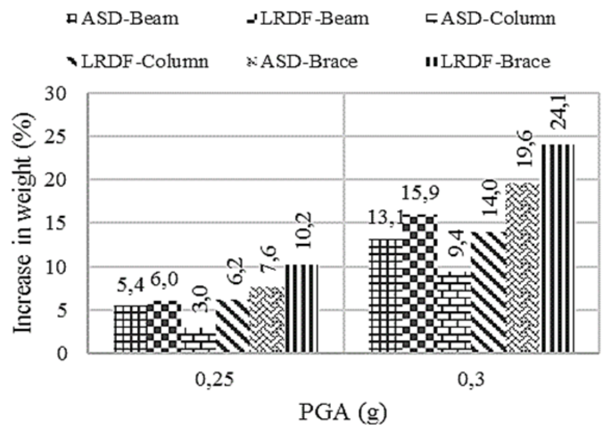


Fig. 4 Increase in the weights of structural elements compared to the design PGA of 0.2 g

IV. Seismic Vulnerability Study

The seismic vulnerability study was carried out using nonlinear time history analysis (NTHA). Totally, nine natural earthquake records were selected and employed in NTHA. The selected earthquake records were scaled to the design response spectra before using them for NTHA. Table VI displays the characteristics of the selected records. In the selection of earthquake records, special attention was paid to their spectral acceleration and PGA/Peak Ground Velocity (PGV) ratios. It has been shown that this ratio has a significant influence on the seismic response of structures [8]-[10]. Therefore, as can be seen from Table VI, the selected records covered a wide range of PGA/PGV ratio in accordance with the classification proposed by Tso et al. [8].

TABLE VI
SELECTED EARTHQUAKE RECORDS

No	Earthquake Name	Date	PGA (cm/s^2)	PGV (cm/s)	PGA/PGV
1	LONG BEACH	1933	62.3	17.3	3.6
2	LOWER CALIFORNIA	1934	156.8	20.8	7.5
3	SAN FERNANDO	1971	98.7	19.3	5.1
4	IMPERIAL VALLEY	1940	341.7	33.4	10.2
5	KERN COUNTY CALIFORNIA	1952	175.9	17.7	9.9
6	BORREGO MOUNTAIN	1968	45.5	4.2	10.8
7	PARKFIELD CALIFORNIA	1966	264.3	14.5	18.2
8	HELENA MONTANA	1935	143.7	7.2	20.0
9	LYTLE CREEK	1970	194.4	9.6	20.3

The inelastic behavior of beams, columns, and braces were taken into account using the plastic hinge approach. Fig. 5 displays the force-deformation relationship of plastic hinges which is employed in this study. The ASCE 41 [11] recommendations along with the cross-sectional properties of beams, columns, and braces were employed to define the required parameters shown in this figure. In the finite element models, plastic hinges were assigned to the beginning and end of beams and columns. One plastic hinge was also assigned to the middle length of each brace. The link beams' plastic hinges were assigned to their middle length as well as their beginning and end.

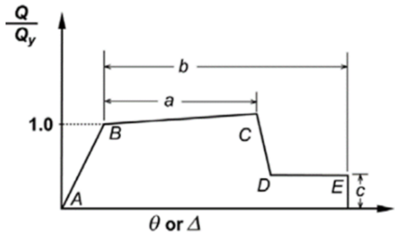


Fig. 5 The generalized force-deformation relationship employed for plastic hinges

A. Results of Inter-Story Drifts

This section compares the obtained results for the inter-story drifts after performing NTHA. Figs. 6-8 display the average of obtained inter-story drifts for the reference structure when it was designed in accordance with the ASD and LRFD methods for different design PGAs. ASCE 41-13 [11] proposes a 0.7% transit drift for the immediate occupancy (IO) structural performance level of braced frames. As can be seen from the figures, for all design PGAs, the reference structure has satisfied the ASCE 41-13 drift requirement regardless of the employed design method. However, for all design PGAs, the maximum inter-story drift obtained from the LRFD method is larger than that of the ASD method. This can be also observed that, except few stories at lower levels, for other stories, the inter-story drifts of the ASD method are smaller than that of the LRFD method. In addition, in the LRFD method increase in the design PGA has increased the maximum inter-story drift of the structure. However, the maximum inter-story drift of the reference structure when designed in accordance with the ASD method shows a negligible change for different design PGAs (i.e. ~0.3%). This can be concluded that although the both design methods satisfy the immediate occupancy drift limit, the LRFD method results in larger inter-story drifts when compared with the ASD method.

