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INVESTIGATION THE SEISMIC PERFORMANCE LEVEL OF COLD- FORMED STEEL FRAMES BRACED WITH SHEATHING PANELS

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ABSTRACT

Today for seismic behavior of structures, types of nonlinear static and dynamic analyzes are rapidly expanding. In this study, the nonlinear static method is used to evaluate the performance based on cold formed steel frames. Since these structures are typically performed with low levels, the first mode of vibration is dominate and evaluation of these structures with nonlinear static method, moreover reducing the analysis time, analytical results are obtained almost exact. In this research, after modeling 38 frames of cold formed steel with four different types of sheathing: OSB (Oriented strand board), DFP (Douglas fir plywood), CSP (Canadian softwood plywood) and GWB (Gypsum wall board) and considering different thickness of sheathings, target displacement and lateral drift ratio are calculates for different performance assessments. The results of the study in all the specimens indicates that the linear behavior of the structure is responsive to seismic design and performance point is elastic in the structural behavior.

Keywords: Cold-formed steel frame; sheathing panel; non-linear analysis; seismic performance assessment.

1. INTRODUCTION

Constructions prefabricated by Light Steel Frames (LSF) are a type of industrial constructions widely considered by engineers due to their exact monitoring in construction stage, easy implementation and also lightness. On the other hand, cold formed steel has many advantages compared to other common construction materials such as masonry, wood and also warm formed steel structures. Typically, cold formed steel constructions (CFS) consist of structural shear wall panels (SWP), load bearing and non-load bearing wall panels, floor panels and roof panels. SWP panels are made up of studs and cracks and covered by structural cover plates with screw connection in two- or one side. Nowadays,

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engineers review and analyze the general behavior of constructions made of cold formed steel frames usually through finite element analysis. In the present study, the Conventional Finite Element Analysis (CFEA) is mentioned.

Generally, the wall of cold formed steel frames is formed from a vertical structural element called stud. In order to brace such walls, sheets or cover plates are applied which are connected using closed belts to the structural system (Fig. 1a). These cover plates are made up of plywood or chalk boards (Fig. 1b). The connection between studs and cover plates is done using closed belts that are connected to the structure using automatic screws. In order to provide the lateral resistance of cold formed steel studs, cover plates structural system and closed belts are applied. Thus it is required to evaluate lateral stiffness and resistance of this structural system to determine its role in lateral inhibition of cold formed steel frames. Influenced by different mechanisms such as lateral deformation of stud, and force between stud and cover plate, connection of stud and closed belts suffers from tilting, tilting with curveting and tilting with curveting and stretching through cover plate.

