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An investigation of seismic parameters of low yield strength steel plate shear walls

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Abstract. Steel plate shear walls (SPSWs) are effective lateral systems which have high initial stiffness, appropriate ductility and energy dissipation capability. Recently, steel plate shear walls with low yield point strength (LYP), were introduced and they attracted the attention of designers. Structures with this new system, besides using less steel, are more stable. In the present study, the effects of plates with low yield strength on the seismic design parameters of steel frames with steel plate shear walls are investigated. For this purpose, a variety of this kind of structures with different heights including the 2, 5, 10, 14 and 18-story buildings are designed based on the AISC seismic provisions. The structures are modeled using ANSYS finite element software and subjected to monotonic lateral loading. Parameters such as ductility (μ), ductility reduction (R_μ), over-strength (Ω_0), displacement amplification (C_d) and behavior factor (R) of these structures are evaluated by carrying out the pushover analysis. Analysis results indicate that the ductility, over-strength and behavior factors decrease by increasing the number of stories. Also, the displacement amplification factor decreases by increasing the number of stories. Finally, the results were compared with the suggestions provided in the AISC code for steel plate shear walls. The results indicate that the values for over-strength, behavior and displacement amplification factors of LYP steel plate shear wall systems, are larger than those proposed by the AISC code for typical steel plate shear wall systems.

Keywords: steel plate shear wall; low yield strength; ductility factor; over-strength factor; response modification factor; displacement amplification factor

1. Introduction

All structures need a lateral-resisting system to withstand lateral loads. Shear wall systems that are built exclusively from reinforced concrete are common in many high-rise buildings around the world. Since 1970's, steel plate shear walls (SPSWs) have been used in many important and high-rise buildings as a lateral-resisting system to withstand wind and earthquake forces in the United States and Japan. Stiffness, ultimate strength, ductility and energy dissipation capacity are the main features of seismic performance of a steel plate shear wall (Astaneh-Asl 2001). A research conducted by Thorburn *et al*. (Thorburn *et al*. 1983) supported the SPSWs design philosophy that lowered the thickness of plates by allowing the occurrence of shear buckling; thereafter, lateral loads are carried in the panel through the diagonal tension field action. In the recent years, steel plate shear wall systems with low yield point (LYP) strength steel, have been introduced as an effective system for lateral resistance. Low-strength steel has very low yield strength and uniform post-buckling characteristics. The use of lateral load resisting system with LYP steel, makes a structure to dissipate a lot of energy in a severe earthquake. This causes high failures to be tolerated by the boundary elements. Therefore, it should be possible to replace them in an appropriate manner. Hence, these systems are connected to the structure using high strength joints and, it is possible to replace them easily. This can be done using the high-strength steel screws.

Many studies have been performed on steel plate shear wall systems with typical steel. In these studies, values have been proposed for the behavior and displacement amplification factors of these structures. On the other hand, in the case of steel plate shear walls with low-yield-strength steel, few studies have been done. The majority of experimental samples include those having one to three stories. In the experimental studies, various samples have been designed, including those with different width to thickness ratios, plates with holes, cutout plates with reinforced corners, dampers made of LYP and samples with various types of joints. Different types of loadings have been applied to the samples. Three single-story LYP steel plate shear wall specimens were designed by Vian and Bruneau (2004), and subjected to quasi-static cyclic loading. They showed that the cutout reinforced corner specimen can be a good design option, to access utilities through the wall. Chen and Kua (2004) used both conventional and high-strength steels in moment-resisting frames to investigate their utilization in conjunction with low yield point steel shear panels. They found that the combination of high-strength and LYP steel leads to a better

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performance in the seismic design of buildings. They suggested that the strength of boundary elements of shear walls should be at least 2 times higher than the strength of plates of the systems. Chen and Jhang (2006), examined five LYP steel plate shear walls under cyclic load, to study the stiffness, strength and energy dissipation capacity of these systems. They concluded that the LYP steel plate shear walls are able to maintain stable, up to story drift angle between 3 to 6 percent. The experimental results indicated that the LYP plates yield first and then, with the propagation of yielding zone, the beam-to-column connections will yield. They also expressed that reducing the width to thickness ratio of plates, from 200 to 100, does not increase remarkably the ultimate strength or ultimate deformation. In 2011, Chen and Jhang (2011) performed other experiments on four one-story samples to study the effects of width-to-thickness ratio of LYP plates on the inelastic shear buckling behavior of LYP steel plate shear walls subjected to monotonic loading. They also designed two multi-story LYP steel plate shear wall samples to examine the deformation, stiffness and energy dissipation capacity of this system under cyclic loading. Excellent deformation and energy dissipation capacity resulted for the specimens. Zirakian and Zhang (2015) examined the buckling and yielding behavior of unstiffened LYP steel plate shear walls by classifying them into three groups including slender, moderate, and stocky shear walls. They provided some practical recommendations for efficient seismic design of LYP steel plate shear wall systems. Zhang and Zirakian (2015) investigated the probabilistic assessment of three nine-story SPSW systems with LYP steel plates, using fragility function method. They showed that the use of thicker LYP steel plates in the seismic design of structures results in a better seismic performance and less damage potential as well. Edalati *et al*. (Edalati *et al*. 2015) studied low yield point steel shear walls retrofitted with Glass Fiber Reinforced Polymer under a pushover loading. They concluded that the use of these fibers in the oblique direction, improves the behavior of shear walls and increases the absorbance of energy. In another study, Zirakian and Zhang (2016) examined the design and retrofit of existing structures using steel shear wall systems with conventional and low yield point steels. They evaluated parameters such as base shear and moment, drift, acceleration, and web-plate ductility demands, and showed appropriate performance of these systems in the design of new structures as well as seismic retrofit of existence buildings.

Astaneh-Asl (2001), suggested some values for the over-strength, behavior and displacement amplification factors of steel plate shear walls, for both steel plate shear walls inside a gravity-resisting steel frames with simple beam-to-column connections, and a dual system with special steel moment frames and steel plate shear walls. The ANSI/AISC 341-05 code (2005) has proposed values for the displacement amplification factor of both special steel plate shear walls and special steel plate shear walls with special moment frames, which are 7 and 8, respectively. The values proposed for the behavior factor of the mentioned structures in this code are 6 and 6.5, respectively. In a case study in 2008, Kurban and Topkaya (2009), studied 44 un-stiffened steel plate shear walls with different geometry. They examined displacement amplification and behavior factors of steel plate shear wall systems. Despite the dispersion of the results, the increase in C_d versus R was evident.