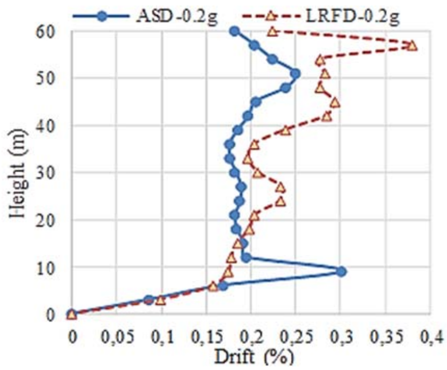


Fig. 6 The average of Inter-story drifts for the design PGA of 0.2 g

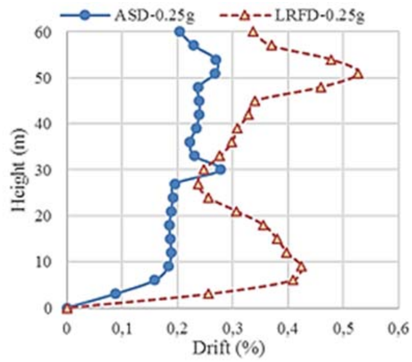


Fig. 7 The average of Inter-story drifts for the design PGA of 0.25 g

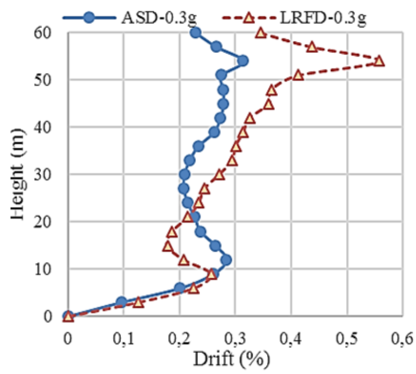


Fig. 8 The average of Inter-story drifts for the design PGA of 0.3g

B. Results of Story Shear Forces

The average of story shear forces obtained from NTHA is presented in Figs. 9-11 considering different design PGAs. It is evident from the figures that the LRFD design method has resulted in a lower shear forces for all design PGAs when compared with the ASD design method. As can be seen from Table V, one main reason for this observation is the lower weights that structural elements have when they are designed in accordance with the LRFD method. The lower structural weights reduce the seismic weights that contribute to the calculation of story shear forces. It is noteworthy that, regardless of the design PGAs, at upper levels, the differences between the story shear forces of the ASD and LRFD methods are smaller. The maximum difference between the shear forces occurs between 20 m to 40 m heights. Moreover, the average of base shears obtained for the ASD design method for the design PGAs of 0.2 g, 0.25 g, and 0.3 g are, respectively, 1.08, 1.10, and 1.17 times larger than that of the LRFD method. This indicates that, as the design PGA increases, the base shear obtained for the ASD method increases slightly more than the LRFD method. It is worth mentioning that, the both design methods have resulted in a similar pattern for the shear force distribution along the height of the reference structure. For the both design methods, the shear force is maximum at the base and decreases along the height almost linearly until the 20-m height. From 20 m to 40 m heights, the change in the story shear forces is insignificant, while from 40 m height until the roof level it decreases rapidly.