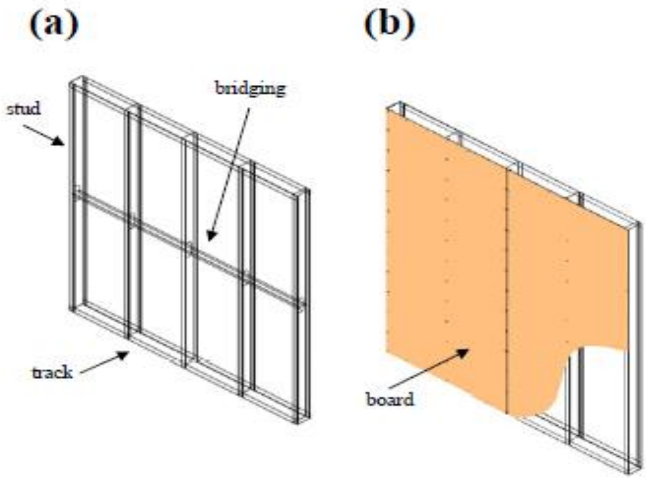

Figure 1. Sheathed wall design with stud braced only by sheathing

2. PURPOSE OF THE STUDY

Performance-Based Design (PBD) targets to design constructions fulfilling the performance targets specified by designer with the professional regulations of construction. Practically, assessment of PBD follows no fixed procedure and since it depends on the property of construction and the wishes of designer, it also uses different procedure for all structural systems. The complexity of application for each method varies; therefore, some methods may not be appropriate to analyze certain types of structures. In one of the functional targets, a seismic hazard and a performance level are combined with each other. The performance level is a measure of the maximum allowable damage for a construction in which the damage level is associated with the cost of structure. In assessing PBD, each selected targets that according to which the construction should be designed, becomes a palpable parameter so that it can be used in the seismic analysis. Also, Grierson *et al* [1] presented a discussion on the advantages and disadvantages of two approaches based on spectrum and displacement. Although their discussion focused on constructions with steel frame, the principles of assessing PBD mentioned can also be used for CFS constructions. Pushover analysis is simpler than dynamic analysis; however, using it, we can obtain exact results for structures with first dominated vibration mode [2]. In CFS constructions with low and medium height and also with lateral high stiffness, the expected frame structures should have first dominated vibration mode. So, it can be expected that fairly detailed results are obtained from pushover analysis. Also, in this study, limiting drifts defining acceptance criteria for CFS walls are determined and their ratios are obtained for each performance level. In the current study, to evaluate the behavior of shear walls, four various covers of OSB[1], DFP[2], CSP[3] and GWB[4] with different thicknesses were modeled with regard to one-sided and two-sided cover. Then, for 38 models, target displacement and limiting drift ratios were calculated and examined in an area with high seismicity [3,4].

3. LITERATURE REVIEW

In engineering, CFS constructions are usually analyzed using conventional finite element analysis (CFEA). Although CFEA method is an effective method for analyzing CFS constructions, due to the large number of elements, the production and analysis of its method is time consuming. It also takes a lot of time to review results of this analysis since in constructions; all studs should be studied to determine studs enduring the large axial forces. Accordingly, Martinez [5] provided a simplified finite element analysis method (SFEA) with acceptable accuracy for assessing PBD of CFS constructions that causes the numbers of elements in the design model to reduce and the model can be produced and analyzed in less time. In his study, he used iso-parametric crustal element including 16 nodes that each of these nodes has 5 freedom degrees (three transmissions in line with three axes and two rotations) and updated Lagrange formulation to form the stiffness matrix of element with 16 nodes using Bathe and Bolourchi [6] relations .Then, applying SFEA and CFEA methods, he analyzed 9- SWP panel, compared their results with each other and observed that SFEA models present relatively accurate results for configuration ratios between o.5 to 3. Also, Martinez provided a model of reducing stiffness for the purpose of nonlinear factors related SWP materials. On one hand, experiments conducted by researchers such as COLA-UCI [7], Fulop and Dubina [8, 9], NAHBRC [10] and Branston et al [11] revealed that the relation between load and displacement is non-linear for SWP. Therefore, Martinez also considered non-linear relation between load and SWP displacement in his finite element model and compared the obtained results with those by previous researchers. In general, predictions are well corresponded with experimental results but there was a difference in the form of DFP curves. Martinez used spectrum-based approach method to assess PBD of CFS constructions. In so doing, he used SODA software

[1] Oriented strand board
[2] Douglas fir plywood
[3] Canadian softwood plywood
[4] Gypsum wall board

which had the capability of analyzing pushover and evaluating PBD for those CFS constructions modeled by crustal and frame elements.

