SEISMIC PERFORMANCE ASSESSMENT OF BUILDINGS WITH COLD-FORMED STEEL SHEAR WALL PANELS

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ABSTRACT

Presented in this paper is a method for seismic performance assessment of cold-formed steel buildings built with shear wall panels as the lateral load resisting system. In the proposed method, performance-based design philosophy is adopted to assess the seismic performance for the cold-formed steel framing system. The structural responses of cold-formed steel buildings are obtained from a simplified finite element based on pushover analysis developed in this study with accounting for the nonlinear behaviour of shear wall panels. A practical example is presented to illustrate the effectiveness of the proposed method. The results from the proposed method are compared to results obtained from the conventional analysis. Issues associated with performance-based design of cold-formed steel buildings are discussed.

KEYWORDS

Performance-based design, seismic performance assessment, cold-formed steel, shear wall panels, nonlinear pushover analysis

INTRODUCTION

The application of cold-formed steel framing in residential building construction has increased significantly in past few decades. The global trend of applying cold-formed steel framing is advancing to mid-rise buildings. This includes hotels, multi-family housing, dormitories, and mixed-use buildings of 4-9 stories. Cold-formed steel framing provides an efficient and economical structural system which has numerous advantages and benefits with broad applications. Among them, the features of being lightweight and having good ductility have made cold-formed steel framing an attractive structural system in seismic regions where mid-rise buildings traditionally have relied on heavier construction materials.

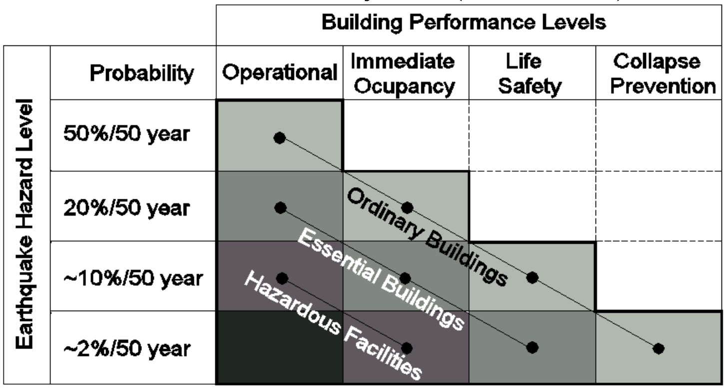
Aimed to more reliable means of achieving life safety and property protection in seismic events, performance-based design principles have been adopted in new generation of building codes globally. In contrast with conventional design practice that focuses primarily on life-safety load levels, performance-based design also requires a structure to meet several performance objectives at lower load levels, within acceptable ranges of reliability. Such change in design philosophy and the consequent necessity for more complex analysis and design techniques to accurately evaluate structural performance at different load levels, creates a new challenge for structural engineers.

The application of performance-based design (PBD) assessment does not follow a prescribed procedure; however, nor is it the same procedure for all structural systems since it depends on the building characteristics and designer’s preferences. For instance, several methods for carrying out the seismic assessment of buildings for PBD are available in current practice, such as linear, nonlinear, static, and dynamic methods, and combinations of those methods. The complexity of the application of each method is different and so are their capabilities; as a result, some of the methods might not be suitable for analyzing certain types of structures. Previous study has shown that results with reasonably accuracy can be expected from applying pushover analysis (nonlinear static method) of cold-formed steel (CFS) buildings with medium height (Martinez 2007, Martinez-Martinez and Xu, 2011). Therefore, the proposed method is limited to low- and mid-rise buildings that have a predominant fundamental mode of vibration in their response, so that single mode pushover analysis can be used. The inelastic behaviour of a building in a seismic event is a consequence of the geometric and material nonlinear behaviour of its structural components. Where, the material nonlinearity is accounted for by incorporating the nonlinear load-to-displacement curve of the lateral-load resisting system of the building.

PBD assessment involves transforming each selected performance objective to be satisfied by the structure into a tangible parameter that can be measured in a seismic analysis. This study adopts a spectrum-based pushover analysis approach, also known as the force-based approach, in which the performance objectives are transformed into corresponding target base shears. When the lateral loads applied on the building reach to the target base shear associated with one of the governing performance objectives, the shear wall panels (SWP) are checked to determine if they comply with the acceptance criteria corresponding to the performance level.

