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Period Calculation of Thin Steel Plate Shear Walls and the Effect of Connection Type and Degree of Opening on it

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Abstract

Since four decades ago, Steel plate shear wall-SPSW- is a new system resistant to against lateral loads, which has attracted the attention of researchers and designers because of its numerous advantages. The behavior coefficient which has direct dependence on the reflection coefficient is in one way or another based on the period or the natural period of vibration of the resistant system, and considering the short life of this system, there has been no serious study on its period, and in ASCE 7 code, this system has been referred to other systems. In this study, the natural period of vibration of different frames and variant number of floors is examined for thin steel plate shear wall, the results of linear and nonlinear analyses -are compared, and then empirical relationships are presented. Moreover, the effects of connection type and different openings on the period degree of this system were being examined.

The results supported the appropriateness of the offered empirical relationship in ASCE 7 code for short and midframes of thin steel plate shear wall in ASCE 7 code; however, it seems that the empirical relationship of tall frames should be increased twice.

Keywords: Steel plate shear wall, natural period of vibration, opening, simple and rigid connections

Introduction

Steel plate shear wall is a lateral resistant system to the forces arising from earthquake or wind, which has attracted the attention of researchers and designers to use this element to design the buildings To design a lateral resistant system using the behavior coefficient method, sheer base is calculated based on the ratio of the effective weight of the building in, which the coefficient of reflection of the building is dependent on the natural vibration period. Basically, calculation of the natural period of vibration of the building prior to the building design is not possible; therefore, earthquake codes suggest an empirical relationship to be used in the preliminary design of the structure [1-5]. These empirical relationships depend on the type of structural system, its dimensions, and used materials. Consequently, mentioned relations suggest conservative values as a function of the structure's height. This relationship has been expressed in the codes of NBCC [6], ASCE [7], and the Euro Code [8] and Iran earthquake standard 2800 [9] as follows Equation (1):

$$T = C.h^{x} \ldots\ldots (1)$$

Where C and ^{x} are cons values and denotes the height of the structure. These values are suggested to be 0.5 and 0.75 for steel plate shear wall systems, respectively, NBCC code has considered that the natural vibration period should not exceed two times as much as the amount of empirical method. This value is limited to 1.4 times and 1.7 for high seismic zones and for low seismic respectively.

Many studies have been done to calculate the natural vibration period of other lateral resistant systems, but may not research had not been conducted on steel plate shear walls. Topkaya and Ghorban [10] have studied of this mentioned system and have suggested mathematical relationships to calculate the natural vibration period. Moreover, Anjan, et al. have presented the empirical relationship of T=0.03h for natural vibration period of steel plate shear wall system [11].

In this study, the natural period of short, mid and tall models of thin steel plate shear wall system with and without opening, having a simple and rigid beam to column connections was examined using finite element linear and non-linear analyses.

Specifications of structural models

Figure 1 shows a building with a five spans rectangular plan in which the span in the middle is filled with SPSW, loaded under live and dead gravity load of 5.5 and 2 kN/m^2 respectively and has been designed in two forms, i.e. simple and rigid beam to column connections in accordance with Iran Steel code [12] and earthquake standard 2800 for different heights. The height of each floor is 3 meters, and three as well as seven-floor frames are considered as a symbol of short buildings, ten-floor frame as mid building and fifteen-floor frame as the tall building (Figure 1).

Steel shear walls were designed based on plate frame interaction method in which maximum capacity of plate is used [13]. The behavior coefficient of the system is considered 8 in accordance with what suggested in AISC code, and sections of beams, columns and plates were calculated for 2 forms of simple beam to column connection and rigid beam to column connection. In all beams, columns and plates, ST37 steel with elasticity module of 2.06 GPa was used.

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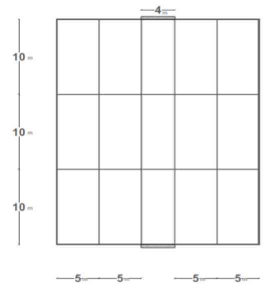

Figure 1: A building with a five spans rectangular plan

Calculation of natural vibration period of the frames in linear analysis without buckling of plates

Models were designed with 3, 7, 10 and 15 floors by using Abaqus finite element software [14], and the special value in of elastic form was analyzed for the purpose of calculating their period in two forms of the simple beam to column connection and rigid beam to column connection. The results of this study are shown in Table 1. In modeling structure's constituents (beam, column, and wall), a four-node SHELL element reduced of one point (S4R) was used. A lumped mass was considered the top of the column in each floor, and the ceiling in each floor was considered rigid.