In performance-based methods, four levels of performance are defined for a structure including Usability (OP), immediate operation (IO), Life safety (LS) and Collapse prevention (CP). In performance level of OP, the least damage will be put on the structural and non-structural components. In this mode, the construction is suitable for normal use and operation. In performance level of IO, minor damage is put on the structural members that have almost all pre-earthquake strength and stiffness of their own. Non-structural components are also safe and effective. In the performance level of LS, much damage and significant drop may occur in stiffness of structural members but safety margin still remains until collapse. It may not be possible to use the construction before repair. The level of prevention from collapse occurs when the construction has reached a general or local instability mode and the strength and stiffness of structural members reduce substantially. The acceptance criteria determine whether or not the construction has properly been designed to withstand seismic hazards. The acceptance criteria constrain the rate of damage that the construction elements allow to withstand it. These criteria are expressed in terms of strength, displacement, deformation, stresses, etc. In assessing PBD, the acceptance criteria are given as a function of levels of parameters. If criteria in FEMA 273 [12] are used (as used here), damage on the construction will move from bottom to top for performance levels of OP, IO, LS and CP. In CFS constructions, SWP elements are basic structural elements in the resistance of lateral loads. It is why the acceptance criteria are situated based on lateral drift of SWP, while the lateral resistance of each SWP should also be checked to avoid an early failure before reaching the limiting drift. In some cases, the lateral drift of a SWP may be less than the limiting amount while the applied lateral loads have greater SWP resistance and vice versa. Thus, both the lateral drift and SWP resistance should be checked to determine whether or not the panel has properly been designed. In FEMA 273 reference, the acceptance criteria are defined by limiting drift ratios for a variety of structural systems resisting against lateral loads such as steel frames and concrete walls, masonry walls and walls with stud. These limiting drift ratios are usually used to determine those desired displacements that are more likely to be experienced by a structure during an earthquake. In various studies, these ratios are used to calculate the desired displacements in pushover analysis based on displacement. In Chapter 8 of this reference, the amount of limiting ratios with three performance levels have been given for several types of SWP. Also, in this bylaw, the desired damages and corresponding limiting ratios have been described for various structural systems in performance levels of OP, IO, LS and CP. A fundamental problem for CFS structures is that for these structures, no ratio of limiting drifts has been provided whether as acceptance criteria or for estimation of desired displacements in the bylaw. Although for wooden SWPs, ratios of limiting drift have been specified in by law, they cannot be used for the SWPs of CFS. SEAOC and COLA-UCI conducted extensive tests on SWPs with wood and steel framing under the same conditions and concluded that their force-displacement curves are alike and the wooden SWP have more deformation and less resistance than the CFS SWPs. Thus, in the present study, ratios of limiting drift are estimated by experimental data for CFS walls. In FEMA 273, using linear normalized force-displacement curve, the relation between the performance levels has been shown for SWP (Fig. 2).

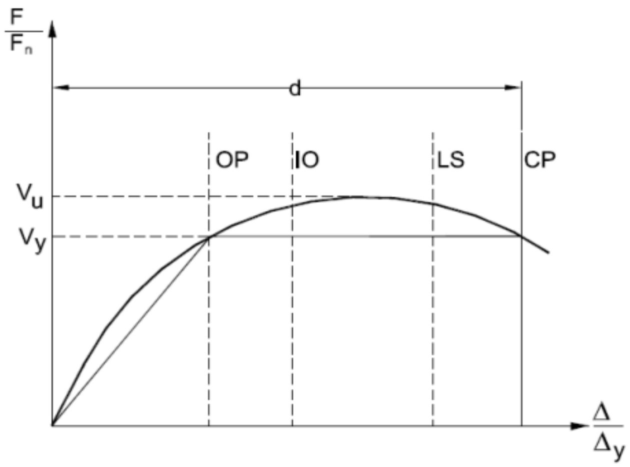

Figure 2. normalized force curve in the ratio of deformation for the wooden elements

In FEMA 273, some equations have been expressed to determine deformations related to various performance levels. These equations are expressed as a function of SWP deformation (*d*) that in performance level of CP, the normalized lateral deformation of SWP is:

$$d = \frac{\Delta_{cp}}{\Delta_y} \tag{1}$$

$$\frac{\Delta_{LS}}{\Delta_y} = 1.0 + 0.8(d-1) \tag{2}$$

$$\frac{\Delta_{IO}}{\Delta_y} = 1.0 + 0.2(d-1) \tag{3}$$

In FEMA 273, no equation has been presented for the normalized deformation corresponding with the performance level of OP. For structures with steel frame, this deformation is calculated at the beginning of submission in elements. About SWP, it has been reported that 40% of submission displacement indicates the load level of service. For SWP, drift ratio is determined by the following equation:

$$LDR(\%) = \left(\frac{\Delta_{PL}}{\Delta_y}\right)\frac{\Delta_y}{h} \times 100 \tag{4}$$

Where *LDR* is the ratio of limiting drift for the performance level expressed as a percentage, *h* is the SWP height, Δ_{PL} is SWP deformation in the selected performance level of IO, LS or CP, Δ_y is deformation in submission state of SWP and Δ_{pl} / Δ_y is normalized deformation.

In a study by Branston *et al* [13], ratios of limiting drift have been specified for the performance targets. The values of these drifts were calculated as 2.5, 1.2, 1.0 and 0.2% for the performance levels of CP, IO, LS and OP, respectively.

