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FEMA P695 seismic performance evaluation of Cold-formed steel buildings with different shear wall strengths and stiffnesses

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ABSTRACT

Lateral load capacity and stiffness of CFS framed OSB sheathed shear walls can be improved by increasing the number of fasteners between sheathing panel and framing members or by providing sheathing on both sides of panel. Both of these methods could be a viable option to meet high seismic demands when the number of walls is limited by architectural or other concerns. With the increasing use of CFS building systems in high seismic regions, such applications are expected to be more widespread in the future. The present study aims to investigate the seismic response of CFS framed archetype buildings utilizing OSB sheathed wall panels with different levels of stiffness and shear resistance. Nonlinear time history analyses were conducted on 28 archetype buildings and collapse performance evaluation was performed according to FEMA P695 methodology. Load-displacement hysteresis behaviors of wall panels measured in a recent experimental study were adopted to simulate the shear wall response in the numerical models. Buildings utilizing shear walls that are sheathed with OSB panels on one side with 300 mm fastener spacing possess limited overstrength and fail to satisfy the performance criteria regardless of the response modification coefficient used for design. With the use of shear walls that are constructed with 50 mm fastener spacing and single or double sided sheathing configurations, the building design is governed by drift limitation instead of strength requirement. Irregular drift profiles occur with lateral displacements localizing at the first floor when shear walls with single sided sheathing and loose fastener layout is utilized. Using shear walls with a dense fastener layout results in a uniform distribution of lateral displacements among floors.

1. Introduction

Cold-formed steel (CFS) framed structural systems have become a popular alternative in constructional steel industry in recent years. This type of systems offers the advantages of high strength to weight ratio, recyclability, sustainability, fast construction and relatively low construction cost. The main lateral load resisting elements in these systems are wall panels that are either sheathed with various sheathing materials or braced by steel straps.

Seismic design provisions for CFS framed buildings are provided by several design standards around the world. These standards adopt capacity design rules in order to ensure elastic response of non-dissipative elements and sufficient energy dissipation in selected elements to develop plastic mechanisms, eventually resulting in a ductile behavior of the structure. The standards also describe a force reduction factor to be used in design against seismic actions to account for overstrength and ductility of the selected lateral force resisting system. This force reduction factor is described as response modification coefficient (R) in North American Standard for Seismic Design of CFS Structures (AISI S400) [1] and ASCE 7 [2], while it is described as behavior factor (q) in Eurocode 8 (EN 1998) [3]. It is worth mentioning that while ASCE 7 specifies R values for CFS lateral force resisting systems, EN 1998-1 does not explicitly detail the seismic design procedure for structures with CFS walls.

North American standards specify the seismic design parameters to be used for structures with CFS framed wood sheathed shear walls. In USA and Mexico, it is allowed to use the response modification coefficient (R) as 6.5, system over-strength factor (Ω_o) as 3 and deflection amplification factor (C_d) as 4. In Canada, R value is designated as 4.25, C_d as 4.25 and Ω_E as 1.33. In the latest edition of the Turkish Seismic Design Code for Buildings (TBDY 2018) [4] R and Ω_o values are given as 4 and 2, respectively. The deflection amplification factor C_d is implicitly

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taken to be equal to R. These values differ from the corresponding values in the North American standards, even though the majority of the provisions specified for CFS building systems in TBDY 2018 were adopted from these standards.

Federal Emergency Management Agency (FEMA) P695 [4] methodology describes analytical methods and a procedure to evaluate the seismic performance of building systems. The procedure involves structural design of archetype buildings according to relevant specifications and by using selected response modification coefficient. Nonlinear models of these archetypes are then subjected to pushover analyses for determination of overstrength and ductility. The same archetypes are also subjected to nonlinear dynamic analyses with multiple ground motion records for evaluation in terms of collapse capacity. The FEMA P695 methodology allows for the evaluation of selected response modification coefficient for the selected lateral force resisting system. CFS building systems have been the subject of many recent studies on collapse performance evaluation by following the FEMA P695 [4] methodology. Shamim and Rogers [5] evaluated the seismic performance of multi-story CFS building archetypes with steel sheathed CFS wall panels considering the FEMA P695 methodology and recommended the ductility and overstrength related force modification factors of $R_d = 2$ and $R_o = 1.3$ producing a R value of 2.6. Another recent study also included steel sheathed shear walls and validated the codified R value of 4.25 [6]. Morello [7] considered four, six and seven story CFS framed structures with wood sheathed shear walls and reported acceptable seismic performance with $R = 4.25$. Vigh et al. [8] studied archetype buildings with CFS shear walls that were sheathed with corrugated steel-sheathing. The buildings were designed according to ASCE 7 by using a response modification coefficient of $R = 4$. The archetypes were reported to meet the FEMA P695 acceptance criteria. Seismic response of a two-story CFS building with oriented strand board (OSB) sheathed wall panels was evaluated within the CFS-NEES research program initiated in North America [9,10]. A full scale office building was studied through shake table tests and three-dimensional finite element models utilizing different levels of detail. It was shown that a better agreement between the experimental and numerical responses was obtained when the lateral force resistance of gravity system and nonstructural components were included in computational models. There are many studies completed in Europe considering CFS structures with various types of shear wall panels. Shakeel et al. [11], Fiorino et al. [12] and Shakeel et al. [13] considered buildings with wood sheathed, strap braced and gypsum board sheathed wall panels. These researchers evaluated the performance of selected archetypes through static pushover and Incremental Dynamic Analysis (IDA) procedures according to FEMA P695 [4] and developed proper behavior factor values. For CFS buildings with OSB sheathed wall panels, a behavior factor of 2.5 was proposed. Landolfo et al. [14] provided a review of previous studies to present a set of design rules for seismic design of CFS buildings with the intention of filling the gap in Eurocode 8 [3] on this subject. Kechidi et al. [15] evaluated the performance of 54 CFS buildings with sheathed wall panels that were designed with varying seismic intensity levels. The investigated CFS buildings were reported to meet the acceptance criteria for low and moderate seismicity levels with a behavior factor of 2.

1.1. Motivation for present study

As shown by earlier research, lateral load capacity and stiffness of CFS framed shear walls can be improved by increasing the number of fasteners between sheathing panels and framing members [16–22] or by providing sheathing on both sides of panels [22,23]. Both of these methods could be a viable option to meet high seismic demands when the number of walls is limited by architectural or other concerns. With the increasing use of CFS building systems in high seismic regions, such applications are expected to be more widespread in the future. The present study aims to investigate the seismic response of CFS framed archetype buildings utilizing OSB sheathed wall panels with different sheathing and fastener configurations. Studies available in the literature on seismic collapse analysis of buildings with OSB sheathed CFS framed shear walls are either limited with walls sheathed on one side with 150/300 mm fastener spacing [10,11,14], or do not provide an analysis of the influence of shear wall configuration on building response [15]. The present study contributes to the existing literature by reporting on the seismic response of CFS building systems with different shear wall configurations.

For a proper evaluation of the selected structural system, FEMA P695 recommends the consideration of archetypes with a wide range of parameters. Aligned with this recommendation, CFS building archetypes designed with different occupancy types and the number of stories have been utilized in the current study for their seismic performance evaluation. On the other hand, the major focus of this work is the assessment of CFS framed buildings utilizing shear wall panels with different load capacities and stiffnesses. Test data reported by various researchers [16–23] on OSB sheathed CFS framed wall panels indicate that increasing the number of connection fasteners and providing sheathing on both sides of panels lead to different levels of increase on the lateral load capacity and stiffness of the panels. For example, recent tests by the authors revealed that decreasing the spacing of connection fasteners from 150 mm to 50 mm results in three times increase in shear strength of wall panel, however the increase in wall stiffness is only 1.6 times [22]. Similarly, the beneficial effect of providing sheathing on both sides of panels was less on wall stiffness than on shear strength. These observations clearly indicate that the relation between the strength and stiffness of wall panels on the overall performance of CFS framed buildings is an important parameter to be considered in lateral load design. In other words, depending on the relation between wall shear strength and stiffness (i.e., layout of connection fasteners and the number of sheathing used in wall panels), lateral design of the building system can be governed by the drift requirement as opposed to the strength requirement. However, a review of studies on seismic performance evaluation of CFS framed buildings [4–8,11–13,15] indicates that their archetypes are based on strength governed design. Therefore, there is a lack of comprehensive investigation on the assessment of CFS building systems utilizing shear walls with different strength and stiffness levels. The current study aims to fill this gap in the literature through FEMA P695 collapse performance evaluation of CFS buildings with shear wall panels utilizing double sided sheathing and dense fastener layout, as well as conventional wall panels. A numerical study has been conducted on a total of 28 archetype buildings. Load-displacement hysteresis behaviors of wall panels measured in a recent experimental study were adopted to simulate the shear wall response in the numerical models utilized in the present study.