In the present study, frames with LYP steel plate shear walls have been designed according to the AISC code. Then, using a pushover analysis, the ductility (μ), ductility reduction (R_μ) and over-strength (Ω_0) factors are computed. Using these coefficients, the behavior (R) and displacement amplification (C_d) factors of the structures are calculated. Finally, the results are compared with the suggestions provided in the AISC code for steel plate shear wall systems.

2. Required parameters

2.1 Different types of steel

In this section, the specifications of different types of steel, commonly used in lateral resistant structural systems have been explained. Numerous experiments, carried out in order to compare different types of steel in terms of strength, indicated a large difference between the strength of high-strength and mild steels. For this reason, many design parameters of these two types of steel, are not similar. For example, a difference in buckling shapes of flanges is produced in different design methods of beams and columns (Yamaguchi and Takeuchi 1998, De Matteis *et al*. 2003).

The term steel is used for iron alloys up to 1.5% carbon, which is frequently polymerized with other metals. Steel properties depend on their alloy metals, the percentage of carbon and heat treatments. From the viewpoint of mechanical properties, steel can be divided into several types. Low carbon steel is relatively soft and weak but it has significant ductility, durability and machining capabilities and also high weldability. In addition, its production cost is relatively low. In medium carbon steel, mechanical properties of steel are improved by special heat treatments and adding some alloys, that is, very high resistance values can be also obtained. By adding chemical elements to alloy steels, a wide variety of properties can be also achieved such as hardness, mechanical strength and chemical resistance.

The typical structural steel and high-strength structural steel, used in most of steel structures are ST-36 and ST-441, with a minimum yield stress of 250 N/mm^2 and 315 N/mm^2, respectively. The low-strength steel has a high plasticity and its nominal yield stress is between 90 N/mm^2 to 120 N/mm^2 (De Matteis *et al*. 2003). In fact, the reasons for the use of low-strength steel in comparison with typical and high-strength steels, can be summarized as follows (De Matteis *et al*. 2003):

- Equal elastic Young's modulus (E) of all three types of steel;
- High ductility of LYP steel in comparison with typical and high-strength steels.

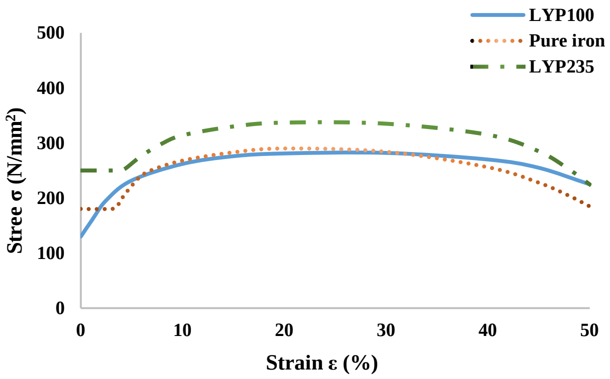


Fig. 1 A comparison of the stress-strain diagrams of pure iron and two low yield steels (Data source: Yamaguchi and Takeuchi 1998).

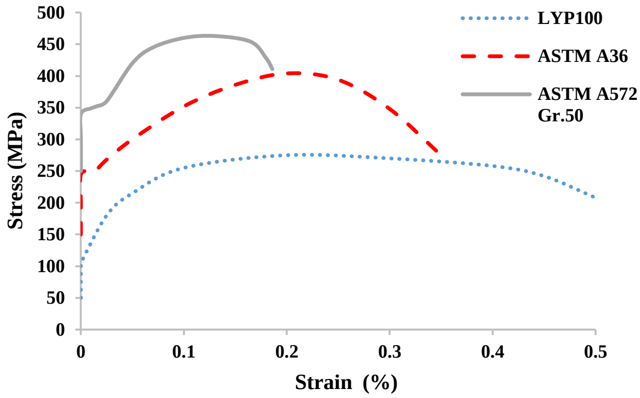


Fig. 2 Stress-strain diagrams of LYP100, typical (A36) and high-strength (A572) steels (Data source: Zhang and Zirakian 2015)

Fig. 1 (Yamaguchi and Takeuchi 1998) shows a comparison between the stress-strain diagrams of pure iron and two kinds of low-yield-strength steel. Furthermore, the stress-strain diagrams of three types of steel (ASTM A572 Gr.50 (σ_y=345 MPa), ASTM A36 (σ_y=250 MPa) and LYP100 (σ_y=100 MPa)) are shown in Fig. 2 (Zhang and Zirakian 2015).

2.2 Required seismic parameters

The main idea of the seismic design is that the structures under the effect of minor earthquakes should endure no damage and remain in the elastic range. However, although structural and non-structural damages could happen under the influence of major earthquakes, the structural overall stability should be preserved. The seismic resistance of regulations is usually less than that required to maintain the structural components within the elastic range under the effect of a severe earthquake. Therefore, the structures enter into the inelastic range during moderate or major earthquakes and a nonlinear analysis is needed. On the other hand, due to the time consuming nature of nonlinear analyses as well as the simplicity of linear methods, a conventional analysis, based on linear analysis of structures, is undertaken with reduced earthquake forces. Reducing the required elastic strength of structures is carried out by means of the strength reduction factor.

On the other hand, in the seismic design of structures, the nonlinear displacement can be estimated by multiplying a displacement amplification factor by the displacement obtained from a linear elastic analysis of the structure. Therefore, an investigation into the behavior factor, displacement amplification factor and the effective parameters influencing the aforementioned seismic response modification factors for various types of structural systems is important.

2.2.1 Ductility factor (μ)

The ability of a structure to withstand large inelastic displacements before rupturing is called ductility. According to the ATC-24 (1992), the ductility factor of a system is obtained from Eq. (1)

$$\mu = \frac{\Delta_{\max}}{\Delta_y} \tag{1}$$

where according to Fig. 3, $\Delta_{\max}$ is the maximum inelastic displacement that the system can tolerate and Δ_y is the displacement at the yield point (Elnashai and Mwafy 2002).