4. CAPACITY SPECTRUM METHOD TO DETERMINE THE PERFORMANCE POINT OF STRUCTURE PROPOSED HASH SIGNATURE SAVING SCHEME

In this study, capacity spectrum method is used to determine the performance point of structure.

In fact, structural capacity and seismic demand curves are plotted on a coordinate set and will be compared together to see whether or not the structural capacity responds to the seismic demand. Hence, it is necessary to plot the capacity-need curve in unit set of Acceleration Displacement Response Spectrum (ADRS). Its process is as follows:

Step 1: By having non-linear behavior curve of members and applying an increasing static lateral load, displacements of structure are obtained at each step. Then, the capacity curve is obtained by plotting the base shear in terms of roof displacement. The pushover method is used to do this analysis. To plot the capacity curve in S_a-S_d coordinate set, the following relations are used:

$$S_a = \frac{\left(V_i / W \right)}{\alpha_1} \tag{5}$$

$$S_{di} = \frac{\Delta_{\mathrm{roof}}}{PF_1 \times \varphi_{1,roof}} \tag{6}$$

Where α_1, PF_1, V, W and $\varphi_{1,roof}$ are the effective mass index in the first mode, participation factor in the first mode, base shear, whole weight of construction with considering a percentage of live load and the first mode form shape in roof floor, respectively.

Step 2: To obtain the seismic demand curve, elastic design spectrum with damping of 5% considered as plan target in designing target is used with the difference that instead of S_a-T format, it will become S_a-S_d format (spectrum ADRS) using a convention so that it can be plotted by the capacity curve in a coordinate set. Then, by applying some coefficients, the elastic spectrum is reduced and seismic demand curve is obtained. The reduction rate of elastic spectrum depends on the behavior range of structural deformation.

Step 3: Each point on the capacity curve has an equivalent damping, an equivalent period, a deformation and a specific yield point that according to Fig. 3, to determine these specifications, the capacity curve with a linear curve is approximated so that the surface below these two curves to the desired point indicating the energy absorbed by the system are equal. Using this equation, the yield point is determined. By having this point, all mentioned information can be determined for any point of the capacity curve.

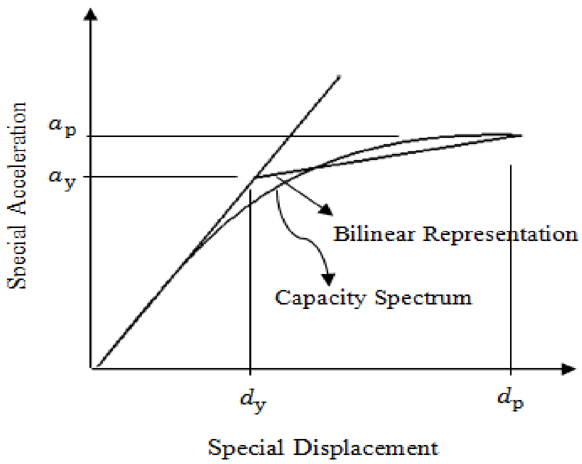

Figure 3. Capacity and Bilinear curves

To reduce seismic response spectrum, the non-linear damping of structure should be estimated. The damping which occurs in the inelastic range during an earthquake in a structure is a combination of elastic viscous damping and Hysteresis damping.

Elastic viscous damping is actually the structural inherent damping assumed equal to 5%. Hysteresis damping is related to cyclic inter area obtained by plotting the reciprocating seismic force (base shear) against the structural displacement. Accordingly, using two-linear model of two-linear structural capacity spectrum curve, guideline of ATC 40 [14] offers the use of effective damping to determine the reduced need spectrum according to the following equation:

$$\beta_{eff} = \frac{63.7k_h\left(a_y d_p - a_p d_y\right)}{a_p d_p} + 5.0 \tag{7}$$

Where coefficient of k_h was used to convert ideal Hysteresis diagram to a parallelogram, other parameters have been specified in Fig. 3.