In this study the acceptance criteria (limit damage state) for CFS buildings are established as a function of the lateral drift and lateral strength of the SWP. FEMA 273 (1997) provides limit drift ratios for several types of structures, including steel moment frames, concrete walls, masonry infill walls, and wood stud walls. The four performance levels shown in Table 1 are associated with limit drift ratios to determine if a structure design is adequate. It is common to use the drift ratios to compute target displacements for the displacement-based PBD assessment of buildings. FEMA 273 provides such limit drift ratios for wood framed panels only and no information is available for CFS shear wall panels. Therefore, in this study, the drift ratios for CFS buildings are determined from experimental test results.

Table 1. Performance objectives (SEAOC, 1995)



SPECTRUM-BASED PBD ASSESMENT OF CFS BUILDINGS

In this study, the spectrum-based approach is preferred over the displacement-based approach for the PBD assessment of CFS buildings. Typically, a building design is carried out using design spectra from pertained seismic design standards, which are used to compute the target base shears and the magnitude of the applied lateral loads of the building. Conversely, if the displacement-based approach is chosen for the PBD assessment of a building, lateral displacement is used as a target parameter. Arbitrary lateral load increments or lateral displacements are applied to the building until the target displacement is reached or until the structural collapse occurs. Then, with the total lateral load applied on the building in the analysis, the base shear can be computed and consequently the acceleration is then compared to the site design spectra to determine if the building satisfies the site requirements. Thus, in the displacement based approach, designers do not know if the building satisfies the site requirements until the analysis is complete. This may require more unnecessary displacement or load increments. The spectrum-based approach, however, is consistent with the seismic design standards. A discussion on the advantages and disadvantages of both approaches was presented by Grierson et al (2005).

For spectrum-based PBD assessment, each performance objective is associated with a particular earthquake hazard. The corresponding base shear is adopted as the damage target parameter, which represents the maximum base shear that a building can undergo during an earthquake. The target base shear for a building is computed as a function of the spectral acceleration (S_a), structural seismic weight (W), and gravity acceleration (g) for each selected performance objective as follows: for each of multiple performance objectives (PO) possible for a building (see Table 1), the base shear is computed as:

$$V_b^{PO} = S_a^{PO} \frac{W}{g} \tag{1}$$

Where S_a^{PO} is the spectral acceleration at the selected performance objective (PO), W is the seismic weight of the building and g is the gravity acceleration.

SPECTRUM-BASED PUSHOVER ANALYSIS FOR CFS BUILDINGS

In the spectrum-based pushover analysis adopted in this study, the building is subjected to incrementally applied lateral loads until the target base shear is reached or the structure collapses (the lateral force resisting elements have lost their strength and stiffness). Throughout this analysis process, the gravity loads are maintained constant on the building. Upon applying a lateral load increment on the structure, the structural analysis can be carried out by the finite element analysis. In this study, however, the analysis is carried out using a simplified finite element method discussed below with accounting for both geometric and material nonlinearities of the CFS shear wall panels (SWP) (Martinez-Martinez and Xu, 2011). From the structural analysis results at each incremental load level, the corresponding incremental displacements are obtained, and then the accumulated total displacements up to the current stage of the loading process are found.

In the PBD assessment of a CFS building, the acceptance criteria for the SWP are checked when the base shear of the building is equal to the target base shear for any selected performance objective, V_b^{PO}. Thus, if the structural elements satisfy both the deformation and strength requirements, the acceptance criteria for the specified performance objective are satisfied.

Simplified Finite Element Analysis

The simplified method can be used with any finite element analysis software with orthotropic shell elements. For that purpose, a SWP in a CFS building needs to be transformed into a flat shell with equivalent properties in both x and y directions as shown Figure 1.