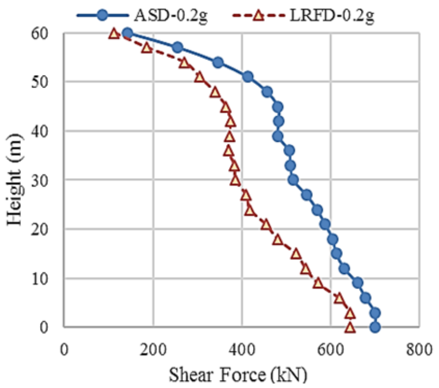


Fig. 9 The average of story shear forces for the design PGA of 0.2 g

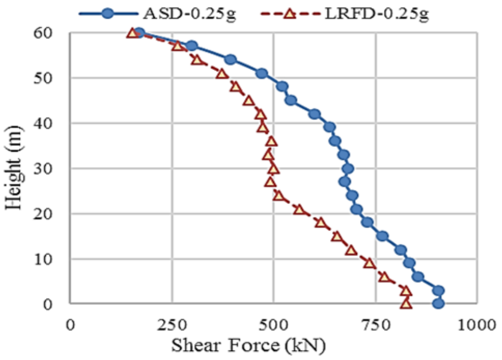


Fig. 10 The average of story shear forces for the design PGA of 0.25 g

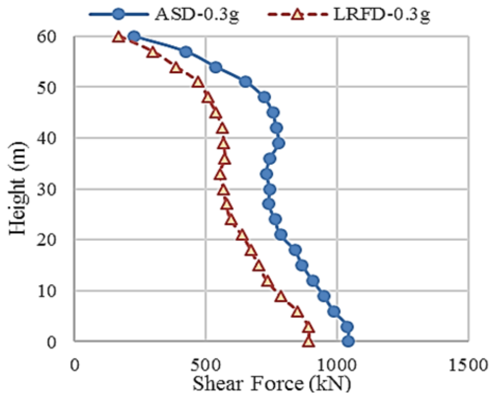
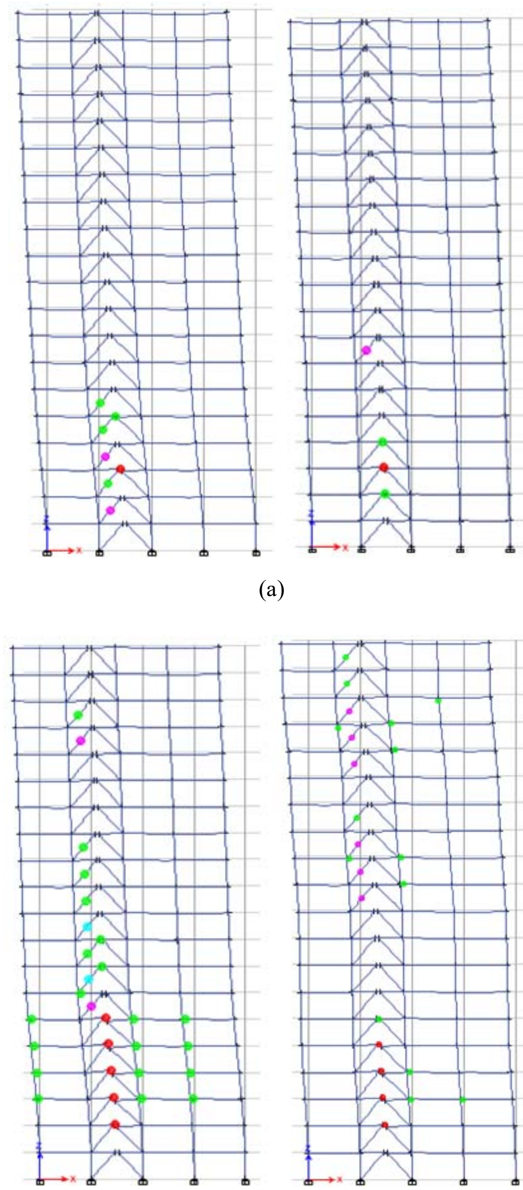
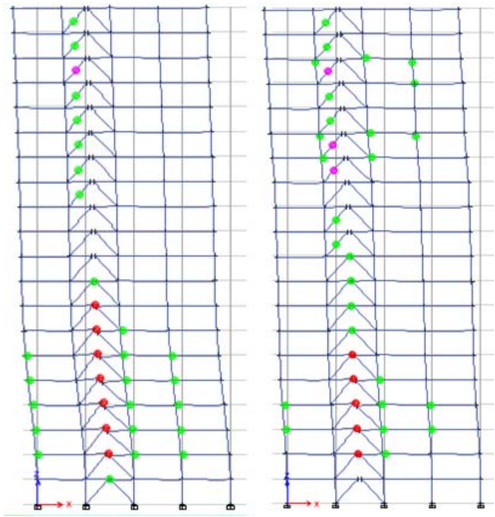


Fig. 11 The average of story shear forces for the design PGA of 0.3 g



(a)

(b)



(c)

Fig. 12 Plastic hinge formation in the reference structure when subjected to the Long Beach record (a) design PGA=0.2 g (b) design PGA=0.25 g (c) design PGA=0.3 g

A. Seismic Performance of the Reference Structure

Fig. 12 shows the plastic hinge formation in the reference structure when it is designed in accordance with the ASD and LRFD design methods and subjected to the first earthquake record of Table VI (i.e. Long Beach). The frame at the left side shows the obtained results for the ASD design method, while the frame at the right side shows the obtained results for the LRFD method. The green dot shown in this figure indicates that the deformation of the plastic hinge has reached to the IO performance level as defined by ASCE 41-13 [11]. Moreover, the life safety (LS) and Collapse Prevention (CP) performance levels have been shown by cyan and pink colors, respectively. A red color dot indicates that the deformation of the plastic hinge has passed the CP level.