2.2.2 Over-strength factor (Ω_0)

The ratio of base shear of a structure at the structural collapse level (V_y) to the base shear at the first plastic hinge (V_s) is called the over-strength and obtained from Eq. (2) (Kurban and Topkaya 2009)

$$\Omega_0 = \frac{V_y}{V_s} \tag{2}$$

The over-strength factor, obtained from Eq. (2), is based on the nominal properties of materials. This factor is influenced by the redistribution of internal forces, strain hardening, members' oversize and the effect of non-structural members. The factor is also influenced by the fact that the material strength may be higher than that considered in the design process. The relationship between the structural over-strength (Ω_0) and the over-strength based on the nominal properties of materials (Ω_n) is given according to the Eq. (3) (Kurban and Topkaya 2009, Mahmoudi and Zaree 2011)

$$\Omega_0 = \Omega_n R_1 R_2 \ldots R_n \tag{3}$$

where R_1, …, R_n are parameters that are used to account for the actual material properties such as difference between the actual static yield strength and nominal yield strength, strain rate effect during an excitation, etc. (Kurban and Topkaya 2009).

In the AISC code (2005), the ratio of the 'expected yield stress' to the 'specified minimum yield stress' has been given for different types of steel as one of the effective factors on the structural over-strength. This ratio for the ASTM A36, ASTM A572 Gr42 and ASTM A572 Gr50, is equal to 1.5, 1.3 and 1.1, respectively.

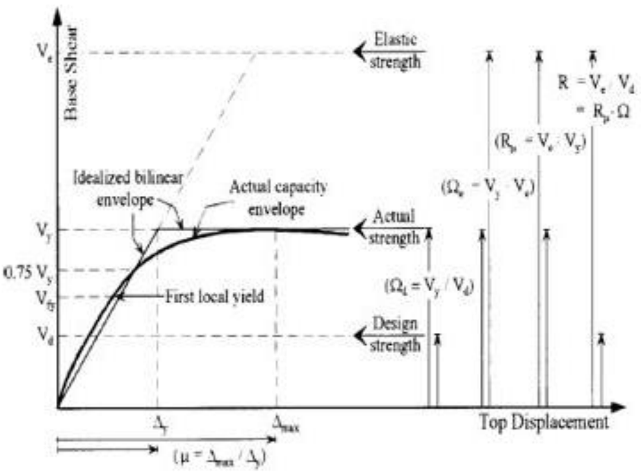


Fig. 3 Idealization of a pushover curve to an elastic-perfectly-plastic curve (Elnashai and Mwafy 2002)

Also, to consider the difference between the actual static yield strength and nominal yield strength, a coefficient equal to 1.05 is applied (Mahmoudi and Zaree 2011). As a result, the structural over-strength, can be calculated using Eq. (4)

$$\Omega_0 = 1.155\Omega_n \tag{4}$$

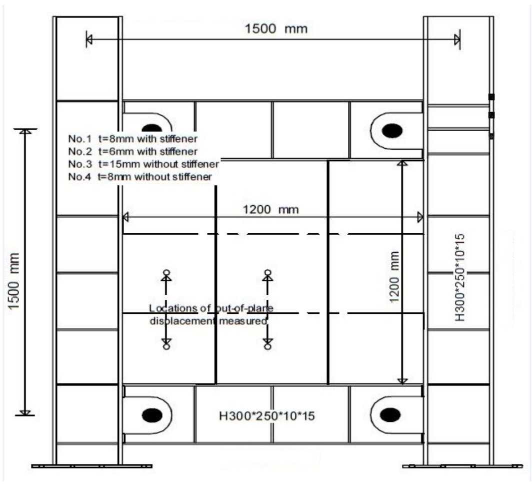
2.2.3 Ductility reduction factor (R_μ)

Ductility reduction factor of a structure, with a fundamental period of T, is defined as the ratio of the 'elastic base shear' to the 'yield base shear'. The relationship between the ductility factor and ductility reduction factor has been studied by different researchers, and several relationships have been proposed (Borzi and Elnashai 1999). Newmark and Hall (Borzi and Elnashai 1999), Nassar and Krawinkler (Borzi and Elnashai 1999) and Miranda and Bertero (1994) have proposed the most important relationships for the ductility reduction factor. These relationships have been used in the present study.

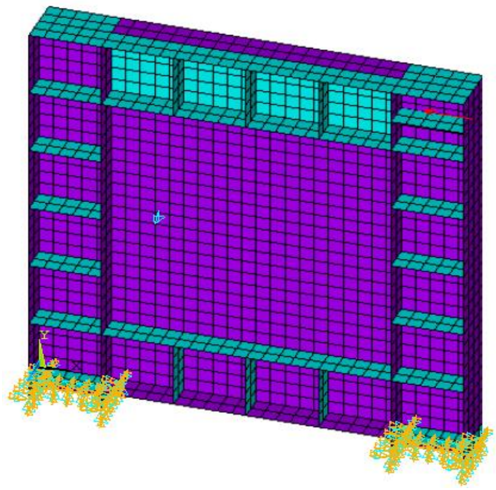
2.2.4 Behavior factor (R)

Reducing the resistance of a structure from the required elastic strength, is carried out using a strength reduction factor. For this purpose, required seismic forces for an elastic design of a structure are obtained from a linear spectrum, which depends on the natural period of the structure and the soil type at the site. To consider the effects of non-linear behavior, energy absorbed by hysteresis behavior, damping and the over-strength of structures, the elastic force is reduced to a design force, by means of a response modification factor (R). Hence, due to the nonlinear behavior of structures, this factor is applied to reduce the forces. For the ultimate strength method, the behavior factor is determined according to Eq. (5) (Kurban and Topkaya 2009)

$$R = R_\mu \Omega_0 \tag{5}$$



(a) Experimental test (Data source: Chen and Jhang 2011)



(b) Finite element sample

Fig. 4 Details of the 1-story steel plate shear wall with low yield strength, tested by Sheng-Jin Chen and Chyuan Jhang, and the finite element model