2. Details of archetypes

The response modification coefficient, occupancy type, number of stories, number of sheathing panel and number of fasteners were considered as variables in the design of archetypes. A total of 28 archetypes, details of which are given in Table 1, were taken into account. Archetypes were designed to have either office type or residential type of occupancies. Most of the archetypes had two stories, while some three story residential archetypes were also considered. These variables were also considered while naming the archetypes such as; O stands for office, R stands for residential, W describes the type of shear wall used in the building and S indicates the number of stories for residential type archetypes. In nine of the office archetypes different types of wall panels were utilized in two floors. In these cases, wall panels with smaller strength and stiffness were considered at the second floor than those at the first floor. Variation in wall strength and stiffness was achieved by changing the number of sheathing panel as well as the number of fasteners providing connection between CFS framing and sheathing panels, as discussed earlier.

The archetype naming convention given in Table 1 indicates the

Table 1
Details of archetype buildings.

Archetype	R	Wall Configuration	Occupancy Type	Building Height	Number of walls for each direction	Design shear strength of wall panel (kN)	Design base shear V_d (kN)	Design overstrength
O6.5W1	6.5	SW1	Office	5.48 m (2-story)	24	6.75	150	1.08
O6.5W2	6.5	SW2			12	20.12	150	1.61
O6.5W3	6.5	SW3			10	31.70	150	2.11
O6.5W2-1	6.5	SW2-SW1			12	20.12	150	1.61
O6.5W3-1	6.5	SW3-SW1			12	31.70	150	2.54
O6.5W3-2	6.5	SW3-SW2			10	31.70	150	2.11
O5W1	5	SW1			30	6.75	194	1.04
O5W2	5	SW2			16	20.12	194	1.66
O5W3	5	SW3			12	31.70	194	1.96
O5W2-1	5	SW2-SW1			16	20.12	194	1.66
O5W3-1	5	SW3-SW1			14	31.70	194	2.29
O5W3-2	5	SW3-SW2			12	31.70	194	1.96
O4W1	4	SW1			36	6.75	243	1.00
O4W2	4	SW2			20	20.12	243	1.66
O4W3	4	SW3			14	31.70	243	1.83
O4W2-1	4	SW2-SW1			20	20.12	243	1.66
O4W3-1	4	SW3-SW1			18	31.70	243	2.35
O4W3-2	4	SW3-SW2			14	31.70	243	1.83
R2S-6.5W1	6.5	SW1	Residential	5.48 m (2-story)	14	6.75	90	1.05
R2S-6.5W2	6.5	SW2			8	20.12	90	1.79
R2S-6.5W3	6.5	SW3			6	31.70	90	2.11
R2S-4W1	4	SW1			22	6.75	147	1.01
R2S-4W2	4	SW2			12	20.12	147	1.64
R2S-4W3	4	SW3			10	31.70	147	2.16
R3S-6.5W2	6.5	SW2		8.22 m (3-story)	12	20.12	142	1.70
R3S-6.5W3	6.5	SW3			10	31.70	142	2.23
R3S-4W2	4	SW2			18	20.12	231	1.57
R3S-4W3	4	SW3			14	31.70	231	1.92

occupancy type, the response modification coefficient used in design, the type of shear walls and the number of floors. For example, O6.5W3-1 designates the office type archetype designed with a response modification coefficient of R = 6.5 and includes SW3 type shear walls at the first floor and SW1 type shear walls at the second floor. Similarly, R3S-4W2 designates the three-story residential archetype designed with a response modification coefficient of R = 4 and with W2 type shear walls at all floors.

2.1. Occupancy type and height of buildings

Two types of occupancies, namely office and residential were considered for archetype buildings with different structural plans, framing span lengths and live loads acting on the structure. Floor plans shown in Fig. 1 were considered for office and residential buildings based on the floor plans used in earlier studies [11,14]. A typical story height of 2.74 m (i.e., 2.44 m wall height and 0.3 m floor height) was considered in building designs. All office building archetypes had two stories with a total height of 5.48 m, while residential building archetypes had either two or three stories with a total height of either 5.48 m or 8.22 m.

2.2. Seismic hazard level

FEMA P695 [4] describes three main Seismic Design Categories (SDC) as B, C and D in accordance with ASCE 7–16 [2]. The methodology also describes earthquakes at two levels; Design Earthquake (DE) and Maximum Considered Earthquake (MCE) for each seismic design category. In the regular methodology, analyses are repeated for low, medium and high seismicity levels, however in the present study all archetypes were considered to be located in a high seismicity location. The analyses were conducted considering Seismic Design Category D_{max}, which represents the highest possible seismic condition. For SDC D_{max}, values of the MCE spectral response acceleration parameter S_{MS} and design spectral response acceleration parameter S_{DS} were adopted respectively as 1.50g and 1.0g.

2.3. Details of shear walls

Three different CFS shear wall types, designated as SW1, SW2 and SW3 were used for the lateral load resisting system of archetype structures. Representation of shear walls is given in Fig. 2. Each wall panel is 1.22 m in length and 2.44 m in height and sheathed with 11 mm thick OSB panels that are connected to wall framings with 4.2 mm diameter screws. Framing of wall panels consists of 1.2 mm thick 140 mm deep C-shaped sections that are made of S350 grade steel. Each of the shear wall configurations represents a different level of shear strength. SW1 has single-sided OSB sheathing with 150 mm screw spacing at panel edges. This spacing represents the maximum spacing value provided in AISI S400-15 [1] Nominal Shear Strength Table E1.3-1, and results in the lowest shear strength among three shear walls utilized in the present study. Shear wall SW2 has single-sided OSB sheathing with a relatively small screw spacing of 50 mm. Shear wall SW3 on the other hand, has OSB sheathing on both sides of the panel with 50 mm screw spacing, producing the largest shear strength and stiffness of all three wall configurations. The experimentally determined load-displacement hysteresis behaviors of the shear walls are given in Fig. 3. More information on the measured response of walls under lateral loading can be found in Refs. [22,25]. Superimposed on the plots in Fig. 3 are cyclic backbone curves and bilinear load-displacement curves determined based on the Equal Energy Elastic-Plastic (EEEP) method as specified in AISI S400-15 [1]. The relatively flexible response of shear walls is evident in the plots. For the three wall panels, the drift ratio at first yield ranges from 1.0% to 2.0% and the drift ratio at maximum load capacity ranges from 2.6% to

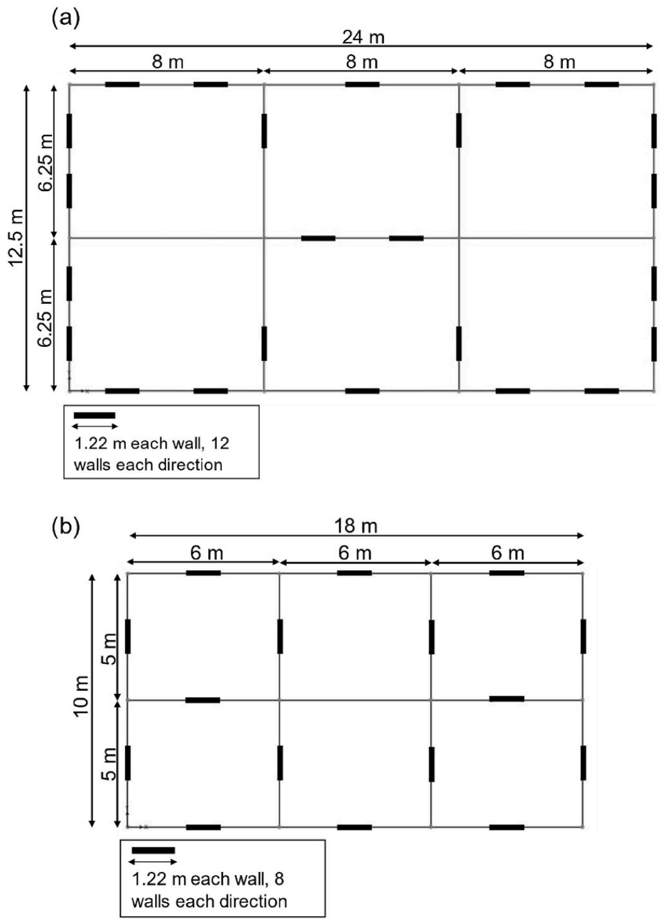

Fig. 1. Example floor plans: a) office building; b) residential building.