Step4: Now, the reduced ADRS spectrum is plotted based on the effective damping. Reduction coefficients of response spectrum including spectrum reduction coefficient in the range of constant acceleration and spectrum reduction coefficient in constant speed range are obtained by Eqs. (8) and (9), respectively.

$$SR_A = \frac{3.21 - 0.68 \times \ln(\beta_{eff})}{2.12} \tag{8}$$

$$SR_V = \frac{2.31 - 0.41 \times \ln(\beta_{eff})}{1.65} \tag{9}$$

Step 5: Collision location of the capacity curve with decreased S_a-S_d spectrum (d_{pi}) shows maximum structural displacement. This displacement is the target displacement and its corresponding forces are determined using results of non-linear static analysis (Fig. 4).

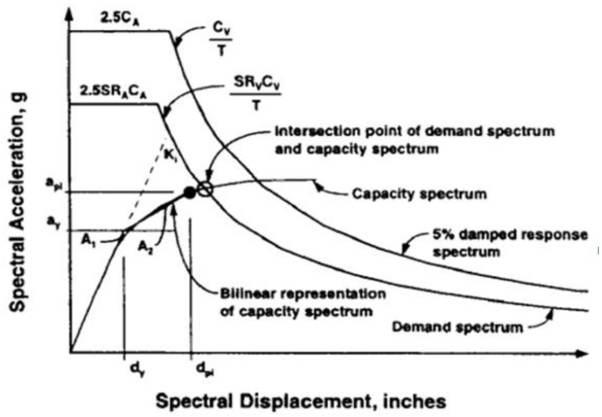

Figure 4. Collision of capacity curve with reduced S_a-S_d spectrum (d_{pi})

Step 6: d_{pi} from Step 5 is compared with its assumed amount in step 3. If the difference is inacceptable rate, the analysis is completed; otherwise, by considering new d_{pi}, steps of 3 to 5 are repeated to achieve convergence.

5. MODELING

In the present study, the numerical method of finite elements is used to model the frames.

5.1 Verification of analytical model by laboratory model

Among from all numerical modeling methods, one way to achieve higher confidence is to corresponding numerical results with laboratory results. Thus, given the similarity degree of the finite element model to the actual conditions of issue and possibility of stimulating its complexities, the possible failures of members and connections and loading conditions, an acceptable model with minimum error can be achieved. In estimating the monotone behavior of cold formed steel frames, the laboratory sample of LSF frame done by Fulop and Dubina [8] is modeled using the finite element program and MSC PATRAN-NASTRAN software [15] in the following and the analysis results are compared with laboratory results [3].

5.1.1 Profiles of sections and materials used in the model

In the present study, the height and length of frames are 2.44 m. Adjunct and central masters are made up of C-shaped sections welded to the front. Central studs were installed with nominal distance of 406 mm compared together. Properties of sections and behavior of materials used in double and single masters, cracks and covers have been provided in Tables 1- 3.

Table 1: Dimension sections and material properties

Member	Thickness (mm)	Dimensions (mm)	Nominal grade F_y (MPa)
Chord studs	1.91	152 x 41 x 12.7	345
Interior studs	1.22	152 x 41 x 12.7	230
Tracks	1. 91	152 x 31.8	345
Connection plate	1.91	300×300	230

Table 2: Material properties of sheathing

Sheathing	Modul of Elasticity (MPa)	Yield stress F_y (MPa)	ν
OSB	9917	3.50	0.30
CSP	7376	3.20	0.25
DFP	10445	3.80	0.30
GWB	1290	2.00	0.20

Table 3: Matrix of braced walls (nominal design dimensions and material properties)

Member	Nominal thickness (mm)	Thickness (mm)	Yield stress Fy (MPa)	Ultimate stress Fu (MPa)	$\frac{F_u}{F_y}$	Elng. %	$\frac{F_y}{F_{yn}}$
Chord studs	1.91	1.91	352	489	1.39	35	1.02
Interior studs	1.22	1.23	336	398	1.19	35	1.46
Tracks	1.91	1.94	348	474	1.36	37	1.01

5.2 Results of the evaluation of modeling accuracy

Results of modeling panel of shear wall with finite element software and laboratory work [8,9] have been presented in Table 4. According to the results, an appropriate correspondence can be observed between lateral resistance and drift from results of numerical and laboratory samples. Difference of less than 5% causes to increase confidence in the results which will be evaluated in the following.