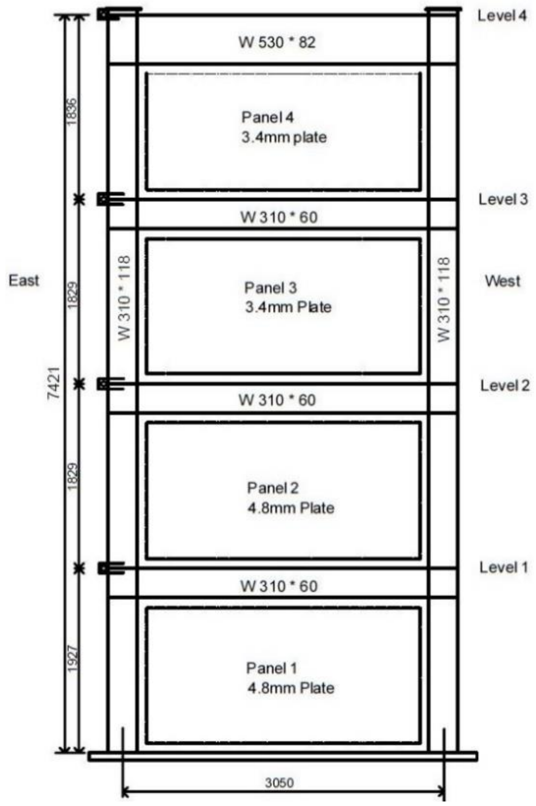
2.2.5 Displacement amplification factor (C_d)

In the seismic design of structures, the non-linear displacement caused by earthquakes, can be estimated by displacement amplification factor (C_d), considering the linear displacements of structures, obtained from a linear analysis. The displacement amplification factor in an ultimate strength method can be calculated using Eq. (6) (Kurban *et al*. 2009). According to Fig. 3 (Elnashai *et al*. 2002), Δ_{max} is the maximum non-linear, and Δ_s is the displacement corresponding to the first yield of the structure

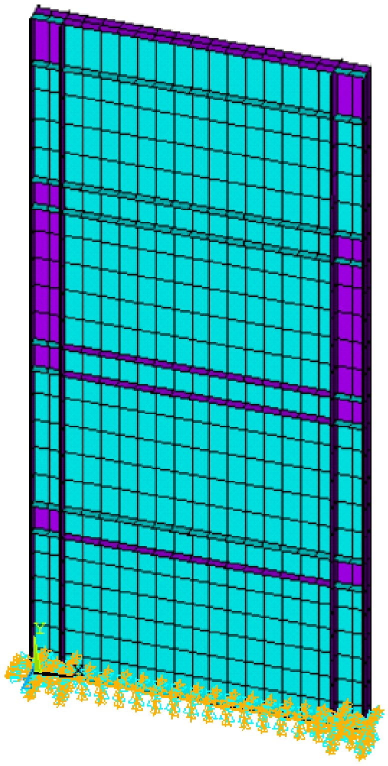
$$C_d = \frac{\Delta_{max}}{\Delta_s} = \mu\Omega_0 \tag{6}$$

3. Calibration of the finite element models

In order to evaluate the accuracy of the finite element modeling, two experimental specimens (i.e., the 1-story steel plate shear wall with low yield strength (Fig. 4), tested by Sheng-Jin Chen and Chyuan Jhang (2011) and the 4-



(a) Experimental test ((Data source: Driver *et al*. 1997))



(b) Finite element sample

Fig. 5 Details of the 4-story steel plate shear wall, tested by Driver et al. and the finite element model

story steel plate shear wall (Fig. 5), tested by Driver *et al*. (Driver *et al*. 1997)) have been used in the present study.

The SHELL181 element has been used for modeling the beams, columns and web-plates of steel plate shear walls. The SHELL181 element is a 4-node element with 6 degrees of freedom (3 degrees for displacement and 3 degrees for rotation), for each node. Using this element, we can consider local buckling of beams and columns. It should be noted that, this element has the ability of applying large deformations and strains, together with buckling and plastic deformations.

In this study, beam-to-column connections and shear wall frames have been considered as rigid-connection and moment-frame, respectively. According to the AISC341 provisions, moment-frames should be used for designing of steel shear walls located in high seismic areas; therefore, all frames of the structures are considered as moment-frames and all connections are rigid.

All analyses have been carried out considering both geometric and material nonlinearities. In these analyses, the load displacement control method has been used. A comparison of the results derived from the numerical analyses with those from the laboratory tests, as illustrated in Figs. 6-7, shows that the numerical models are able to predict an acceptable stiffness and energy dissipation level under monotonic loading. Also, Fig. 8 shows the same shear buckling pattern for both the numerical model of the one-story steel plate shear wall with low yield strength and the experimental model (Chen and Jhang 2011).

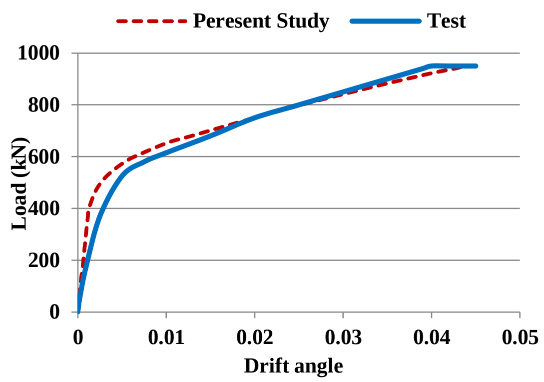


Fig. 6 Analytical (2011) and numerical results of Chen and Jhang 1-story sample

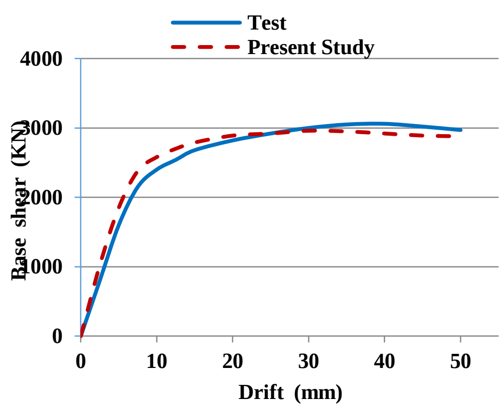
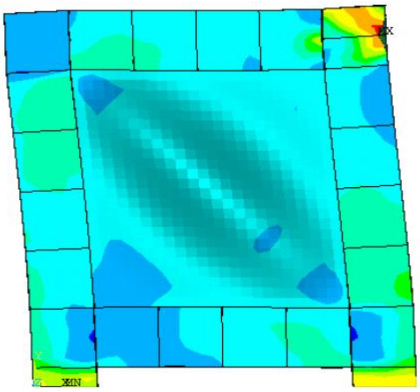