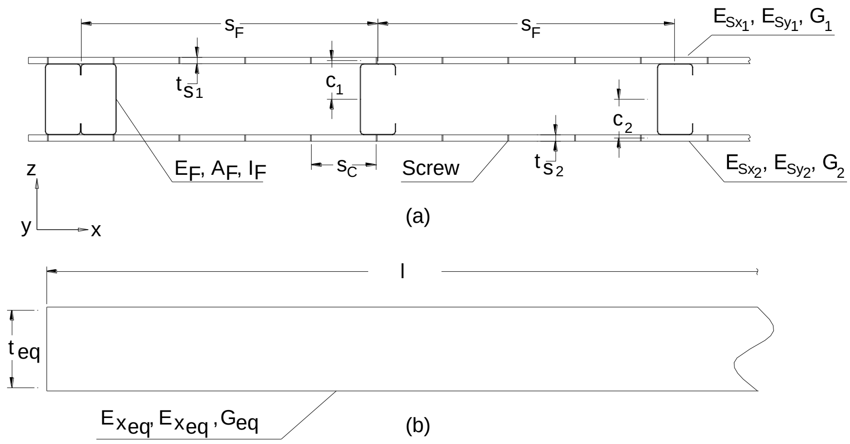


Figure 1 a) SWP cross section; b) equivalent shell

Figure 1 shows the cross section of a CFS SWP and its equivalent shell cross section, where: s_F is the spacing between CFS C-shape studs, s_C is the distance between screws on the edge of the panel, and c_1 and c_2 are the distances from the centre of the studs to the mid-plane of Oriented Structural Board (OSB) sheathing on side 1 and 2, respectively. The parameters E_F, A_F, and I_F denote the modulus of elasticity, cross sectional area, and moment of inertia about the strong bending axis of the studs, respectively; while l is the length of the panel and equivalent shell element; and t_{S1} and t_{S2} are thicknesses of the sheathing on side 1 and side 2, respectively. E_{sx}, and E_{sy} are the modulus of elastic of the sheathing for the x and y directions for each side, and the G_s is the sheathing shear modulus of elasticity.

Troitsky (1976) developed equations to determine the equivalent rigidity of orthotropic ribbed plates in bending. Martinez-Martinez and Xu (2011) extended Troitsky equations to account for the axial rigidity of the panel in the direction along the longitudinal axis of the studs (i.e., y direction). The equivalent thickness and modulus of elasticity in the y direction of a shell element are determined by equating and solving the axial and flexural rigidities of a panel and an orthotropic element. Afterwards, the equivalent modulus of elasticity in x direction is determined by equating and solving the bending rigidity of a panel and an orthotropic shell. The equations for computing the equivalent properties for the SWP are provided by Martinez-Martinez and Xu (2011).

Nonlinear finite element analysis

The shell element used in this study to model SWP in a CFS building is an isoparametric element, which typically has between four and sixteen nodes. It is recommended that sixteen-node shell element to be used because of its accuracy of modeling a typical 4' × 8' (1200mm × 2400mm) CFS SWB with one shell element. Each node of the sixteen-node shell element has five degrees of freedom (i.e., three translations along the *x*, *y*, and *z* axes, and two rotations about the *x* and *y* axes), which is adequate to simulate CFS panels used in construction practice.

An updated Lagrangian formulation is adopted to develop the element stiffness matrix of the sixteen-node shell element, where the values of all parameters, static and kinematic, are computed in accordance with the latest deformed configuration of the element. The procedure to achieve the elastic and geometric stiffness matrices for the sixteen-node shell element is presented in Bathe and Bolourchi (1982).

PERFORMANCE ACCEPTANCE CRITERIA FOR CFS BUILDINGS

The acceptance criteria determine if a building is properly designed to satisfy the performance levels (demands) imposed by the seismic hazards. In PBD assessment, the acceptance criteria are associated with the performance levels. If FEMA 273 (1997) criteria are adopted, as that in this study, the intensities of building damage increase from low to high for the Operational (OP), Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) performance levels. In CFS buildings, the SWP's are the primary structural elements in resisting lateral loads; consequently, acceptance criteria based on SWP lateral drift is established. In addition, the strength of each SWP also needs to be checked to prevent premature structural failure prior to reach the limit drift. In some cases, the lateral drift of a SWP might be less than the limit value but the applied lateral loads may exceed the strength of SWP, or vice versa. Therefore, both the lateral drift and strength of the SWP need to be checked to determine if the panel has been designed appropriately.

For CFS SWP, FEMA 273 (1997) does not provide any information about the limit drift ratios and acceptance criteria for estimating target displacements. Although wood and CFS shear wall panels are built in a similar manner, their force-displacement curves are not similar. Thus, the limit drift ratios for wood-framed SWP cannot be used for CFS-framed SWP. In this study, the limit drift ratios for CFS shear walls are estimated from experimental data by Branston *et al*. (2006). FEMA 273 (1997) provides the equations for determining the normalized deformation associated with the three performance levels CP, LS and IO. The equations are expressed as a function of the ductility *d* of the SWP, which is the normalized lateral deformation of the SWP at the CP performance level. The equation for the normalized deformation corresponding to the OP performance level is not provided by FEMA 273 (1997). For SWP, it has been reported that 40% of the yield displacement represents the service load level (Branston *et al*, 2006). The limit drift ratios for SWP at any performance level are given by the following equation:

$$LDR(\%) = \left(\Delta_{PL}/\Delta_y\right)\frac{\Delta_y}{h}\times 100 \tag{2}$$

where *LDR* is the limit drift ratio for the selected performance level (PL); *h* is the height of the SWP; and ratio $\left(\Delta_{PL}/\Delta_y\right)$ is the normalized deformation.