No. of Stories	Period of wall systems(t)	Period of dual systems(t)
3	0.2255	0.2108
7	0.7708	0.6968
10	1.338	1.1248
15	2.635	2.4277

Table 1: Period of structures in linear analysis.

Comparison of natural vibration period of the frames using finite element method and analytical relationships

The Equation (2) shows the differential equation of calculating the period of steel shear walls taking into account shear and rotational deformations of the plate [15,16].

$$m\frac{\partial^2 u}{\partial t^2} + EI_W\frac{\partial^4 u}{\partial x^4} - mr^2\left(1 + \frac{E}{KG}\right)\frac{\partial^4 u}{\partial x^2 \partial t^2} + \frac{m^2 r^2}{KGA_W}\frac{\partial^4 u}{\partial t^4} = 0$$
...... (2)

Where m is mass multiplied by height unit, E- elasticity module, G- shear module, Iw - second moment of area- neutral axis ratio, K- shear constant, A_{w}- wall area, r- gyration radius of wall's section, x- the distance from origin area and u, lateral movement of the wall.

In this relationship, SPSW is assumed to be a cantilever beam.

This relationship was calculated by Soul-Dunkerley with adequate approximation and based on the period of SPSW [17].

Thus, the sheer frequency of a frame with simple joints is calculated by Equation (3):

$$f_s = r_f\frac{1}{4H}\sqrt{\frac{KGA_w}{m}} \text{ ...... (3)}$$

Where f_s is the natural rotational frequency of the wall resulting from shear deformation, H- structure's height, and r_f - a coefficient used by Zalka spreading lumped mass multiplied by the building's height [18].

In addition, the natural rotational frequency due to the bending of a resistant system (f_b) can be calculated as:

$$f_b = r_f\frac{0.5595}{H^2}\sqrt{\frac{EI_W}{m}}$$

Where f_s is the natural rotational frequency of the wall resulting from shear deformation, H- structure's height, and r_f - a coefficient used by Zalka spreading lumped mass multiplied by the building's height [18].

In addition, the natural rotational frequency due to the bending of a resistant system (f_b) can be calculated as equation (4):

$$f_b = r_f\frac{0.5595}{H^2}\sqrt{\frac{EI_W}{m}} \text{ ......(4)}$$

Using the relationships 3 and 4, the natural vibration period of steel shear wall with simple joint is calculated as shown in the Equation (5):

$$T_W \approx \sqrt{\frac{1}{f_b} + \frac{1}{f_s}} \text{ ...... (5)}$$

Moreover, natural vibration period of steel shear wall with rigid connection (T_{sys}) is calculated using the Equation (6):

$$T_{sys} = \frac{2\pi}{\lambda_{sf}^2 r_f}\sqrt{\frac{m}{EI}} \text{ ...... (6)}$$

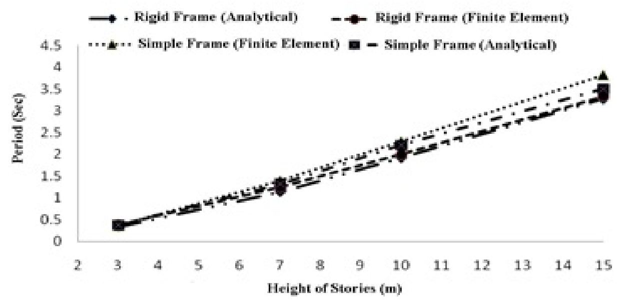

Figure 2: Comparison of period of steel shear wall with simple and rigid connections in two methods of finite element and analytical.

Where λ_{sf} denotes the frame's special value. The natural vibration period of models was calculated by using finite element method and analytical method, and the results were compared as the graph shows, the results have adequate consistency Figure 2.

Calculation of natural vibration period of the frames in non-linear analysis and buckling of the plates

To design SPSW, plates with low thickness were used to generate a diagonal tension field, field; their extraordinary back buckling capacity is used during lateral loading.

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Therefore, linear analysis of models, given non-buckling of the plates in the previous stage, must be done once more using non-linear analysis, given the buckling of the plates and their results are compared. To buckle the plates, Initial imperfections was created in the middle of the plates.

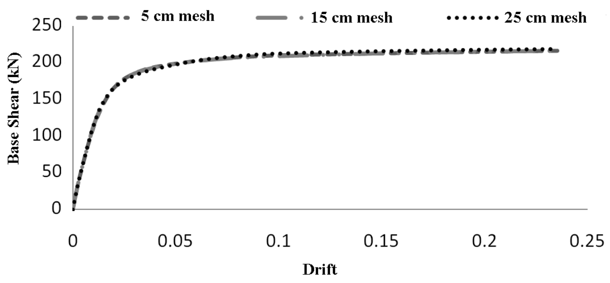

Figure 3: Sensitivity analysis of meshing.