Fig. 7 Analytical (Driver *et al*. 1997) and numerical results of Driver *et al*. 4-story sample

(a) Experimental specimen of Chen and Jhang (2011)



(b) Analytical specimen

Fig. 8 Shear buckling of experimental and analytical specimens (Chen and Jhang specimen No. 4)

Therefore, it is found that the finite element modeling is reliable enough to undertake a numerical study for the determination of the seismic parameters of steel frames with low yield strength steel plate shear walls.

4. Numerical study

In the present study, frames with low yield strength steel plate shear walls, have been designed according to the AISC code. Also, all of the pushover analyses have been undertaken using ANSYS (v.16) (2015). Therefore, using a pushover analysis, the base shear-roof displacement curves have been extracted and the ductility (μ), ductility reduction (R_μ) and over-strength (Ω_0) factors have been computed. Using these parameters, the behavior (R) and displacement amplification (C_d) factors of the structures have been calculated.

4.1 Design of structures

To achieve the objectives of this study, five steel structures with low yield strength steel shear walls have been designed according to the provisions of the AISC (2005). The structures include 2, 5, 10, 14 and 18-story buildings to cover a variety of structures from low- to high-rise ones. Also, these building heights are very common in Iran, where this study has been done; therefore, these

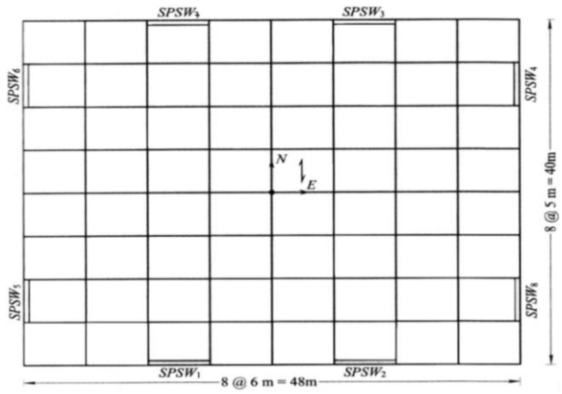


Fig. 9 Plan of the structures

building heights have been chosen. A same plan was used for all structures (Fig. 9) and SPSW1 steel plate shear wall was designed.

Steels used in structures, were LYP100 for the plates of the steel plate shear walls and A572.Gr50 for the horizontal (HBE) and vertical boundary elements (VBE), with the yield stress of 100 N/mm^2 and 500 N/mm^2, respectively. According to the plan of the structures, buildings have eight steel shear walls. To design the steel plate shear walls, the AISC provisions (2005) have been used. Also, the lateral and gravity loads have been considered based on the Iranian code of practice for the seismic resistant design of buildings (2007) and the sixth issue of the national regulations of Iran (2014), respectively. It should be noted that the LRFD method has been used in accordance with the ASCE (2006) and the behavior factor has been considered as 7 (R=7).

For the structures studied, according to the AISC code (2005), the connections of steel frames of shear walls, are considered as moment connections. Tables 1-5, show the sections used for the LYP steel plate shear wall systems with different heights. To introduce plate girder sections (Pl), four numbers have been used. The numbers indicate the web-height, web-thickness, flange-width and flange-thickness of a section, respectively. Also, t_w implies the plate thickness of a steel plate shear wall.

4.2 Pushover analysis

After the finite element modeling of the structures in ANSYS, pushover analyses were carried out. Different load patterns have been defined in the guidelines to implement the pushover analysis. In this study, according to the preliminary investigations carried out and due to the absence of significant differences between the results of different load patterns, the triangular lateral load pattern was selected. Then, the capacity curves of the structures were obtained and the parameters required for the calculation of the seismic response modification factors were extracted.

With regard to the effects of higher modes in the case of high-rise building structures, another pushover analysis, with a load pattern based on the FEMA356 (2000), was also carried out for the 14 and 18-story structures. A small difference about one percent was observed for the behavior factors obtained from the two analyses mentioned.

Fig. 10 shows the pushover curves obtained for the structures and Figs. 11-12 also illustrate the von Mises stresses and yielding of the plates of shear walls at the end of the analysis for the 2 and 10-story structures. It should be noted that the unit of stress in these figures is N/mm^2 and the images have been magnified four times in scale. As can be observed in Fig. 10, the structure's initial stiffness decreases by increasing the number of stories, and this reduction rate grows by increasing the number of stories.

The results show that the plates of LYP steel plate shear walls yield at first when the structures are subjected to a monotonic loading. Then, the yielding range increases and thereby the stress of the boundary elements increases, too, but they do not yield, yet. Also, it can be seen in Figs. 11-12 that by increasing the number of stories and consequently increasing the lateral loads and overturning moment, the role of boundary elements of steel shear wall system are more important. As a result, if the boundary elements are not strong enough to absorb interaction forces between the LYP steel plate and steel frame, the structure would not have appropriate stability under the applied loads.

4.3 Idealization of pushover curves

To extract the response modification factors, it is necessary to replace a pushover curve with an idealized curve. For this purpose, various methods have been proposed. One of the most common methods is the replacement of the pushover curve with an elastic-perfectly-plastic curve. Fig. 3 (Elnashai and Mwafy 2002) shows a schematic diagram of the mentioned method, which has been used in this study.