3.0%.

Since the OSB sheathed CFS shear walls constitute the main lateral load resisting components in the investigated archetype buildings, their behavior in terms of stiffness and shear strength directly affects the overall seismic performance of buildings. The experimentally determined stiffness and shear strength of the three shear wall types considered for the analyses are given in Table 2. The table presents elastic stiffness K_e, yield load capacity P_y and design load capacity P_{design} obtained from the EEEP approach [1]. The maximum load capacity P_{max} obtained from tests and nominal shear strength calculated according to AISI S400-15 [1] are also provided. Load capacities obtained for shear walls SW1 and SW2 are in good agreement with codified values. AISI S400-15 suggests that with double-sided sheathing, load capacity of a wall panel can be considered to be two times the nominal shear strength of a single side sheathed panel provided that minimum stud and track thickness and fastener size requirements are satisfied. For shear wall SW3 however, buckling of end studs prevented the shear wall to reach the codified shear strength value [22]. It should be noted that the amount of changes in wall stiffness and shear strength differ among the three wall panels. For instance, decreasing the screw spacing from 150 mm (in SW1) to 50 mm (in SW2) results in three times increase in shear strength of wall panel, however the increase in wall stiffness is only 1.6 times. The relation between shear wall stiffness and load capacity is a parameter of significant importance for performance evaluation of CFS building systems, and to the knowledge of the authors the effect of this parameter has not been investigated so far. For this reason, the shear wall type was considered as the main parameter in the current study.

3. Archetype design

All archetype structures were designed using relevant parts of ASCE 7–16 [2], AISI-S100 [26] and AISI-S400 [1]. As defined in ASCE 7 [2], a floor live load of 2.4 kPa was considered for office buildings, whereas for residential buildings a floor live load of 1.9 kPa was used. In addition to that, roof slab in residential buildings was considered to be accessible while an inaccessible roof slab was considered for office buildings. Roof live load was taken as 1.9 kPa and 1.0 kPa, respectively for residential and office buildings. Two sets of dead loads were adopted for office and residential buildings as shown in Table 3. These values are similar to the loadings used earlier by other researchers in analysis of CFS building systems [5,11,14].

The archetypes were designed by adopting three different response modification factors of $R = 6.5$, 5 and 4, and with a deflection amplification factor $C_d = 4$. The R values of 6.5 and 4 are upper and lower bound values described in specifications covering CFS wood sheathed

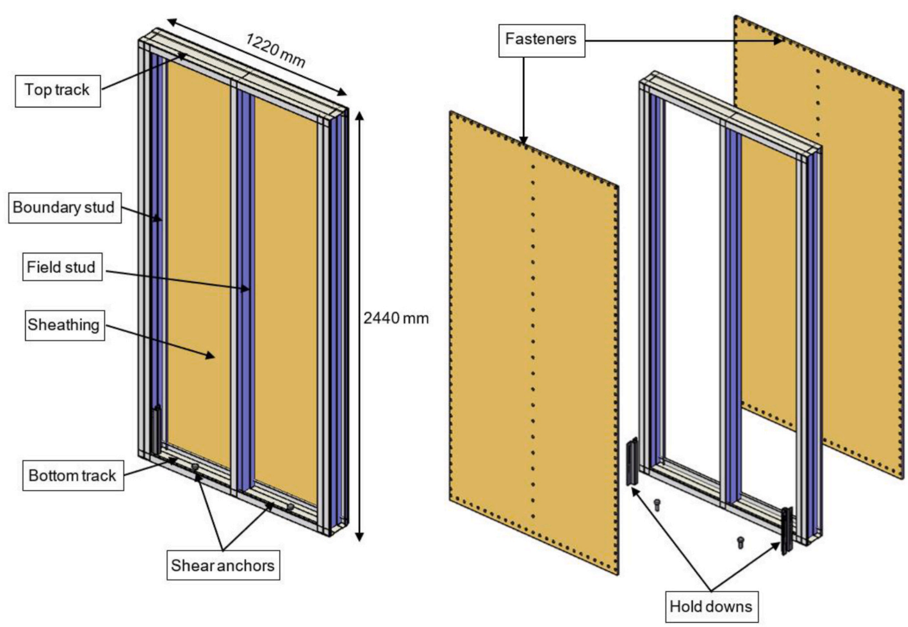

Fig. 2. Details of wall panels.

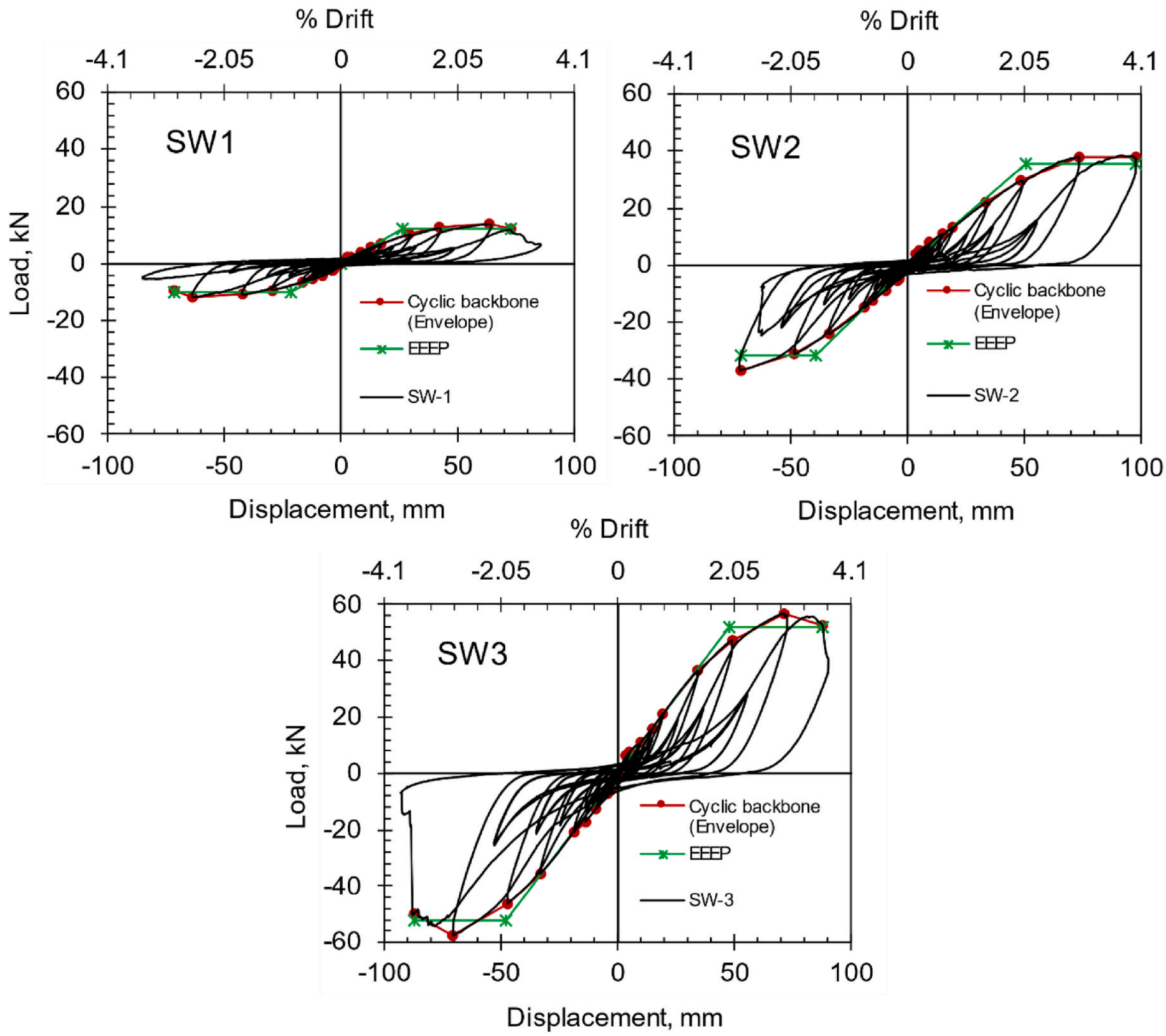

Fig. 3. Load-displacement behavior of CFS shear walls used in archetype buildings.