Based on the number of elements and the time of calculation, investigation of the accuracy of results needs to have an acceptable size element to nonlinear analyses.

Therefore, models were designed by 5, 15 and 25 cm size element, and the first mode period amount and their basic shear were compared. Figure 3 shows the results of the basic shear based on drift in the frame with 3 floors given different mesh dimensions, Finally, to evaluate the accuracy the results, 15cm elements size were used.

To examine the effects of non-linearization and buckling of the plates on the frames' period, first, a special value of frequency was analyzed, and the movement of all floors was calculated by using the first mode.

Then, considering the movements in different drifts and steps, the period of different drifts was calculated analyzing frequency special value with the use of Lancsoz method.

Drifts within the range of 0.25% to 5% of the movement in the highest floor were applied to the models.

As it is shown in Table 2, the highest increase of in period arising from non-linearization and buckling of the plates was related to the model with three floors, whose amount in models with simple joint and fixed joint in different drifts is approximately calculated to 20 to 28%.

Since models enter a non-linear stage in an approximate drift of 0.25%, increase the ratio of a non-linear period to linear period is simulated 20% in the frame with three floors and 1 to 8% in other models.

The results in Table 2 show that the as the frame is higher, the effects resulting from non-linearization and buckling of the plates are less on the structure's period.

This is related to greater effects of bending mode comparing to a shear mode in tall structures. Furthermore, according to the results, non-linearization and buckling of the plates with a simple connection are more effective compared to the ones with rigid connection [19].

	Drift										
Model	**Connection**	0.25	0.5	0.75	1	1.25	1.5	1.75	2	2.5	5
3-story	Simple	1.201	1.212	1.22	1.227	1.232	1.236	1.24	1.244	1.25	1.279
	Rigid	1.209	1.22	1.228	1.234	1.239	1.243	1.247	1.2	1.257	1.284
7-story	Simple	1.088	1.101	1.107	1.122	1.141	1.162	1.185	1.198	1.201	1.262
	Rigid	1.085	1.095	1.1	1.11	1.12	1.131	1.139	1.145	1.153	1.172
10-story	Simple	1.056	1.066	1.069	1.074	1.085	1.104	1.117	1.129	1.152	1.126
	Rigid	1.032	1.045	1.049	1.056	1.062	1.068	1.071	1.073	1.073	1.071
15-story	Simple	1.036	1.043	1.045	1.046	1.053	1.054	1.055	1.059	1.073	1.121
	Rigid	1.017	1.025	1.029	1.03	1.031	1.033	1.035	1.036	1.037	1.033

Table 2: Ratio of non-linear period to linear period in different drifts of frames.

The effect of opening on period of thin steel plate shear wall

In order to examine the effect of opening on the period of SPSW with a thin plate, models consisting of to simple and rigid connections walls were designed.

These models have square openings of 10, 30, 50 and 80% of plate area and underwent linear and non-linear analyses.

Table 3 shows the ratio of a non-linear period to linear period in different degrees of opening and in the drift of 0.25% for different models. Since the frames enter the non-linear stage at the drift of 0.25%, the results in this drift are shown in (Table 3).

Model	**Degree of opening (%)**	**Simple connection**	**Rigid Connection**
3 story	10	1.272	1.276
	30	1.22	1.2379
	50	1.2688	1.1255
	80	0.9949	1
7 story	10	1.0967	1.0897
	30	1.1713	1.1834

	50	1.1135	1.1723
	80	1	1
10 story	10	1.0604	1.0283
	30	1.1052	1.007
	50	1.0055	1.0513
	80	1	1
15 story	10	1.0401	1.0181
	30	1.0834	1.0372
	50	1.0834	0.9999
	80	1	0.9999

Table 3: Ratio of non-linear period to linear period in drift of 0.25% of models with opening.

Table shows, the highest degree of increase in the period of the system due to non-linearization and existence of opening that, which is 27%, was reported in the model with 3 floors and 10% of opening.

The more the number of floors is, the less the amount of increase due to non-linearization and existence of opening is, in a way that this amount can be ignored in tall structures. Moreover, by increasing e in the amount of opening size, the ratio of the non-linear period to linear period is decreased declined.

In this way, the effects of non-linearization in short frames with a smaller percentage of the opening is more however, with by increasing an opening percentage and height of the wall, the amount of the structure's period is increased.