In the present study, in order to judge the results appropriately, five structures are considered. However, with regard to the selection of sample structures in the range of short, medium and high-rise buildings, the results can be reliable. Analyses results indicate that the ductility, ductility reduction and over-strength factors of the LYP steel plate shear wall systems, decrease by increasing the number of stories. As a result, the behavior and displacement amplification factors of the systems mentioned also decrease. For each structure, the ductility reduction factor has been calculated using the three methods mentioned above. As a result, for each structure, three behavior factors have been obtained. Figs. 13-14 show the ductility and ductility reduction factors versus behavior factor of the structures obtained from the three methods, respectively. In all figures and tables, N-H, K-N and M-B, represent the Newmark-Hall, Krawinkler-Nassar and Miranda-Bertero methods, respectively. In figures, it is evident that, there is a

Table 1 Characteristics of the elements for the 2-story structure

Level	1	2
t_w (mm)	2	4
HBE	Pl500*16/280*24	Pl500*16/280*24
VBE	IPB$_v$ 320	IPB$_v$ 320

Note: Thickness of plates of HBEs are presented in mm

Table 2 Characteristics of the elements for the 5-story structure

Level	1	2	3	4	5
t_w (mm)	8	8	6	5	3
HBE	Pl500* 16/280*24	Pl500* 16/280*24	Pl500* 16/280*24	Pl500* 16/280*24	Pl500* 16/280*24
VBE	IPB$_v$ 360	IPB$_v$ 360	IPB$_v$ 360	IPB$_v$ 320	IPB$_v$ 320

Note: Thickness of plates of HBEs are presented in mm

Table 3 Characteristics of the elements for the 10-story structure

Level	1	2	3	4	5	6	7	8	9	10
t_w (mm)	14	14	14	12	12	10	7	7	5	3
HBE	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 18/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26
VBE	IPB$_v$ 700	IPB$_v$ 700	IPB$_v$ 700	IPB$_v$ 700	IPB$_v$ 700	IPB$_v$ 650	IPB$_v$ 650	IPB$_v$ 650	IPB$_v$ 600	IPB$_v$ 600

Note: Thickness of plates of HBEs are presented in mm

Table 4 Characteristics of the elements for the 14-story structure

Level	1	2	3	4	5	6	7	8	9	…
t_w (mm)	18	18	18	16	16	16	14	14	12	
HBE	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 18/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	
VBE	IPB$_v$ 900	IPB$_v$ 900	IPB$_v$ 900	IPB$_v$ 900	IPB$_v$ 900	IPB$_v$ 800	IPB$_v$ 800	IPB$_v$ 800	IPB$_v$ 700	
Level	10	11	12	13	14					
t_w (mm)	10	7	7	4	4					
HBE	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26					
VBE	IPB$_v$ 700	IPB$_v$ 700	IPB$_v$ 650	IPB$_v$ 650	IPB$_v$ 650					

Note: Thickness of plates of HBEs are presented in mm

Table 5 Characteristics of the elements for the 18-story structure

Level	1	2	3	4	5	6	7	8	9	…
t_w (mm)	20	20	20	20	18	18	18	18	15	
HBE	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 18/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	
VBE	IPB$_v$ 1000	IPB$_v$ 1000	IPB$_v$ 1000	IPB$_v$ 900	IPB$_v$ 900	IPB$_v$ 900	IPB$_v$ 900	IPB$_v$ 900	IPB$_v$ 800	
Level	10	11	12	13	14	15	16	17	18	
t_w (mm)	15	15	12	12	9	9	9	6	6	
HBE	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	Pl550* 16/300 *26	
VBE	IPB$_v$ 800	IPB$_v$ 800	IPB$_v$ 700	IPB$_v$ 700	IPB$_v$ 700	IPB$_v$ 700	IPB$_v$ 650	IPB$_v$ 650	IPB$_v$ 650	

Note: Thickness of plates of HBEs are presented in mm

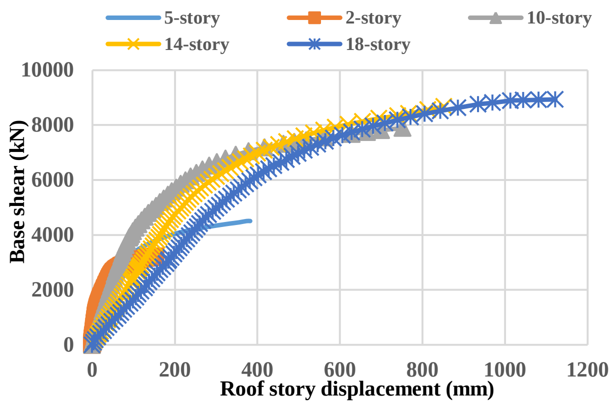


Fig. 10 Pushover curves of the structures under monotonic loading

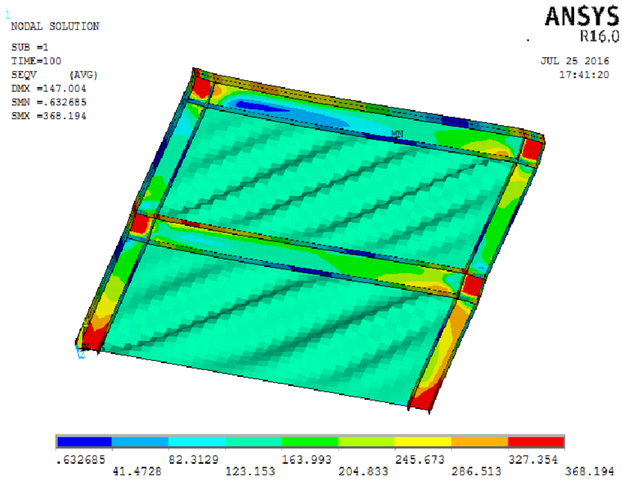


Fig. 11 A plot of von Mises stress response of 2-story structure at the end of the analysis, (stress unit is N/mm^2)

little difference between the results of the three methods. However, the results shown in Fig. 13, indicate that less values are obtained for the behavior factor by using the Newmark-Hall method between the three methods. According to the results given in Fig. 14, it seems that other parameters, in addition to the ductility factor, influence the reduction factor. Because, if other parameters will not be efficient in the calculation of the reduction factor, by changing the way of idealization of the pushover curves (in these three methods), we will not reach different values for the reduction factor.