Table 2
Shear wall stiffness and strength values.

	SW-1	SW-2	SW-3
K_e (kN/mm)	0.47	0.75	1.10
P_{yield} (kN)	11.25	33.54	52.80
P_{design} (kN)	6.75	20.12	31.68
P_{max} (kN)	14.15	38.7	57.34
Nominal shear strength per AISI (kN)	14.64	36.60	73.20

Table 3
Dead load values used in archetypes.

Office				Residential			
Floor DL (kPa)		Roof DL (kPa)		Floor DL (kPa)		Roof DL (kPa)	
Floor system	1.4	Roof system	1.0	Floor system	1.2	Roof system	1.3
Vertical partitions	0.6	Ceiling	0.1	Vertical partitions	0.6	Ceiling	0.1
Ceiling	0.1			Ceiling	0.1		
Total	**2.1**	**Total**	**1.1**	**Total**	**1.9**	**Total**	**1.4**

shear walls (i.e. $R = 6.5$ for the USA, $R = 4.25$ for Canada and $R = 4$ for Turkey). The deflection amplification factor C_d is specified as 4 for the USA and Turkey and 4.25 for Canada. In the current study the value of $C_d = 4$ was adapted.

As part of the seismic design procedure, the required number of shear walls for each archetype structure was determined based on the calculated base shear. The approximate fundamental period T_a was calculated for two and three story archetype buildings as 0.17 s and 0.24 s respectively, by using Eq. (1). With these period values the designed archetypes remain in the constant acceleration region of the response spectrum. Accordingly, seismic response coefficient C_s was calculated for each archetype building with the selected response modification coefficient by using Eq. (2) and total base shear was calculated by using Eq. (3). The number of required shear walls in each direction was then determined by dividing the base shear to design shear strength of selected wall panel type. The design shear strength of wall panels was calculated by multiplying the yield load capacities obtained from the EEEP method [1] with the code specified resistance factor of 0.60. With the number of shear walls determined this way, an initial 3-D analysis model of each archetype building was created. As the second step, linear elastic analysis was performed for each archetype model with the seismic forces distributed over the height of building according to Eq. (4).

$$T_a = C_t h_n^x \tag{1}$$

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \tag{2}$$

$$V = C_s W \tag{3}$$

$$F_x = V_d \frac{w_x h_x}{\sum_i^n w_i h_i} \tag{4}$$

In Eqs. (1)–(4) C_t and x are approximate period parameters taken respectively as 0.0488 and 0.75, h_n is the structural height, S_{DS} is the design spectral response acceleration and was taken as 1.0 g, R is the response modification coefficient, I_e is the importance factor and was taken as 1, W is the effective seismic weight, V_d is the total design base shear, w_i and w_x are seismic weights assigned to level i and x, h_i and h_x are heights to level i and x from the base.

Interstory drift values obtained from elastic analysis were amplified by a deflection amplification factor of $C_d = 4$ and were checked against the allowable interstory drift of $0.025h_{sx}$ per Table 12.12-1 of ASCE 7–16 [2], where h_{sx} is the story height and taken as 2.74 m for all archetypes. The number of shear walls was increased as required to satisfy the interstory drift requiresment. Other than archetypes in which shear wall type SW1 was used, interstory drift limit happened to be the governing criteria, and the number of shear walls was increased beyond what was required for strength requirement for these archetype buildings. For three office and two residential buildings with SW1 type shear walls, the number of shear walls calculated based on the base shear for each horizontal direction already satisfied the interstory drift limit. Important design parameters used for design of archetype buildings are presented in Table 1. Design overstrength Ω_{design} shown in the table for each archetype was calculated by dividing the total design shear strength of all wall panels at first floor by the design base shear, V_d (Eq. (5)). Total design shear strength was determined by multiplying the number of walls, n_{wall} with design shear strength of wall panels, P_{design} presented in Table 2. The design overstrength value calculated with Eq. (5) this way actually represent the ratio of the number of walls provided in archetype building to the number of walls required by the strength requirement.

$$\Omega_{design} = \frac{n_{wall} \bullet P_{design}}{V_d} \tag{5}$$

Buildings utilizing SW1 type shear wall at first floor have very limited design overstrength value (i.e., close to 1.0) due the fact that the design of these buildings was governed by the shear strength requirement. For the remaining buildings, on the other hand, drift requirement governed the design, which resulted in much higher design overstrength.

Table 1 shows a notable decrease in the required number of shear walls as the response modification coefficient increases. Additionally, archetypes designed with SW1 type walls necessitated approximately twice the number of shear walls compared to those with SW2 type walls. In some instances, the number of SW1-type shear walls required for a specific archetype reached exceptionally high values, such as archetype O4W1, where as many as 36 shear walls were needed in each direction. It is worth mentioning that this archetype represents an office-type building with relatively large dimensions, enabling the placement of shear walls within the designated plan dimensions without the need for any alterations to the building's layout. Utilizing SW3 type walls further decreased the required number of shear walls slightly when compared to the archetypes with SW2 type walls.

4. Numerical modeling of archetypes

For a structure with CFS framed sheathed wall panels as the main lateral load resisting system, the overall nonlinear seismic response is dictated by the response of individual wall panels. For this reason, accurate representation of the nonlinear hysteretic response of wall panels has a crucial importance in numerical modeling. In the present study, nonlinear analysis of archetype buildings was conducted by using SAP2000 computer program. Modeling process consisted of two steps. First, single wall models for all three shear wall panels were created with available experimental data. Following that, these single wall models were utilized to create three dimensional models of archetype buildings.

4.1. Single wall panel models

Single shear wall models were created for walls SW1, SW2 and SW3, and were calibrated with available experimental test results. A representation of the numerical model is shown in Fig. 4. Diagonal nonlinear link elements were used with a pivot type hysteresis behavior to represent pinched cyclic behavior of shear walls. Pivot model utilizes several parameters to create pivot points and control the degrading hysteretic loop. For each shear wall type, the points on the backbone curve obtained from experimental testing were used to define the load-displacement backbone curve of nonlinear wall models. Furthermore, accurate representation of the pinched cyclic behavior of shear walls was achieved by utilizing pivot model parameters. Values of these parameters were optimized through an iterative process until the simulated shear wall behavior demonstrated significant agreement with the experimental data. The parameters used in the current study to define the pivot hysteresis model are $\alpha_1 = 10$, $\alpha_2 = 10$, $\beta_1 = 0.5$, $\beta_2 = 0.5$, and $\eta = 0$. Force and displacement values obtained from experimental testing were transformed by using Eqs. (6)–(8) and were used for calibration of hysteresis parameters.

$$cos\ \theta = \frac{W}{\sqrt{W^2 + H^2}} \tag{6}$$

$$f = \frac{P}{2\ cos\ \theta} \tag{7}$$

$$d = \Delta \bullet cos\ \theta \tag{8}$$

where, θ, W, H, f, P and Δ are as indicated in Fig. 4.

Linear frame elements with corresponding CFS section properties were used to represent stud and track members. Hold downs that are connecting shear walls to the foundation were modeled as zero-length nonlinear link elements. Multilinear elastic material property was assigned to these link elements based on the available experimental data on hold down devices. A hold down device that was manufactured in the laboratory by the authors and investigated in a previous study [25] was used for shear wall models. An axial tensile stiffness value of 10 kN/mm, which was determined based on experimental results, was used to represent the behavior of the hold down. A relatively large axial compressive stiffness of 10,000 kN/mm was used for hold down elements.

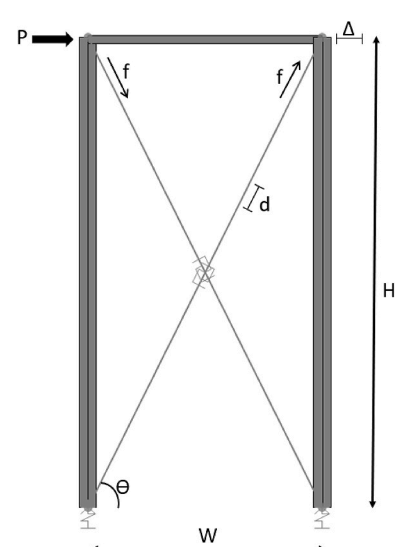

Fig. 4. Single wall numerical model.