The limit drifts for the performance objectives need to be computed before the pushover analysis is initiated. The limit drifts ($Drift^{PO}$) for a SWP are evaluated by multiplying the SWP height by the corresponding performance level *LDR* (i.e., 2.5%, 2.1%, 1.0% and 0.20% for the CP, LS, IO and OP performance levels, respectively). If the SWP lateral drifts obtained from the pushover analysis are less than the corresponding limit drifts, the displacement performance criterion is satisfied and otherwise the design must be improved.

In addition to checking the limit drifts of the SWP, the adequacy of lateral strength also needs to be satisfied to ensure the lateral strength of SWP, P_R, is greater than the applied lateral force, P_a. The lateral strength of the SWP, P_R, can be obtained from the published codified data (AISI, 2012) which is determined by testing. In the absence of applicable date, rational analysis such as the method proposed by Xu and Martinez (2006) can be considered to determine the corresponding strength. While design a CFS SWP, the following design considerations have significant impact on the lateral strength of the SWP: increasing the sheathing thickness, attaching sheathing on the both sides of the SWP (vs. sheathing being used only on one side), reducing the spacing of the screws for the sheathing-to-framing connections at the perimeter of the SWP, and increasing the diameter of the screws; while modifications below are considered to have minor influences on the lateral strength of a SWP: increasing the thickness and depth of the CFS wall studs, reducing the spacing between the studs, and reducing the spacing of the screws for the sheathing-to-framing connections in the field of the SWP. The effects of the foregoing design considerations on the lateral strength of a SWP have been observed from experimental tests by Serrette et al. (2002) and Branston et al. (2006); and confirmed by the numerical analysis conducted by Xu and Martinez (2006).

STIFFNESS DEGRADATION MODEL FOR THE SWP

Experimental investigations, such as those reported by COLA-UCI (2001), Fulop and Dubina (2004), and Branston et al. (2006), have shown that the load-displacement relationship for a CFS SWP is nonlinear. In addition, it is observed in the aforementioned experimental investigations that the loss of stiffness and the failure of SWP are primarily due to the failure of sheathing-to-framing connections. Therefore, the nonlinear relationship between load and displacement of SWP needs to be accounted for in the finite element model of the building.

Shown in Figure 2 is the proposed nonlinear model to characterize the stiffness degradation of SWP subjected to lateral forces: K_i is the initial stiffness of the SWP as the tangent at $P_r = 0$; $K_a^{(q)}$ is the secant stiffness corresponding to the load increment, q; P_R is the lateral strength of the SWP; P_a is the magnitude of the total lateral force applied on the top of the panel at load increment q; $\lambda^{(q)}$ is the stiffness degradation coefficient that characterizes the nonlinear behaviour of the SWP under the applied lateral forces until failure and is defined by,

$$\lambda^{(q)} = 1 - \left(\frac{P_a^{(q)}}{P_R} \right)^{\beta} \tag{3}$$

where β=(38mm/s_C) is a stiffness degradation nonlinearity exponent, calibrated in accordance with experimental results from Branston et al. (2006) and COLA-UCI (2001). It is observed from the results of the experimental investigations that the nonlinear load-displacement relationship for a SWP is primarily influenced by the sheathing material and edge screw spacing. For reasons of simplicity, β was calibrated considering the screw spacing on the SWP perimeter (s_C) only in this study as described by Martinez (2007).

$\lambda^{(q)}$ shown in Eq. (3) is equal to unity at the initial load increment, indicating that the SWP lateral stiffness has not yet been affected. When the applied load P_a is equal or greater than the maximum strength P_R, $\lambda^{(q)}$ becomes zero, which indicates that the SWP has completely lost its stiffness to resist lateral deformation. However, the modulus of elasticity of the CFS studs remains unchanged in the y direction of the shell element, which means the equivalent shell may still have sufficient stiffness contributed by the studs to allow the SWP to continue carrying gravity loads.