Determining empirical equation for calculating the period of thin steel plate shear wall

In order to determine the empirical relationship as to estimate of period of SPSW with thin plates, the results of the period in the frame with 8 floors with simple and rigid connections and also the results of the samples by Tapkaya and Ghorban [10] were reported in figure 4. In NBCC, ASCE and Iran Earthquake 2800 codes, it has been mentioned that the period value calculated using finite element method should not be more than the values of empirical relationships specified in the forsaid codes.

Therefore, the period value calculated using finite element method should not be more than two times in NBCC code, not more than 1.4 and 1.7 times for high seismic and low seismic zones respectively in ASCE 7 code, and not more than the value calculated using empirical relationships in Iran Earthquake 2800 code. (Figure 4) shows the respective values.

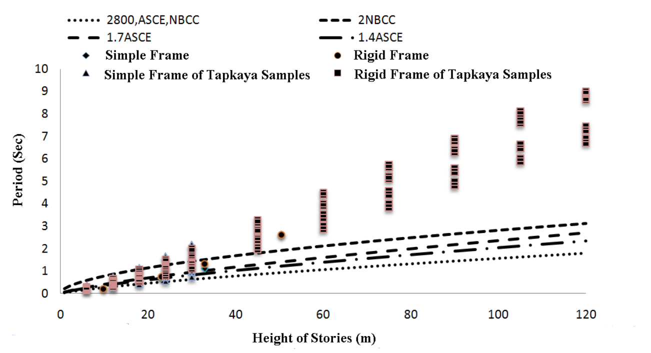

Figure 4: Comparison of period of thin steel plate shear wall frames with empirical relationships of codes.

As the figures show, the results of only 6 models out of 96 models have period values lower than code coefficient relationships, in other cases, it is more.

In addition, by increasing the height of the frames, the difference between the results from finite element method and code coefficient relationships become more.

The results from finite element method is divided into two ranges of short and mid frames and tall frames. (Figure 5 and 6) show the structure's period based on the floors in short and mid structures with rigid and simple connection respectively.

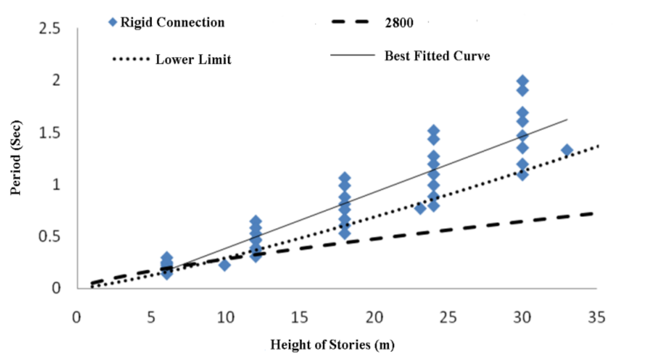

Figure 5: Period of short and mid frames versus height of stories in frames with rigid connection.

In addition, since SPSW with simple connection is not used in tall structures, (Figure 7) displays only the period of tall structures with rigid connection. In the foresaid figures, period value of each model, figure related to Iran Earthquake 2800 code, fitting curve and lower curve are shown.

Fitting the best curve into period results in proportion to the height, 3 relationships from 7 to 9 were presented for short and medium SPSW with simple connection, short and mid SPSW with rigid connection and tall SPSW with rigid connection:

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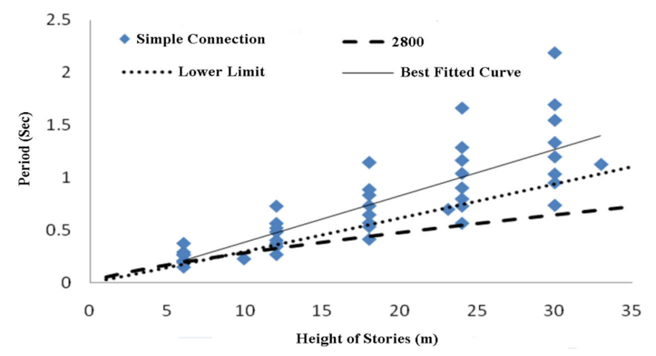

Figure 6: Period of short and mid frames versus height of stories in frames with simple connection.