Load-displacement responses obtained from numerical models are compared with the experimentally determined responses in Fig. 5 for all three wall panels. In order to facilitate the direct comparison between the two sets of results, the same cyclic loading protocol that was used in experimental testing [27] was also utilized in wall numerical models. Energy dissipation is another parameter that has been used previously by other researchers for comparison of the experimentally determined and numerically predicted response of CFS walls [28]. A similar comparison for the wall panels utilized in the present study is provided in Fig. 6. The plots in Figs. 5 and 6 indicate that the numerical models accurately capture the actual load-displacement response as well as the energy dissipation characteristics of wall panels. It should be noted that the level of accuracy achieved is attributed to the construction of shear wall models based on the envelope curves that were derived from tested wall specimens. This type of modeling approach necessitates the availability of load-displacement data for each shear wall configuration utilized in the archetype to achieve precise calibration. Despite this requirement, the chosen shear wall modeling approach is particularly well-suited for large-scale modeling of CFS structures. A comparative analysis between this method and an alternative approach has already been included in a prior publication by the authors [22].

4.2. Three-dimensional building models

Different researchers have used various modeling approaches for seismic assessment of CFS building systems. Among those approaches the most frequently adopted one involves linear elastic modeling of CFS stud and track members with the nonlinear behavior concentrated on separate link elements [5,8,15]. In some studies, load capacity of these CFS members were limited to their respective capacities and some level of non-linearity was introduced [11,13]. Different approaches have also been adopted regarding the modeling of hold down devices. While the effect of these devices has been omitted in some of the studies [11], their non-linear behavior was included with distinct link elements in others [10,13]. In the current study, all structural elements, including CFS shear walls, studs, tracks and floor system were included in the building numerical models. All elements other than the shear walls were represented with linear elastic frame elements. It should be noted that among three different shear walls utilized in this study, shear wall SW3 experienced buckling at its chord studs during experimental testing. As the effect of this buckling behavior is already included in the experimentally determined cyclic behavior adopted for this wall, there was no need to assign additional non-linear behavior to stud members in the building numerical models. It can be argued that CFS wall studs in a building system can be subjected to larger axial force levels than in laboratory tests and may suffer from an unexpected buckling failure. This, however, can be avoided by proper sizing of CFS studs in wall panels during the design stage such that the maximum axial compressive force under the combined effects of seismic and gravity loading remain below the buckling capacity of these members.

For each building model utilized in the current study a rigid diaphragm was defined at floor levels. Moment releases were defined at the ends of stud members to prevent any moment transfer from floor system to these members. Base of wall panels were connected to link elements that are representing hold down devices, while pin support condition was adopted at the base of the gravity load carrying studs. A representative building numerical model is shown in Fig. 7.

Gravity loading, consisting of 100% of dead load and 50% of live

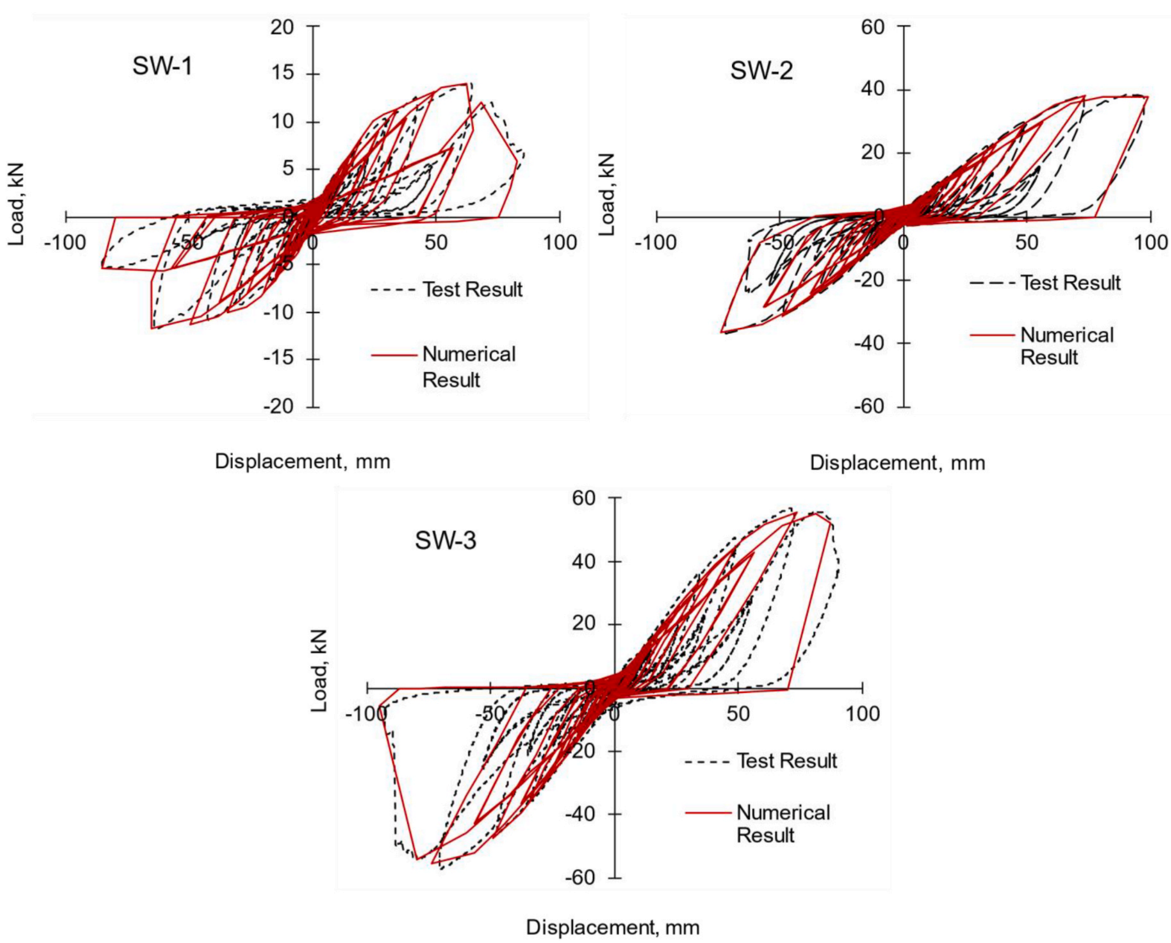

Fig. 5. Comparisonof predicted and measured load-displacement behaviors of wall panels.