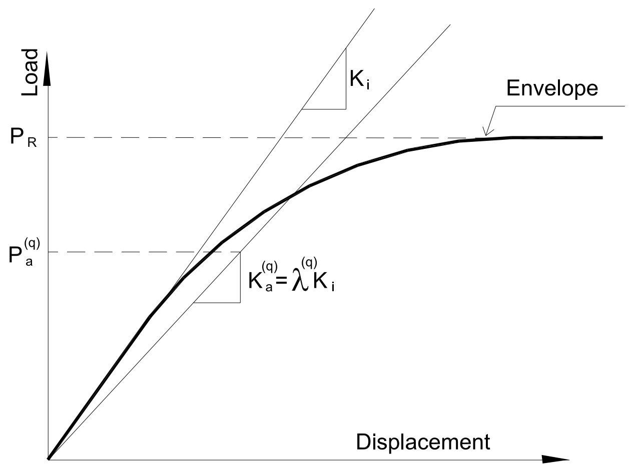


Figure 2. Characterization of the SWP loss in strength

PROCEDURE OF PBD ASSESSMENT FOR CFS BUILDINGS

A flowchart shown Figure 3 summarizes the procedure of the proposed method for PBD assessment of CFS buildings.

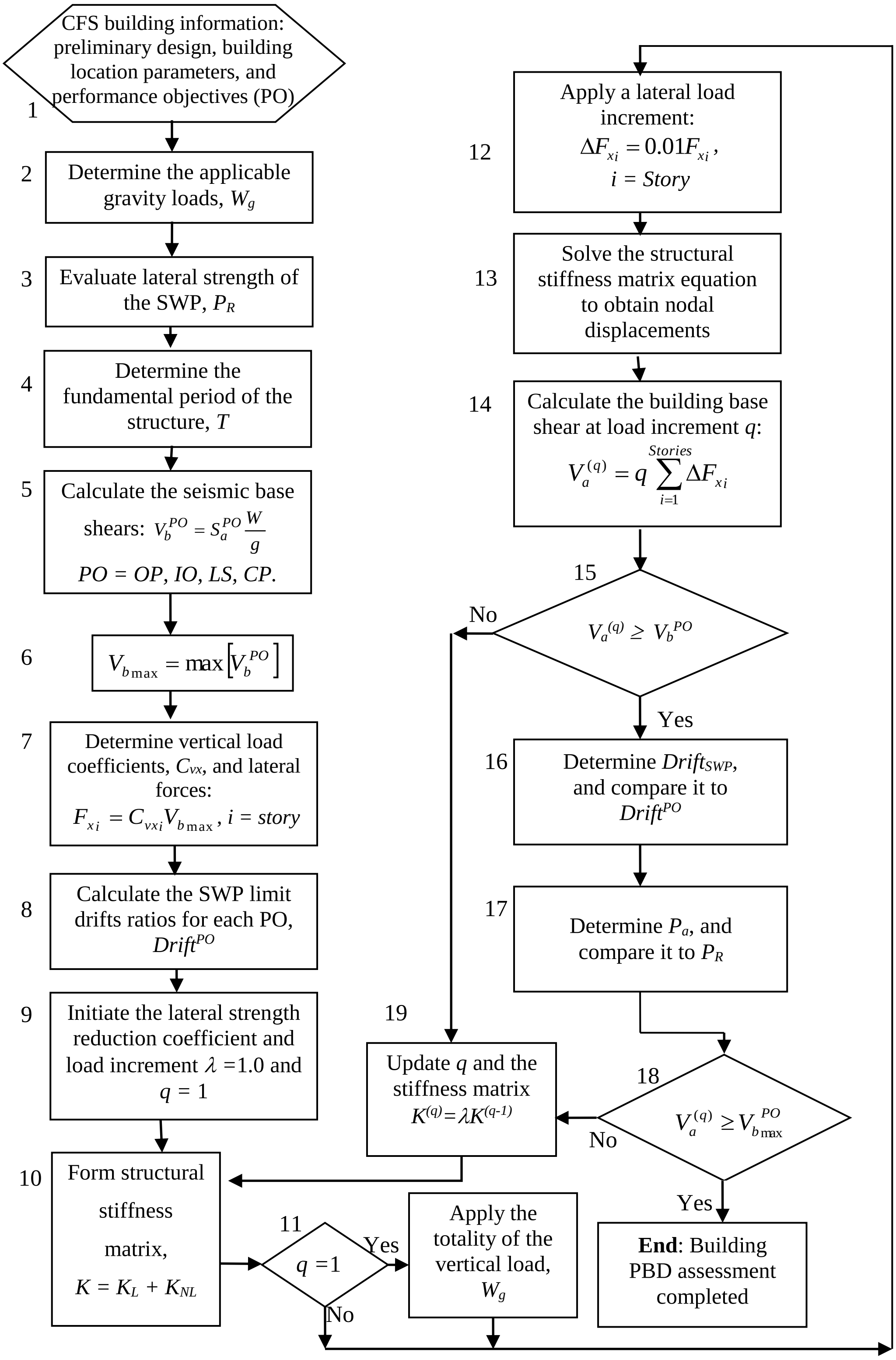


Figure 3. Proposed PBD assessment procedure

EXAMPLE

Shown in Figure 4 is a three-storey CFS building, for which the lateral displacements are to be computed for four performance levels using the pushover analysis, and the results obtained from the pushover analysis will also be compared to that of conventional linear elastic analysis. The typical floor plan of the building is shown in Figure 5. All the SWP in the building are built with cold-formed steel C-shape studs 152S51–1.73 mm (600S200-68 mils), sheathed on both sides with 12.5 mm (1/2 in) Douglas Fir Plywood (DFP). The studs in

SWP 2 are spaced 610 mm on centre. In all the SWPs, a single end-stud is used. No. 8 screws are used to attach the sheathing to the framing at 102 mm and 305 mm spacing on the edge and in the field of the SWP, respectively. The size and spacing of the screws are included to evaluate the lateral strength