$$T = 0.0336H^{1.0484}$$

$$T = 0.0223H^{1.224}$$

$$T = 0.0309H^{1.1627}$$

Relationships 10 to 12 also show lower period of these frames based on height respectively:

$$T_L = 0.0265H^{1.0484}$$

$$T_L = 0.0176H^{1.224}$$

$$T_L = 0.0273H^{1.1627}$$

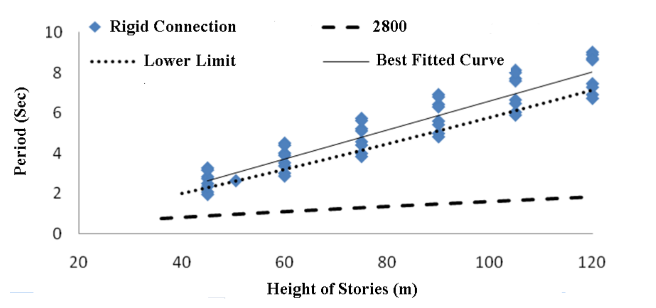

Figure 7: Period of tall frames versus height of stories in frames with rigid connection.

Since most codes consider structure's period a coefficient of the power 0.75 of the structure's height, the figures related to values of empirical relationships $T=0.05H^{0.75}$, offered by Iran Earthquake 2800 code for other systems, are shown in figure 5 to 7.

As the figures show, the mentioned relationship is an adequately conservative relationship for short and mid structures with both types of simple and rigid connections; however, by increasing the height of the frame, it will be too conservative in a way that it will lead to a profitable plan. Therefore, it is suggested that in tall structures (taller than 40 meters), having SPSW system with thin plates, instead of the coefficient 0.05 of code relationship, a coefficient of 0.1 is used. The relationship curve $T=0.1H^{0.75}$ is displayed in Figure 8.

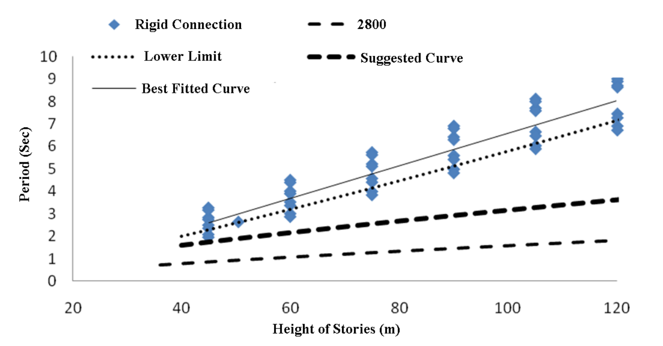

Figure 8: Suggested curve for period of tall structures with thin steel plate shear walls.

Connection Type:According to the studies, the shear walls of the corrugated steel have a lower degree of uniformity than the shear walls. It is also known in this study although, in low moves, the corrugated sheet has a high resistance, but the behavior of the shear wall with a flat sheet relative to the shear wall Corrugated is more stable. The energy absorption of the shear wall is smoother than the shear wall. So use a shear wall with Smooth sheet is recommended in areas with high seismic risk. Corrugated Sheet Wall to Flat Sheet The beginning of the periodic loading, the amount of energy absorbed more, But with increasing displacement, the energy absorption of the shear wall with The corrugated sheet is flattened less than the shear wall Therefore, in low displacement, a shear sheet of corrugated sheet Better than a smooth shear wall; So areas where Less earthquake is the use of a shear wall with a corrugated sheet The use of a smooth shear wall is preferable; but in Seismic areas, which are more likely to be displaced above, are recommended for flat shear walls. In fact the effects of non-linearization and buckling of plates on steel shear wall with rigid connection is less than those on steel shear wall with simple connection.

Conclusion

The natural period of thin steel plate shear wall frames in 3, 7, 10 and 15 story was calculated using two types of linear and non-linear analyses as well as plates buckling assumption. Non-linear analyses were performed under different drifts taking into account plates buckling assumption. The results revealed that the effect of non-linearization and buckling of plates on natural period of thin steel plate shear wall system drastically decreases upon increase in the frame's height, and can be ignored.

The results also showed that the effects of non-linearization and buckling of plates on steel shear wall with rigid connection is less than those on steel shear wall with simple connection.

Moreover, the effects of opening on period degree of thin steel plate shear wall were examined. The results revealed that upon increase in area percentage of opening and frame's height, the effects of non-linearization and buckling of plates on natural period of thin steel plate shear wall system drastically decrease and can be ignored.

Based on the results, it is suggested that in case of thin steel plate shear wall in two forms i.e. with simple connection and rigid connection, it is proper to use $T=0.05H^{0.75}$ relationship to predict the period of short and mid frames up to 40 meters in height and use

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$T=0.1H^{0.75}$ to predict the system's period for frames taller than 40 meters.

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