Table 4: Comparison of results experimental and analytical

Specimen	analysis results of sheathing walls (2.44 m x 2.44 m)	
	Experimental	Analytical
Lateral Resistance (kN)	78.76	74.93
Difference Percent of Resistance	5%	
Maximum Lateral Drift (%)	0.175	0.184
Difference Percent of Drift	5%	

5.3. PARAMETRIC STUDY

After ensuring the accuracy of presented analytical model, samples were modeled with four types of various covers in finite element software. Height and length of all frames are assumed constant (2.44m). Each frame consists of upper and lower crack, adjunct and central masters, and also cover panels. In all samples, sections and materials used in studs, cracks and covers were the same and their specifications have been provided in Tables 1- 3 in section 5.1.1. In this study, it is assumed that structure locates on soil of type 2 with too much relative risk. So, according to Fig. 5, the demand spectrum is plotted in terms of S_a-S_d.

By considering one- and two-sided cover, each of these covers with thicknesses of 10, 12.5, 15, 17.5 and 20 mm were modeled in order to consider the impact of thickness on the seismic performance. Since for DFP cover, the least base thickness equals 12.5 mm, for this type of cover, various thicknesses was considered 12.5 mm. In total, among from frames made in the earlier part, 38 frames are to be evaluated in this study. The capacity spectrum of all frames is created and then, by doing the operation detail described in the previous sections, the spectrum is located on S_a-S_d diagram. It should be noted that the response curve from the analysis can create many errors compared to correct curve. To obtain correct results, the size of load growth is considered in terms of the type of variable structure. For example, Garrison et al considered a load growth equal to 5% of the total lateral loads for the analysis of steel framing constructions [16]. To determine the appropriate size of load growth, for CFS constructions, the load growth equal to 1% of the total load is used for pushover analysis.

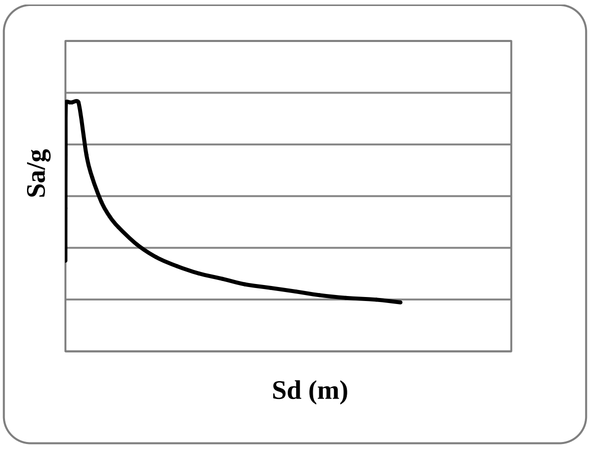

Figure 5. Demand spectrum

6. DISCUSSION AND RESULTS

Results from this part of the thesis are after laboratory researches of Branston et al [13]. The conducted studies show appropriate correspondence of results obtained from provided numerical analysis with the previous laboratory researches. However, the difference between this study and the previous ones is the consideration of the effect of parameters such as thickness, material, materials of cover sheet and finally the type of lateral brace (being one- and two sides frame cover) (Tables 5 and 6).

As it has been shown in Tables 5 and 6, compared to limiting drift, the highest and lowest percentage are related to GWB models with cover sheet thickness of 10mm and DFP with thickness of 20 mm. Given the values of effective parameters such as Δ_{target} and Δ_y mentioned in these tables, this result is justified.

A very important point is the minimal difference (less than 3%) of the ratio percentage of limiting drift among one- and two sided cover sheets. Studies show that in the case of using frames braced by one-sided cover sheets, no significant difference will not only be occurred in their seismic performance, but also significant economic saving will bring about.

As it is expected, the target displacement (Δ_{target}) of all models is less than displacement of the lowest performance level (OP). This issue shows an appropriate performance of these light structures against the earthquake of the plan. The most target displacement occurred among all samples studied in GWB sample with the thickness of 12.5 mm being 62% less than displacement to the performance level of usability (OP). In all samples with changing cover from two-sided state to one-sided state, the rate of target and structural yield displacements has been reduced in a significant rate (Fig. 6).