of the SWP in accordance with the procedure described in Xu and Martinez (2006). All the SWP on the first storey have pin supports. The floor panels consist of a concrete slab of 127 mm thickness, supported by 254 mm (10 in) deep CFS joist, designated 254S64-1.37 mm (1000S250-54 mils). The height for all the storeys is 2850 mm, and the pitched roof has 17% slope. For this example, for the reason of simplicity, the roof panels are assumed to be identical to the floor panels.

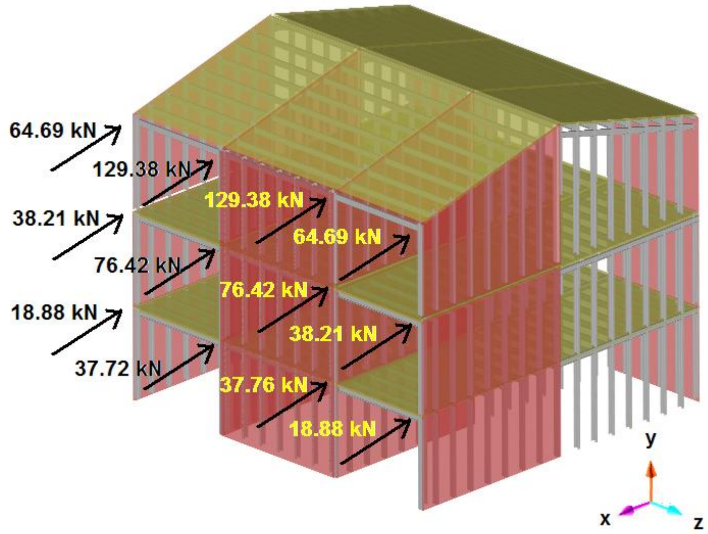


Figure 4. Example: three-storey CFS building

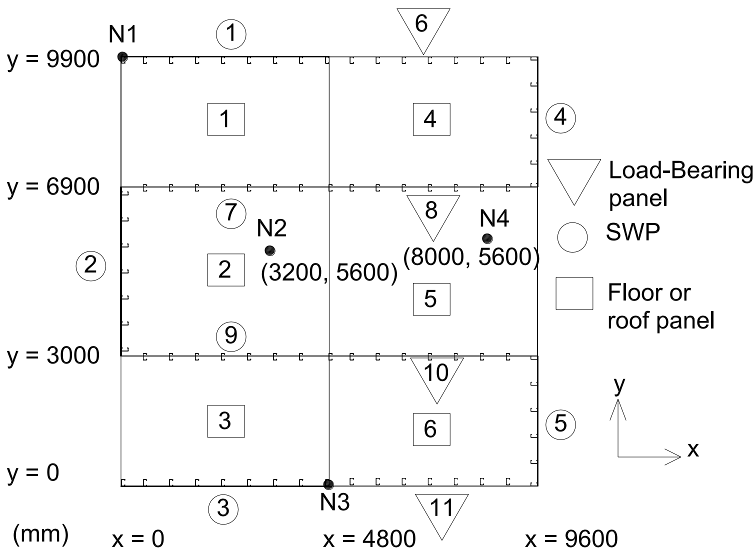


Figure 5 Typical floor plan and element illustration of the three-storey CFS building

The three-storey building is considered to be ordinary according to the categories established by SEAOC (1995), so that the building must satisfy the four performance objectives described in Table 1. The building is assumed to be located in California at latitude 41.0° N and longitude 115.2°W on a site class B. The four performance objectives are the following: OP for a 50%/50 year earthquake, IO for a 25%/50 year earthquake, LS for a 10%/50 year earthquake, and CP for a 2%/50 year earthquake.