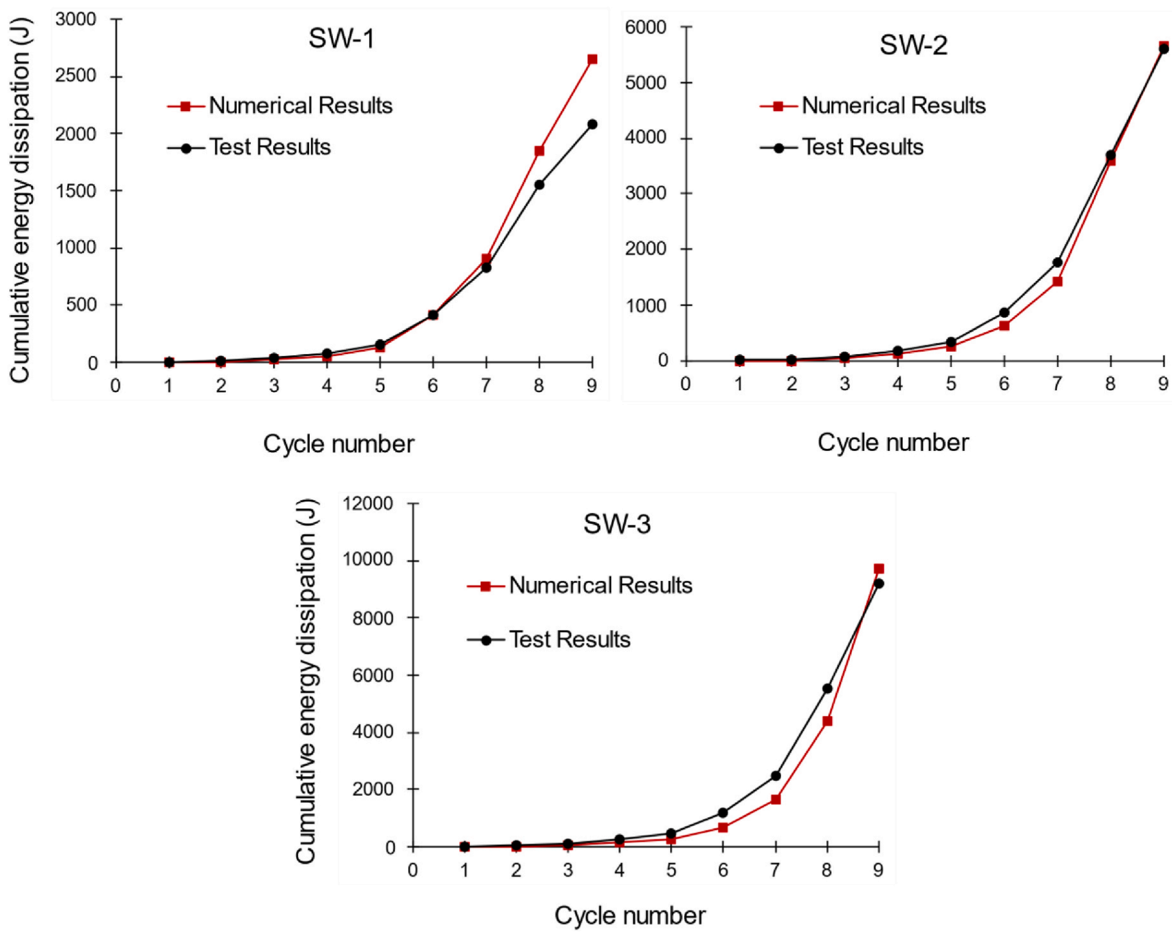

Fig. 6. Comparison of predicted and measured energy dissipation of wall panels.

load, was applied to each shear wall stud and gravity load carrying stud at story levels based on their tributary areas. Similarly, seismic mass, which was taken as 100% of dead load as per ASCE 7 [2], was equally distributed to the top joint of each stud element. P-delta effect was included in all lateral load analyses. For dynamic analyses, Rayleigh damping ratio was considered to be 5%, which is the value adopted in previous studies [10,15,29].

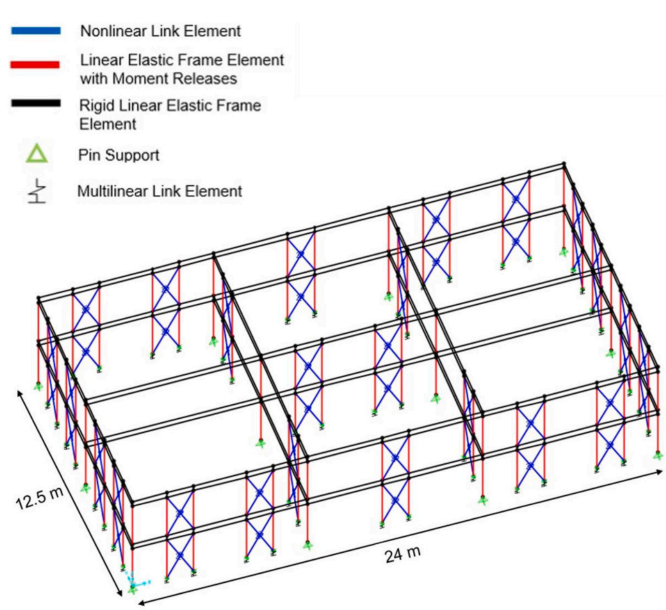

Fig. 7. Representation of archetype building numerical model.

5. Nonlinear analyses

5.1. Nonlinear static analyses

According to FEMA P695 methodology, the first step in the evaluation of building performance is to perform a pushover analysis in order to determine archetype overstrength and period-based ductility. The period-based ductility is then used to determine spectral shape factor (SSF) that is used in dynamic analyses. Pushover analyses were conducted for each archetype by utilizing a lateral load distribution across height that is in proportion to the fundamental mode shape. Displacement-controlled analysis was conducted with the central node at the top of the building being the controlling point.

Results from pushover analysis of nine office archetype buildings utilizing different wall panel types and designed with different response modification coefficients are presented in Fig. 8 and Table 4. The pushover curves resemble the backbone curves defined for shear walls, as expected. This is due to the fact that the shear walls are the main lateral load resisting elements and the only source of nonlinearity in building models. However, it should be noted that the pushover curves shown in Fig. 8 indicate the combined response from the first and second floor shear walls. The steep decline in the shear strength after the peak point for office archetypes with SW2 and SW3 type shear walls is related with the shape of backbone curves utilized for these walls. The sudden failure of walls SW2 and SW3 evident in Fig. 5 leads to the abrupt reduction in lateral load capacity of corresponding buildings in Fig. 8.

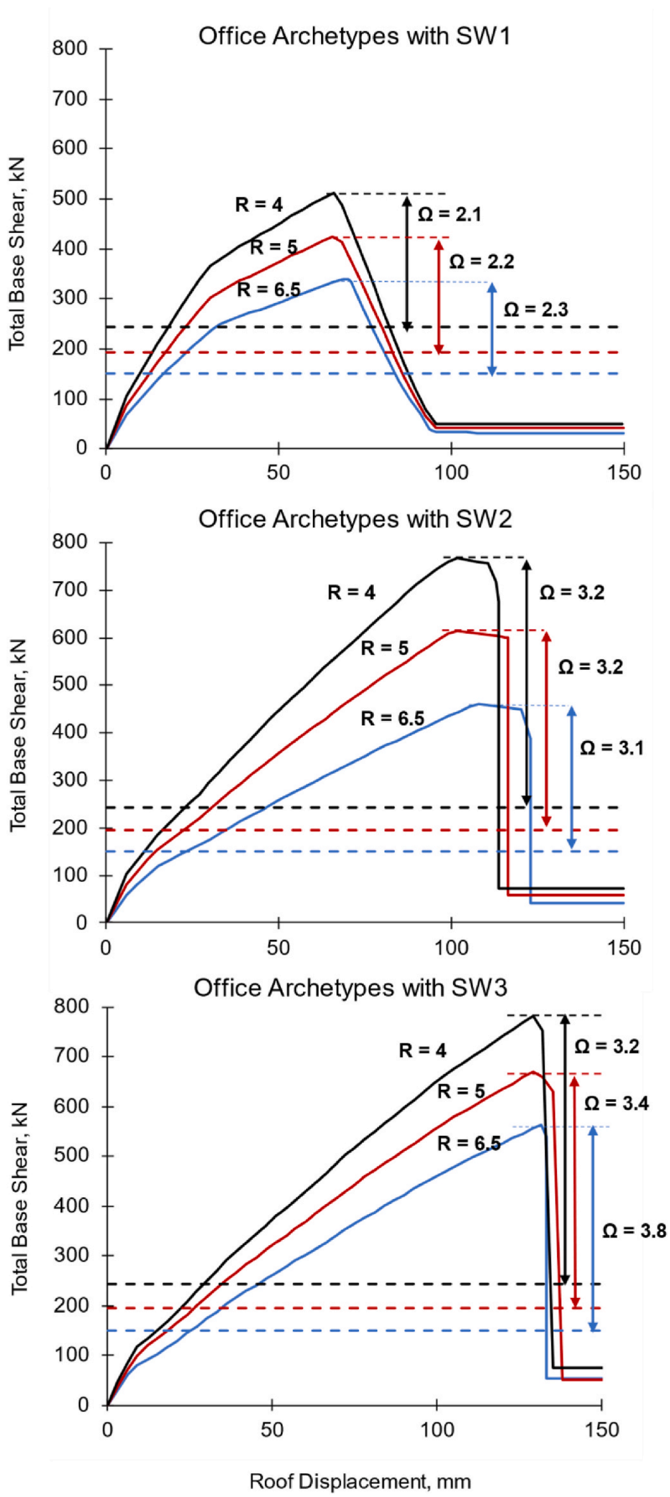

Fig. 8. Pushover analysis results for office archetypes.