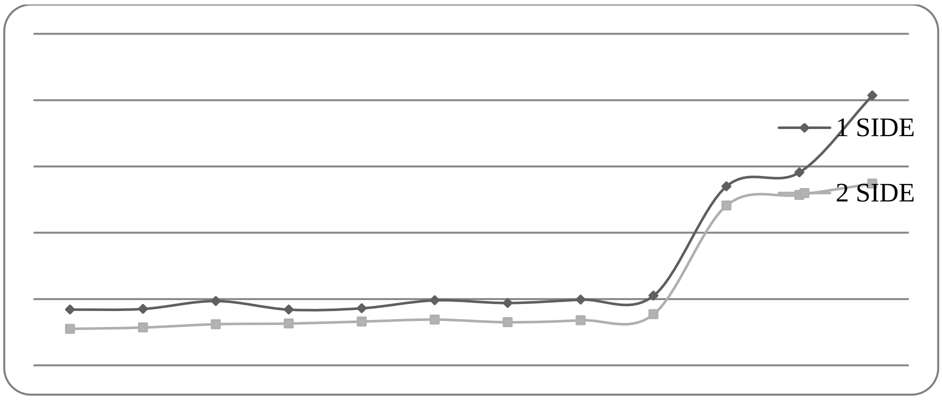

Figure 6. Comparison rate of target for variable sheaths

Table 5: One side frame cover

Sheathing	Thickness (mm)	Δ_{target} (mm)	Δ_y (mm)	d $\left(\frac{\Delta_{faliuer}}{\Delta_y}\right)$	Normalized displacement (mm)				Drift ratio (%)			
					CP	LS	IO	OP	CP	LS	IO	OP
GWB	10	5.18	29.27	4.70	120.15	101.98	47.24	11.17	4.92	4.29	1.94	0.48
	12.5	4.07	26.76	4.62	120.01	101.86	46.10	10.70	4.92	4.27	1.89	0.44
	15	2.91	13.74	4.50	71.73	60.13	25.34	5.50	2.94	2.46	1.04	0.23
	17.5	2.70	12.72	4.10	57.29	48.37	21.63	5.09	2.35	1.98	0.89	0.21
	20	2.52	12.04	3.54	42.64	36.52	18.16	4.82	1.75	1.50	0.74	0.20
CSP	10	1.19	14.91	6.82	99.78	82.98	31.80	5.96	4.07	3.39	1.30	0.24
	12.5	1.05	13.63	6.79	99.30	82.48	31.35	5.45	4.07	3.38	1.28	0.22
	15	0.99	9.39	6.66	63.70	52.88	20.26	3.76	2.61	2.17	0.83	0.15
	17.5	0.94	8.93	5.75	51.38	42.89	17.42	3.57	2.11	1.76	0.71	0.15
	20	0.87	8.51	4.49	38.24	32.30	14.46	3.40	1.57	1.32	0.59	0.14
OSB	10	1.13	12.86	6.52	83.37	69.26	26.96	5.14	3.42	2.84	1.10	0.21
	12.5	0.98	11.76	6.48	83.35	69.26	26.56	4.70	3.42	2.84	1.09	0.19
	15	0.86	8.19	6.24	56.81	47.09	17.91	3.28	2.33	1.93	0.73	0.13
	17.5	0.84	7.64	5.99	45.73	38.11	15.26	3.06	1.87	1.56	0.63	0.13
	20	0.80	7.44	4.57	34.04	28.72	12.76	2.98	1.39	1.18	0.52	0.12
DFP	12.5	0.97	10.81	7.10	76.80	63.60	24.01	4.32	3.15	2.61	0.98	0.18
	15	0.85	7.64	6.24	47.71	39.70	15.65	3.06	1.96	1.63	0.64	0.13
	17.5	0.83	7.31	5.34	39.04	32.69	13.66	2.92	1.60	1.34	0.56	0.12
	20	0.82	7.15	4.06	29.05	24.67	11.53	2.86	1.19	1.01	0.47	0.12