The seismic weight of the building is W=405 kN. The natural period of the building is T=0.324 second in the x direction, which is computed according to the procedure adapted by Martinez (2007) for CFS buildings.

Shown in Table 2 are the target base shears corresponding to the specified four performance objectives, which are computed as functions of the zone seismic parameters and natural period of the building. After the base shears are computed, the lateral load for the pushover analysis is defined by the maximum target base shear $V_{b\,max}$=730.70 kN. The lateral loads are applied on the building in 1% increments of 7.307 kN, until the maximum target base shear is reached or the structural collapse occurs.

Table 2 Target base shears (kN)

PO	S_S	S_1	S_{MS}	S_{M1}	S_{DS}	S_{D1}	T_o	T_s	S_a	V_b
	(g)						(sec)		(g)	(kN)
OP	0.109	0.035	0.109	0.035	0.073	0.023	0.064	0.321	0.073	**71.75**
IO	0.180	0.058	0.180	0.058	0.120	0.039	0.064	0.322	0.120	**118.91**
LS	0.250	0.080	0.250	0.080	0.167	0.053	0.064	0.320	0.167	**164.01**
CP	1.100	0.410	1.100	0.410	0.733	0.273	0.075	0.373	0.733	**730.70**

Presented in Table 3 are the inter-storey drifts and limit drifts for the SWP related to each performance objective, which are computed by the limit drift ratios 0.2%, 1.0%, 2.1% and 2.5% for the OP, IO, LS and CP performance level, respectively. The limit inter-storey drifts are obtained by multiplying the lateral drift ratios to the height of the SWP. SWP 1 and 3 exhibit the largest lateral drifts, because the seismic loads are applied on these panels. It is found that both the lateral drifts and the lateral strengths of SWP 1 and 3 satisfy the specified performance objectives.

Table 3 SWP inter-storey drift (mm)

SWP	Performance objective											
	OP (limit=4.8)			IO (limit=24.0)			LS (limit=50.4)			CP (limit=60.0)		
	Storey			Storey			Storey			Storey		
	1	2	3	1	2	3	1	2	3	1	2	3
1	0.34	0.39	0.43	0.76	0.77	0.76	1.22	1.19	1.10	21.40	14.84	9.44
3	0.36	0.40	0.43	0.78	0.78	0.76	1.25	1.20	1.11	21.50	14.89	9.46
6	0.34	0.39	0.43	0.75	0.76	0.76	1.22	1.18	1.11	21.38	14.81	9.48
7	0.35	0.39	0.43	0.76	0.77	0.75	1.23	1.19	1.10	21.43	14.85	9.41
9	0.35	0.39	0.43	0.77	0.77	0.75	1.24	1.19	1.10	21.47	14.87	9.41

Indicated in Table 4 are the lateral strengths P_R and lateral forces P_a for the SWP at different performance objectives. The lateral strengths are computed in accordance with method proposed by Xu and Martinez (2006), and the lateral forces are obtained from the pushover analysis. Since the lateral forces in the SWP are less than the corresponding strengths, the performance objectives are deemed to be satisfied. The largest forces are obtained for the CP performance objective, since it is associated with the maximum target base shear. It is observed from the table that the summation of lateral forces in the SWP is different from the lateral loads applied on the building, i.e., for the CP performance objective the summation of forces is 677 kN while the lateral force applied on the building is 730 kN. The reason for the difference is that the panels in the *y* direction (i.e., perpendicular to the SWP 1, 3, 7 and 9) are resisting a small proportion of the lateral loads. The loads that are resisting by SWP 2, SWP4 and SWP5 are 23.79 kN, 14.85 kN and 14.77 kN, respectively.

Listed in Table 5 is the stiffness degradation coefficient, represented by λ, for each SWP. It can be observed that none of the panels has failed, although several panels on the first storey are very close to failure for the CP performance objective due to the loss of lateral load resisting capability ($\lambda \leq 0.1$).

From the results of the spectrum-based pushover analysis, it is observed that the building satisfies all seismic requirements. The four performance objectives are deemed to be satisfied since the inter-storey drifts and lateral strengths for the SWP are less than the limit values. Therefore, the PBD assessment for seismic forces is completed, and the lateral design of the building does not require modification.