The design base shear, fundamental period and base shear obtained from analysis as well as overstrength and period based ductility for each archetype building are presented in Table 4. Overstrength factor Ω is the ratio of the maximum base shear obtained from pushover analysis V_{max} to design base shear V_d (Eq. (9)).

$$\Omega = \frac{V_{max}}{V_d} \tag{9}$$

Another parameter that is obtained from pushover analysis is the period based ductility μ_T, which is defined as the ratio of ultimate roof displacement δ_{max} to the effective roof yield displacement $\delta_{y,eff}$ (Eq. (10)).

$$\mu_T = \frac{\delta_{max}}{\delta_{y,eff}} \tag{10}$$

Ultimate roof displacement δ_{max} is the displacement at roof level at the point of 20% strength loss from maximum base shear on pushover curve (i.e. at a base shear of $0.8V_{max}$). Effective roof yield displacement $\delta_{y,eff}$ is computed using Eq. (11) as defined in FEMA P695.

$$\delta_{y,eff} = C_0 \frac{V_{max}}{W} \left[\frac{g}{4\pi^2}\right] (\max(T, T_1))^2 \tag{11}$$

Pushover analyses were conducted separately for the two principal directions of buildings, as indicated in FEMA P695. The Ω and μ_T values reported in Table 4 are the average of the values obtained in two principal directions. It should be mentioned that values obtained in two directions are almost the same due to the number of walls being identical in both directions for each archetype building. Pushover analysis results indicate that the building models utilizing shear wall types SW2 and SW3 possess higher overstrength than those utilizing shear wall type SW1. This is due to the previously mentioned behavior that archetype design is governed by the strength requirement with SW1 shear walls and by the interstory drift requirement with SW2 and SW3 shear walls. Providing additional shear walls in order to meet the interstory drift requirement resulted in larger overstrength in the structural system in building models utilizing shear wall types SW2 and SW3. Overstrength values for each archetype are presented in Fig. 9. As evident, while the archetypes utilizing only SW1 type shear walls possess overstrength factors ranging between 2.1 and 2.3, utilizing SW2 and SW3 type shear walls leads to higher overstrengths of 3.0–3.8. On the other hand, design overstrength Ω_{design} values, indicating basically the ratio of the number of walls provided in archetype building to the number of walls required by the strength requirement, range between 1.0 and 2.5.

5.2. Nonlinear dynamic analyses

The second step of the performance evaluation in the framework of FEMA P695 is to perform non-linear dynamic analyses in order to determine collapse capacities of archetype buildings. The methodology utilizes incremental dynamic analysis (IDA), in which selected ground motions are applied on archetype buildings with increasing scale of intensities. The median collapse intensity S_{CT} and the collapse margin ratio (CMR) values are then determined from the IDA results. S_{CT} is defined as the earthquake intensity level at which half of the applied ground motions cause collapse of the building. CMR is the ratio of S_{CT} to the maximum considered earthquake S_{MT}. CMR value is modified by spectral shape factor (SSF) to calculate adjusted collapse margin ratio (ACMR). ACMR of the archetype is compared with the target ACMR value to evaluate the performance of the building.

The approach that was used for dynamic analyses and performance evaluation in the present study is slightly different than the general FEMA P695 procedure. Instead of performing incremental dynamic analysis, in which archetypes are analyzed under a set of scaled ground motions, ground motions were scaled up to the collapse level earthquake with a collapse margin ratio of 20%. This corresponds to the performance level defined by FEMA P695 as $ACMR_{20\%}$. While IDA is an effective method for design of new structural systems, scaling the ground motions with a predefined scaling factor deemed sufficient for the evaluation of existing buildings. Therefore, the latter approach was adopted in the current study. According to the FEMA P695 methodology, each individual archetype building is expected to have ACMR values larger than $ACMR_{20\%}$. The methodology describes another performance level corresponding to 10% collapse probability, indicated as $ACMR_{10\%}$. The average value of ACMR for a performance group is expected to be greater than $ACMR_{10\%}$. In the present study, each archetype

Table 4
Nonlinear static analysis results, SSF values and scaling factors for archetypes.

Archetype	T_1 (sec)	V_d (kN)	V_{max} (kN)	Ω (overstrength)	$\delta_{y,eff}$ (mm)	δ_u (mm)	μ_T	SSF	Scaling Factor
O6.5W1	0.55	150	341	2.3	22.0	77	3.5	1.21	1.93
O6.5W2	0.52	150	462	3.1	32.2	124	3.9	1.21	1.93
O6.5W3	0.52	150	563	3.8	39.8	133	3.4	1.20	1.95
O6.5W2-1	0.57	150	450	3.0	37.8	149	3.9	1.23	1.90
O6.5W3-1	0.54	150	495	3.3	39.0	137	3.5	1.21	1.93
O6.5W3-2	0.53	150	555	3.7	40.0	146	3.7	1.21	1.93
O5W1	0.50	194	428	2.2	21.7	75	3.5	1.20	1.95
O5W2	0.46	194	614	3.2	33.0	116	3.5	1.20	1.95
O5W3	0.47	194	669	3.4	37.8	136	3.6	1.20	1.95
O5W2-1	0.50	194	603	3.1	38.0	140	3.7	1.21	1.93
O5W3-1	0.49	194	583	3.0	36.0	136	3.8	1.21	1.93
O5W3-2	0.48	194	680	3.5	39.6	142	3.6	1.20	1.95
O4W1	0.40	243	510	2.1	21.0	73	3.5	1.20	1.95
O4W2	0.40	243	768	3.2	31.4	114	3.6	1.20	1.95
O4W3	0.44	243	782	3.2	41.0	133	3.2	1.19	1.97
O4W2-1	0.43	243	729	3.0	37.0	143	3.9	1.22	1.92
O4W3-1	0.44	243	753	3.1	38.0	138	3.6	1.20	1.95
O4W3-2	0.45	243	778	3.2	40.0	148	3.7	1.21	1.93
R2S-6.5W1	0.53	90	199	2.2	25.7	79	3.1	1.19	1.97
R2S-6.5W2	0.53	90	307	3.4	37.6	135	3.6	1.21	1.93
R2S-6.5W3	0.56	90	334	3.7	45.6	142	3.1	1.19	1.97
R2S-4W1	0.42	146	314	2.1	23.5	78	3.3	1.19	1.97
R2S-4W2	0.43	146	461	3.1	38.6	130	3.4	1.20	1.95
R2S-4W3	0.42	146	557	3.8	43.0	139	3.2	1.19	1.97
R3S-6.5W2	0.67	142	454	3.2	55.4	167	3.0	1.21	2.04
R3S-6.5W3	0.68	142	540	3.8	67.0	210	3.1	1.21	2.04
R3S-4W2	0.54	231	693	3.0	54.2	170	3.1	1.19	2.08
R3S-4W3	0.56	231	762	3.3	65.0	200	3.1	1.19	2.08

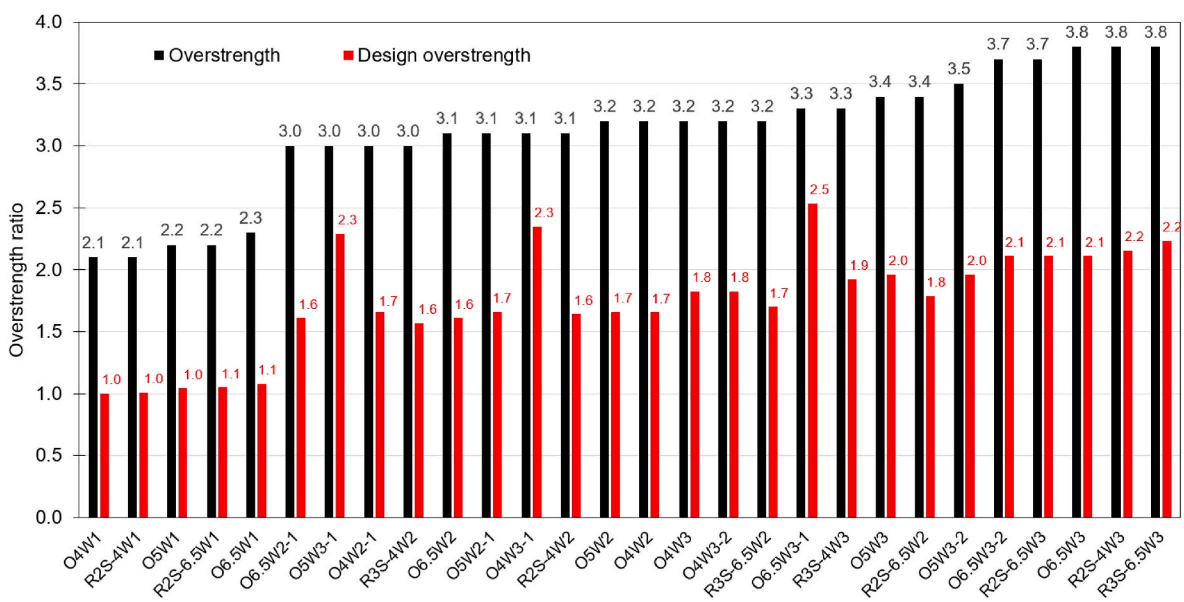

Fig. 9. Overstrength values of archetype buildings.

building was analyzed individually without being part of a performance group due to varying shear wall types and different response modification coefficients. For this reason, only $ACMR_{20\%}$ level scaling was considered for performance evaluation. A similar approach has been adopted by other researchers for seismic performance evaluation of various structural systems [14,30].