5. Discussion of results

In the present study, in order to judge the results appropriately, five structures are considered. However, with regard to the selection of sample structures in the range of short, medium and high-rise buildings, the results can be reliable. Analyses results indicate that the ductility, ductility reduction and over-strength factors of the LYP steel plate shear wall systems, decrease by increasing the number of stories. As a result, the behavior and displacement amplification factors of the systems mentioned also decrease. For each structure, the ductility reduction factor

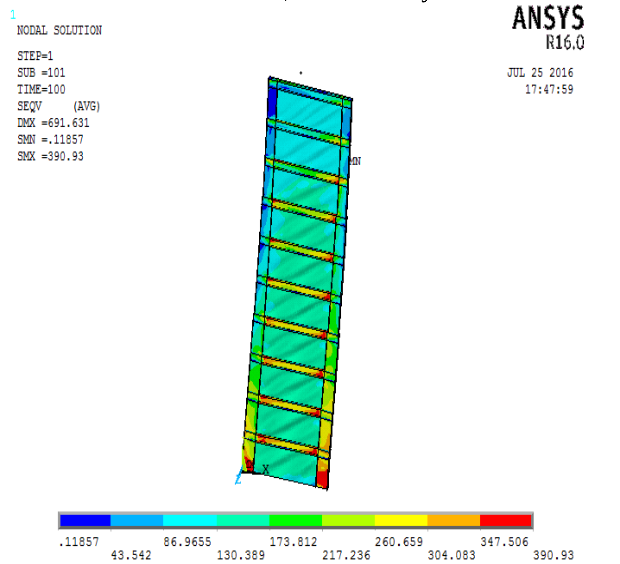


Fig. 12 A plot of von Mises stress response of 10-story structure at the end of the analysis, (stress unit is N/mm^2)

has been calculated using the three methods mentioned above. As a result, for each structure, three behavior factors have been obtained. Figs. 13-14 show the ductility and ductility reduction factors versus behavior factor of the structures obtained from the three methods, respectively.

In all figures and tables, N-H, K-N and M-B, represent the Newmark-Hall, Krawinkler-Nassar and Miranda-Bertero methods, respectively. In figures, it is evident that, there is a little difference between the results of the three methods.

However, the results shown in Fig. 13, indicate that less values are obtained for the behavior factor by using the Newmark-Hall method between the three methods. According to the results given in Fig. 14, it seems that other parameters, in addition to the ductility factor, influence the reduction factor. Because, if other parameters will not be efficient in the calculation of the reduction factor, by changing the way of idealization of the pushover curves (in these three methods), we will not reach different values for the reduction factor.

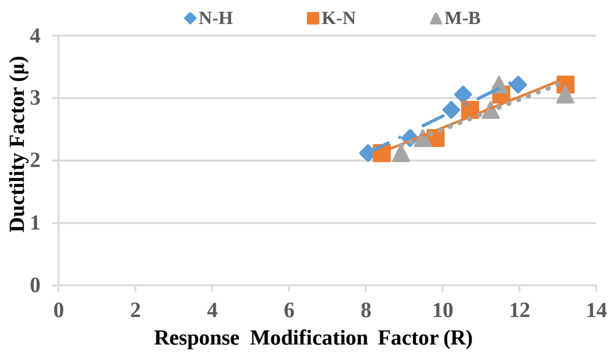


Fig. 13 Ductility factors versus the behavior factors of the structures

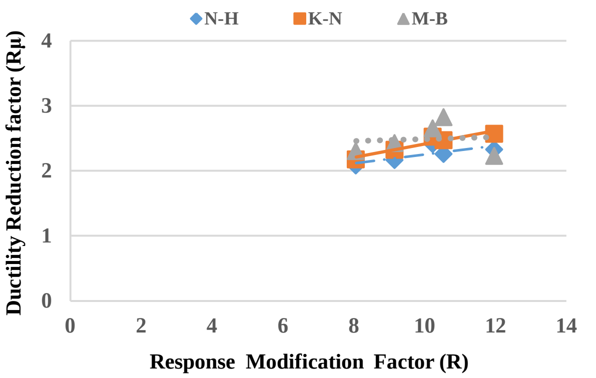


Fig. 14 Ductility reduction factors versus the behavior factors of the structures

A summary of the results obtained for the behavior and displacement amplification factors and also the effective parameters influencing them are listed in Table 6.

Also, the mean values for the calculated parameters have been shown in Table 7. The average values of the behavior and displacement amplification factors for the steel frames with low yield strength shear walls are 7.35 and 8.44, respectively. As expected, ductility, ductility reduction and over-strength factors of the systems, examined in this investigation, decrease with the increase in the number of stories, thus the behavior and displacement amplification factors decrease, as well. Decrease in the displacement amplification factor with the increase in the number of stories is evident in Table 7.

The results of the study are compatible with the previous studies in this field, such as Kurban and Topkaya (2009), ASCE (2006) and Kim and Choi (2005). A comparison of the behavior, ductility reduction and displacement amplification factors of the structures, obtained from the N-H, K-N and M-B methods, is presented in Figs 15-17. According to the results obtained, values of 3.0, 7.35 and 8.44 can be suggested for the over-strength, behavior and displacement amplification factors for the LYP steel plate shear wall systems, respectively, which are larger than those proposed by the AISC code (2005) for steel plate shear wall systems, for the LRFD method (Ω_0=2, R=7 and C_d=6). This may reflect the fact that, in comparison with a conventional steel plate shear wall system, a structure with LYP steel plate shear wall system has a pronounced difference between the formation of the first plastic hinge and yield point. It can be one of the benefits of the LYP systems under the effect of the lateral loads. However, further investigations are needed to appropriately judge.

In most regulations, the behavior and displacement amplification factors are independent of the height and the period of structures; therefore, constant values are suggested for these factors in the AISC (2005) and the Iranian seismic code (2007). However, according to Chopra (2007), in some of the regulations such as Eurocode8 and the Mexico code, the behavior factor is considered to be dependent on the period of structures.