It is evident from Fig. 12 that, regardless of the design method, frames that have been designed for higher PGAs have more plastic hinges passed the IO performance level. It can also be seen that regardless of the design method, frames that have been designed for higher PGAs have more plastic hinges formed at their link elements that have passed the CP performance level. In addition, Fig. 12 shows that, for the both ASD and LRFD design methods, no plastic hinge beyond the IO performance level has been formed in columns and beams (except link beams). Therefore, this can be concluded that the both designed methods have resulted in a satisfactory seismic performance for beams and columns. However, deformation in some plastic hinges of braces for the both design methods has reached to the CP performance level. It is evident from Fig. 12 that all the braces with the CP plastic hinge are located in the spans in which no plastic hinge has formed in their link beams. Since this type of failure mode is not a desirable seismic performance for a ductile structure, the ASD and LRFD methods have not been able to provide a satisfactory seismic performance for braces.

Table VII summarizes the obtained results for plastic hinge formations in beams (inclusive of link beams), columns, and braces considering all nine earthquake records used for NTHA. This table also confirms that, regardless of the design method, when frames have been designed for higher PGAs, they have had more plastic hinges formed at IO, LS, and CP performance levels. Moreover, this table shows that no CP performance level is observed for columns designed in accordance with the ASD and LRFD methods. This can also be seen that, for the both design methods, the number of plastic hinges at IO performance level is more than the CP performance level, and only a few plastic hinges are in the LS performance level. This implies that the transition in the seismic performance level of plastic hinges from IO to CP has been rapid. The main reason for this observation is that the majority of plastic hinges that are in CP level belong to the link beams. Since the shear failure mode governs the seismic behavior of the link beams, the acceptance criteria for LS and CP performance levels are very close to each other [11].

A comparison between the total numbers of plastic hinges shows that, when the reference structure has been designed in accordance with the LRFD method, the summation of all plastic hinges that have been formed in beams, columns and braces is less than the case where the reference structure is designed in accordance with the ASD method. Therefore, considering the obtained results for weights of structural elements and inter-story drift, this can be concluded that, for the case study in this research, the LRFD method in terms of safety and economy is superior in comparison with the ASD method.

TABLE VII
SUMMARY OF PLASTIC HINGE FORMATIONS FOR ALL EARTHQUAKE RECORDS USED IN NTHA

Methods Design	Design PGA	Element Type	IO	LS	CP
ASD	0.20 g	Brace	48	0	3
ASD	0.20 g	Beam	20	3	16
ASD	0.20 g	Column	3	0	0
		SUM	71	3	19
LRFD	0.20 g	Brace	31	3	8
LRFD	0.20 g	Beam	15	0	2
LRFD	0.20 g	Column	8	0	0
		SUM	54	3	10
ASD	0.25 g	Brace	79	1	3
ASD	0.25 g	Beam	32	1	30
ASD	0.25 g	Column	2	1	0
		SUM	113	3	33
LRFD	0.25 g	Brace	31	3	8
LRFD	0.25 g	Beam	15	0	2
LRFD	0.25 g	Column	8	0	0
		SUM	54	3	10
ASD	0.30 g	Brace	63	0	10
ASD	0.30 g	Beam	44	5	49
ASD	0.30 g	Column	0	0	0
		SUM	107	5	59
LRFD	0.30 g	Brace	50	4	8
LRFD	0.30 g	Beam	24	1	41
LRFD	0.30 g	Column	5	0	0
		SUM	79	5	49

V. Conclusions

In this study, the ASD and LRFD methods which are widely employed by designed engineers for steel structures are compared in terms of their vulnerability against seismic actions and their economical aspect. A typical 20-story tall steel structure was selected, and one of its frames was designed in accordance with the requirements of the ASD and LRFD methods. Three different design PGAs were included in the seismic design of the frame. After completion of the design, the weights obtained for beams, columns, and braces were compared. It was observed that the ASD method resulted in at least 1.4 times higher weight for structural elements in comparison with the LRFD method. In order to examine the vulnerability of the designed frames, they were subjected to nine natural earthquake records, and their nonlinear responses were compared. Results indicated that the both design methods satisfied the limit for the immediate occupancy performance level. However, the frames designed in accordance with the ASF method showed smaller inter-story drifts when compared with the LRFD method. In addition, although both design methods showed a similar pattern for shear force distribution along the height of the frames, the frames designed in accordance with the LRFD method had smaller base shears when compared with the ASD method. It was also observed that the numbers of plastic hinges formed in the beams, columns, and braces of the frames designed in accordance with the LRFD method were significantly less than that of the ASD method. Considering the obtained results, it was concluded that the LRFD method was a superior design approach both from safety and economy perspectives.

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