Table 6: Two side frame cover

Sheathing	Thickness (mm)	Δ_{target} (mm)	Δ_y (mm)	d $\left(\frac{\Delta_{faliuer}}{\Delta_y}\right)$	Normalized displacement (mm)				Drift ratio (%)			
					CP	LS	IO	OP	CP	LS	IO	OP
GWB	10	5.18	29.27	4.70	120.15	101.98	47.24	11.17	4.92	4.29	1.94	0.48
	12.5	4.07	26.76	4.62	120.01	101.86	46.10	10.70	4.92	4.27	1.89	0.44
	15	2.91	13.74	4.50	71.73	60.13	25.34	5.50	2.94	2.46	1.04	0.23
	17.5	2.70	12.72	4.10	57.29	48.37	21.63	5.09	2.35	1.98	0.89	0.21
	20	2.52	12.04	3.54	42.64	36.52	18.16	4.82	1.75	1.50	0.74	0.20
CSP	10	1.19	14.91	6.82	99.78	82.98	31.80	5.96	4.07	3.39	1.30	0.24
	12.5	1.05	13.63	6.79	99.30	82.48	31.35	5.45	4.07	3.38	1.28	0.22
	15	0.99	9.39	6.66	63.70	52.88	20.26	3.76	2.61	2.17	0.83	0.15
	17.5	0.94	8.93	5.75	51.38	42.89	17.42	3.57	2.11	1.76	0.71	0.15
	20	0.87	8.51	4.49	38.24	32.30	14.46	3.40	1.57	1.32	0.59	0.14
OSB	10	1.13	12.86	6.52	83.37	69.26	26.96	5.14	3.42	2.84	1.10	0.21
	12.5	0.98	11.76	6.48	83.35	69.26	26.56	4.70	3.42	2.84	1.09	0.19
	15	0.86	8.19	6.24	56.81	47.09	17.91	3.28	2.33	1.93	0.73	0.13
	17.5	0.84	7.64	5.99	45.73	38.11	15.26	3.06	1.87	1.56	0.63	0.13
	20	0.80	7.44	4.57	34.04	28.72	12.76	2.98	1.39	1.18	0.52	0.12
DFP	12.5	0.97	10.81	7.10	76.80	63.60	24.01	4.32	3.15	2.61	0.98	0.18
	15	0.85	7.64	6.24	47.71	39.70	15.65	3.06	1.96	1.63	0.64	0.13
	17.5	0.83	7.31	5.34	39.04	32.69	13.66	2.92	1.60	1.34	0.56	0.12
	20	0.82	7.15	4.06	29.05	24.67	11.53	2.86	1.19	1.01	0.47	0.12

Due to the lack of significant difference between results from pushover and linear analyses, the linear analysis can always be used for seismic SWP design. This finding indicates that the linear analysis may affect the geometric parameters (size) of cold formed steel constructions.

However, the study results show that linear analysis is not capable of estimating the internal forces of studs and lateral drifts to a great extent. Therefore, to determine the seismic performance level of cold formed steel structures, applying pushover analysis is required.

Using pushover analysis, the weight of structure and number of floors play a significant role in determining the performance level of structure. Due to the type of thin-walled sections applied in cold formed steel structures, a significant reduction in their weight is created compared to conventional construction systems. On one hand, regulations establish that cold formed steel structures should be implemented as short or medium order. The issue causes the dominant mode on these structures to be consistent with the first mode. Therefore, by applying pushover analysis in these structures, very precise results are obtained. The performance level of CFS structures is independent of the type of cover used in these structures, thickness of cove and type of location (as one- or two sided). As it is observed, in all cases even at the lowest thickness, the performance level satisfies the usability (OP).

7. CONCLUSIONS

Methods called performance design are an attempt to find methods of analysis and more accurate design to seek the real performance of structures under earthquakes with various levels. In these methods, by defining various performance levels for structural behavior, applying different seismic levels of plan and doing nonlinear analyses, it is tried to predict the actual behavior of construction against earthquake.

According to all walls studied in this study, the structure responds the earthquake at the linear state and the performance point is at the elastic stage of structural behavior.

Generally, due to low freedom degrees and small periods, one-floor-structures can respond to earthquake by some linear increase in their stiffness.

Also, these studies show that LSF construction system essentially indicates a good reaction to deal with earthquake. Therefore, given the proper performance of system against earthquake, this construction method can be considered as a proper method for construction industry in the country.

According to experiments, the weight of LSF system is almost a third of that of conventional traditional systems and is 60% lighter than wooden systems. Since the earthquake load depends on the dead load of construction, using LSF construction system can decrease earthquake problems.

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