Table 4. SWP lateral strength P_R and lateral force P_a (kN)

SWP	P_R	Lateral force, P_a											
		OP			IO			LS			CP		
		Storey			Storey			Storey			Storey		
		1	2	3	1	2	3	1	2	3	1	2	3
1	212.80	9.14	7.66	4.31	17.89	14.77	8.60	26.66	21.91	12.89	171.46	140.23	85.71
3	212.80	9.29	7.77	4.18	18.13	14.92	8.47	26.97	22.08	12.76	171.71	140.41	85.66
7	212.80	8.14	6.62	4.47	16.93	14.02	9.10	25.64	21.34	13.67	166.81	138.51	87.89
9	212.80	8.43	6.75	4.43	17.23	14.16	9.06	25.95	21.49	13.64	167.02	138.67	87.86

Table 5. SWP stiffness degradation coefficient λ

SWP	OP			IO			LS			CP		
	Storey			Storey			Storey			Storey		
	1	2	3	1	2	3	1	2	3	1	2	3
1	0.69	0.71	0.77	0.60	0.63	0.71	0.54	0.57	0.66	0.08	0.14	0.30
2	0.93	0.95	0.93	0.92	0.95	0.91	0.91	0.94	0.90	0.85	0.90	0.80
3	0.69	0.71	0.78	0.60	0.63	0.71	0.54	0.57	0.66	0.08	0.14	0.30
4	0.82	0.76	0.75	0.82	0.76	0.75	0.82	0.76	0.75	0.79	0.74	0.75
5	0.84	0.77	0.75	0.84	0.77	0.74	0.84	0.77	0.74	0.85	0.75	0.71
7	0.70	0.73	0.77	0.61	0.64	0.70	0.55	0.58	0.65	0.09	0.15	0.30
9	0.70	0.72	0.77	0.61	0.64	0.70	0.54	0.58	0.65	0.09	0.15	0.30

Illustrated in Figure 6 is a comparison of the inter-storey drifts and lateral displacements for SWP 1, obtained from the pushover analysis and the linear elastic analysis at the level of CP performance objective.

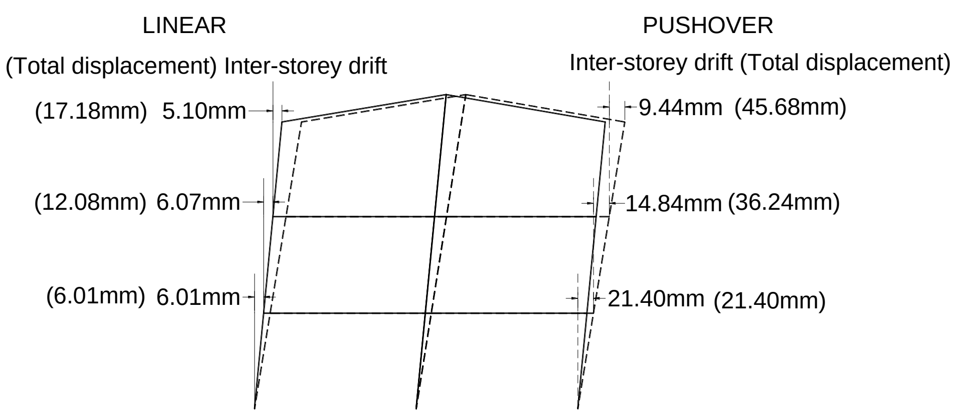


Figure 6. Inter-storey drifts and displacements of SWP 1: pushover vs. linear analysis

CONCLUSIONS

One of the primary difficulties to conduct the PBD assessment of a structural system is to establish the acceptance criteria and levels of damage associated with the performance levels of the lateral load resisting components. In this study, the acceptance criteria of CFS buildings for different performance levels are established according to the lateral drift and strength of the SWP. To account for the nonlinear behaviour of the SWP, a stiffness degradation model was introduced in proposed push-over analysis which captures very well the nonlinear behaviour of the SWP. In addition, the model monitors the stress in the SWP during the analysis and predicts the failure of the SWP.

The results obtained from the proposed PBD assessment method of the presented example were compared to that of the conventional linear analysis. At the CP performance level, the building was subjected to the same loading condition for both types of analysis. The maximum lateral displacement of the building obtained from the pushover analysis was 2.65 times larger than that of the linear analysis which greatly underestimated the lateral drifts of the building. Therefore, considering the fact there is no appropriate method for assessing the seismic performance of CFS buildings in current practice, the proposed PBD assessment method is recommended for the seismic assessment of low- and mid-rise CFS buildings.

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