The results obtained in the present study and other similar studies (e.g., Kurban and Topkaya 2009, ASCE

Table 6 Summary of the results obtained for the LYP systems

Structure	2-story	5-story	10-story	14-story	18-story
Period (sec)	0.182	0.362	0.609	0.783	0.946
Ductility factor	3.210	3.051	2.804	2.352	2.116
Ductility reduction factor ($R_{\mu(N-H)}$)	2.328	2.258	2.410	2.167	2.082
Ductility reduction factor ($R_{\mu(K-N)}$)	2.568	2.469	2.526	2.325	2.174
Ductility reduction factor ($R_{\mu(M-B)}$)	2.231	2.826	2.651	2.426	2.303
Nominal overstrength factor	3.091	2.807	2.553	2.541	2.328
Structural overstrength factor	3.570	3.242	2.948	2.934	2.689
Behavior factor ($R_{(N-H)}$)	8.311	7.320	7.105	6.358	5.598
Behavior factor ($R_{(K-N)}$)	9.168	8.004	7.446	6.822	5.846
Behavior factor ($R_{(M-B)}$)	7.965	9.162	7.815	7.118	6.193
Displacement amplification factor	11.459	9.891	8.266	6.901	5.690

Table 7 Mean value of the calculated parameters

Parameter	μ	$R_{\mu(N-H)}$	$R_{\mu(K-N)}$	$R_{\mu(M-B)}$	Ω_n	Ω_0	$R_{(N-H)}$	$R_{(K-N)}$	$R_{(M-B)}$	C_d
Mean value	2.71	2.25	2.41	2.48	2.66	3.08	6.94	7.46	7.65	8.44

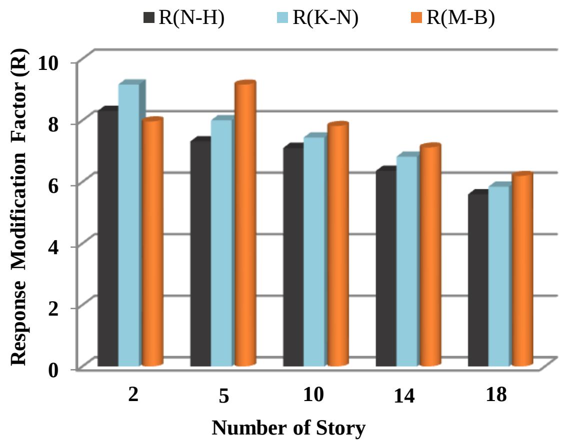


Fig. 15 Behavior factor of the structures

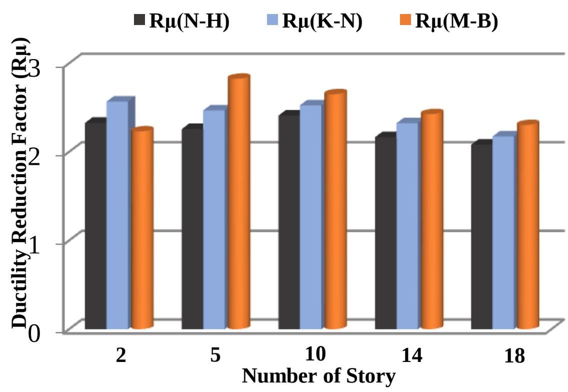


Fig. 16 Ductility reduction factor of the structures

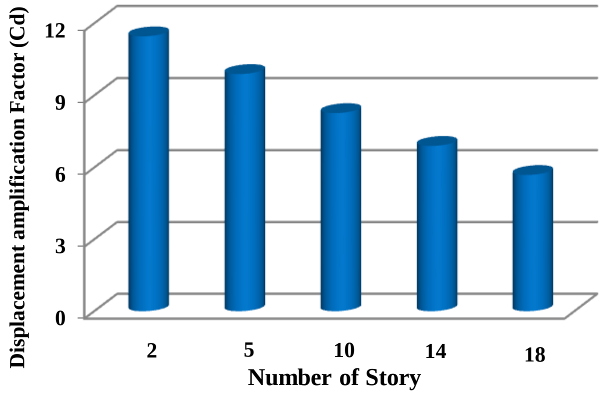


Fig. 17 Displacement amplification factor of the structures

2006, Kim and Choi 2005) indicate the fact that the behavior and displacement amplification factors for each type of structures are related to the height and they, consequently, decrease by increasing the height and period of the structure. Therefore, it seems that it is necessary to introduce the behavior and displacement amplification factors of structures, considering the height and period of structures.

6. Conclusions

Regarding the design and analysis of the structures with LYP steel plate shear walls, and evaluation of the behavior and displacement amplification factors as well as the effective parameters influencing these factors, the following conclusions can be obtained:

• It is observed that by increasing the number of stories as well as by increasing the lateral loads, the boundary elements of steel plate shear wall systems will be more important and thus, if these elements are not strong enough to absorb forces resulting from the interaction of LYP steel plate with the frame, the structure will never be able to reach its maximum bearing capacity.

• Applying monotonic loads, the LYP steel plate shear wall yields at first and then the yielding range increase; thereby, the stress of boundary elements also increases.

• The results obtained from the present study demonstrate that the behavior factor for LYP steel plate shear wall systems, like any other types of structures, is related to the height and decreases by increasing the height and period of the structures.

• By increasing the number of stories, the ductility, ductility reduction and over-strength factors of the LYP steel plate shear wall systems decrease. Consequently, the behavior and displacement amplification factors decrease, too. In this study, the mean values for the over-strength, behavior and displacement amplification factors of the structures can be suggested as 3.08, 7.35 and 8.44, respectively. These results are 154%, 105% and 140% larger than those proposed by the AISC for the over-strength, behavior and displacement amplification factors, respectively, in the case of steel plate shear wall systems.

• The results, obtained in the present study and other similar studies, indicate that the behavior and displacement amplification factors decrease by increasing the height and period of the structures. Therefore, it is necessary to introduce the behavior and displacement amplification factors of structures in such a way that they are dependent on the height and period of structures.

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AT

Nomenclature

T	Period (sec)	C_d	Displacement amplification factor
E	Elastic Young's modulus	t_w	Maximum inelastic displacement (mm)
μ	Ductility factor	Δ_{max}	Maximum inelastic displacement (mm)
Ω_0	Over-strength factor	Δ_y	Yield point displacement (mm)
R_μ	Ductility reduction factor	V_y	Mechanism step resistance
R	Behavior factor	V_s	First plastic hinge resistance