The far-field record set of FEMA P695 was used for dynamic analysis of archetype buildings. Far-field record set consists of 22 pairs of ground motions that were recorded at sites located greater than or equal to 10 km distance from the fault rupture. The ground motion pairs utilized in dynamic analyses are listed in Table 5. Response spectra for these 22 pairs of ground motions are shown in Fig. 10(a). Scaling of ground motion records starts by normalizing each ground motion record with respect to peak ground velocities using normalization factors defined in FEMA P695. Following this normalization, the ground motions were scaled up to MCE level, which is represented by $S_{MT} = 1.5$g at the approximate fundamental period T_a. An example MCE level ground motion scaling is given in Fig. 10(b). As evident, the median spectrum of the record set was anchored to MCE response spectra of seismic design category D_{max} at a period of $T_a = 0.175$ s. The MCE scaling factors were further increased by the calculated collapse margin ratios to reach collapse level earthquake with $CMR_{20\%}$. FEMA P695 provides Eq. (12) in order to calculate collapse margin ratios. The 1.2 value in this equation is an amplification factor that is applied to CMR when dynamic analyses are executed with three-dimensional models, as suggested by FEMA P695. SSF is the spectral shape factor, which is a function of period-

Table 5
FEMA P695 far-field ground motion record set.

ID	Earthquake				Normalization Factor
	Magnitude	Name/Year	Component 1	Component 2	
1	6.7	Northridge/1994	NORTHR/MUL009	NORTHR/MUL279	0.65
2	6.7	Northridge/1994	NORTHR/LOS000	NORTHR/LOS270	0.83
3	7.1	Duzce, Turkey/1999	DUZCE/BOL000	DUZCE/BOL090	0.63
4	7.1	Hector Mine/1999	HECTOR/HEC000	HECTOR/HEC090	1.09
5	6.5	Imperial Valley/1979	IMPVALL/H-DLT262	IMPVALL/H-DLT352	1.31
6	6.5	Imperial Valley/1979	IMPVALL/H-E11140	IMPVALL/H-E11230	1.01
7	6.9	Kobe, Japan/1995	KOBE/NIS000	KOBE/NIS090	1.03
8	6.9	Kobe, Japan/1995	KOBE/SHI000	KOBE/SHI090	1.10
9	7.5	Kocaeli, Turkey/1999	KOCAELI/DZC180	KOCAELI/DZC270	0.69
10	7.5	Kocaeli, Turkey/1999	KOCAELI/ARC000	KOCAELI/ARC090	1.36
11	7.3	Landers/1992	LANDERS/YER270	LANDERS/YER360	0.99
12	7.3	Landers/1992	LANDERS/CLW-LN	LANDERS/CLW-TR	1.15
13	6.9	Loma Prieta/1989	LOMAP/CAP000	LOMAP/CAP090	1.09
14	6.9	Loma Prieta/1989	LOMAP/G03000	LOMAP/G03090	0.88
15	7.4	Manjil, Iran/1990	MANJIL/ABBAR–L	MANJIL/ABBA–T	0.79
16	6.5	Superstition Hills/1987	SUPERST/B-ICC000	SUPERST/B-ICC090	0.87
17	6.5	Superstition Hills/1987	SUPERST/B-POE270	SUPERST/B-POE360	1.17
18	7.0	Cape Mendocino/1992	CAPEMEND/RIO270	CAPEMEND/RIO360	0.82
19	7.6	Chi-Chi, Taiwan/1999	CHICHI/HY101E	CHICHI/CHY101-N	0.41
20	7.6	Chi-Chi, Taiwan/1999	CHICHI/TCU045-E	CHICHI/TCU045-N	0.96
21	6.6	San Fernando/1971	SFERN/PEL090	SFERN/PEL180	2.10
22	6.5	Friuli, Italy/1976	FRIULI/A-TMZ000	FRIULI/A-TMZ270	1.44

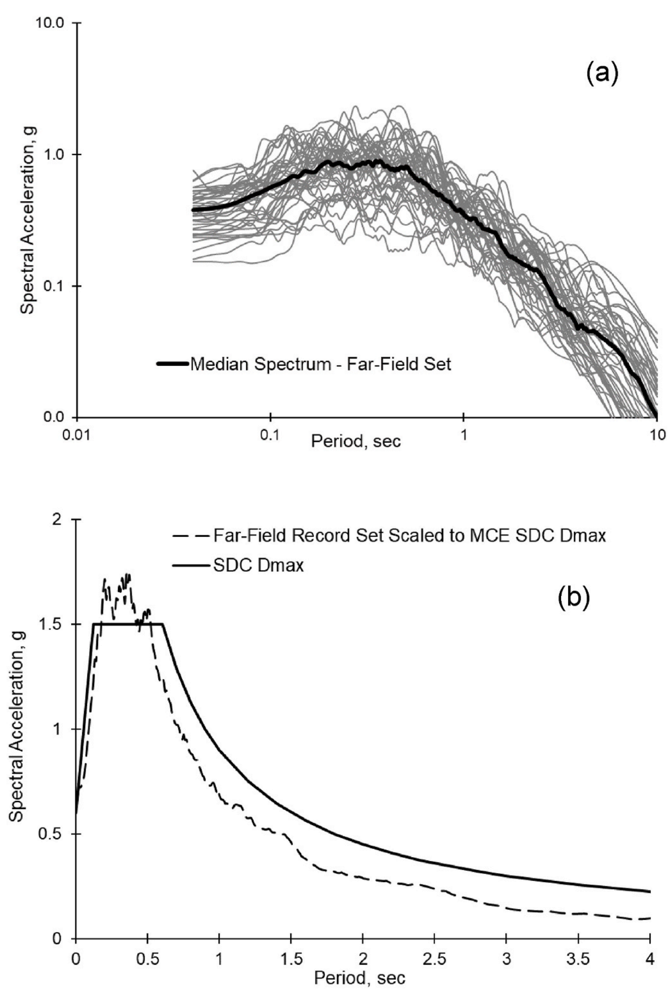

Fig. 10. Response spectra for far-field ground motion record set: (a) response spectra; (b) anchoring far-field record set to MCE spectral demand.

based ductility μ_T, and the fundamental period T_1. ACMR is dependent on total system collapse uncertainty β_{TOT}, which is calculated with Eq. (13).

$$CMR = \frac{ACMR_{20\%}}{1.2 \times SSF} \tag{12}$$

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2} \tag{13}$$

The value of β_{TOT} depends on four uncertainty parameters; record-to-record uncertainty β_{RTR}, uncertainty due to design requirements β_{DR}, test data uncertainty β_{TD} and modeling uncertainty β_{MDL}. In the present study quality rating of (B) Good was adopted, which led to β_{DR}, β_{TD} and β_{MDL} values to be equal to 0.2. The value of β_{RTR} was taken as 0.4, as the period based ductility values for all archetypes were larger than 3. As a result, the total system collapse uncertainty was calculated to be 0.53, which corresponds to $ACMR_{20\%}$ value of 1.56 based on Tables 3–7 of FEMA P695. Since the selection of the total system collapse uncertainty value has an impact on the results, it is important to compare it with values reported in other studies. Previous studies in the literature have considered various β_{TOT} values such as 0.48 [11], 0.49 [13], 0.525 [10], and 0.53 [12,15]. It can be seen that the value of 0.53 adopted in this study closely aligns with those reported in the literature. The SSF values and scaling factors determined for each archetype with the procedure outlined above are given in Table 4. Both SSF values and scaling factors do not change considerably between archetypes. SSF values range between 1.19 and 1.23 and scaling factors range between 1.90 and 2.08.

6. Analysis results

6.1. Lateral drift response of archetypes with different shear wall types

Nonlinear time history analysis results for one office archetype (O6.5W2) and one residential archetype (R3S-6.5W2) are provided in Figs. 11 and 12 as examples. Both of these two archetypes utilized SW2 type shear walls in all floors. The ground motion record pair selected to demonstrate the response of archetype buildings is ground motion #17, which is the record pair from the Superstition Hills 1987 Earthquake (i.e., records SUPERST/B-POE270 and SUPERST/B-POE360 records in Table 5). It should be mentioned that the response spectra for these two records closely follow the median response spectrum of the far-field record set, as shown in Fig. 13. Interstory drift ratio time histories,

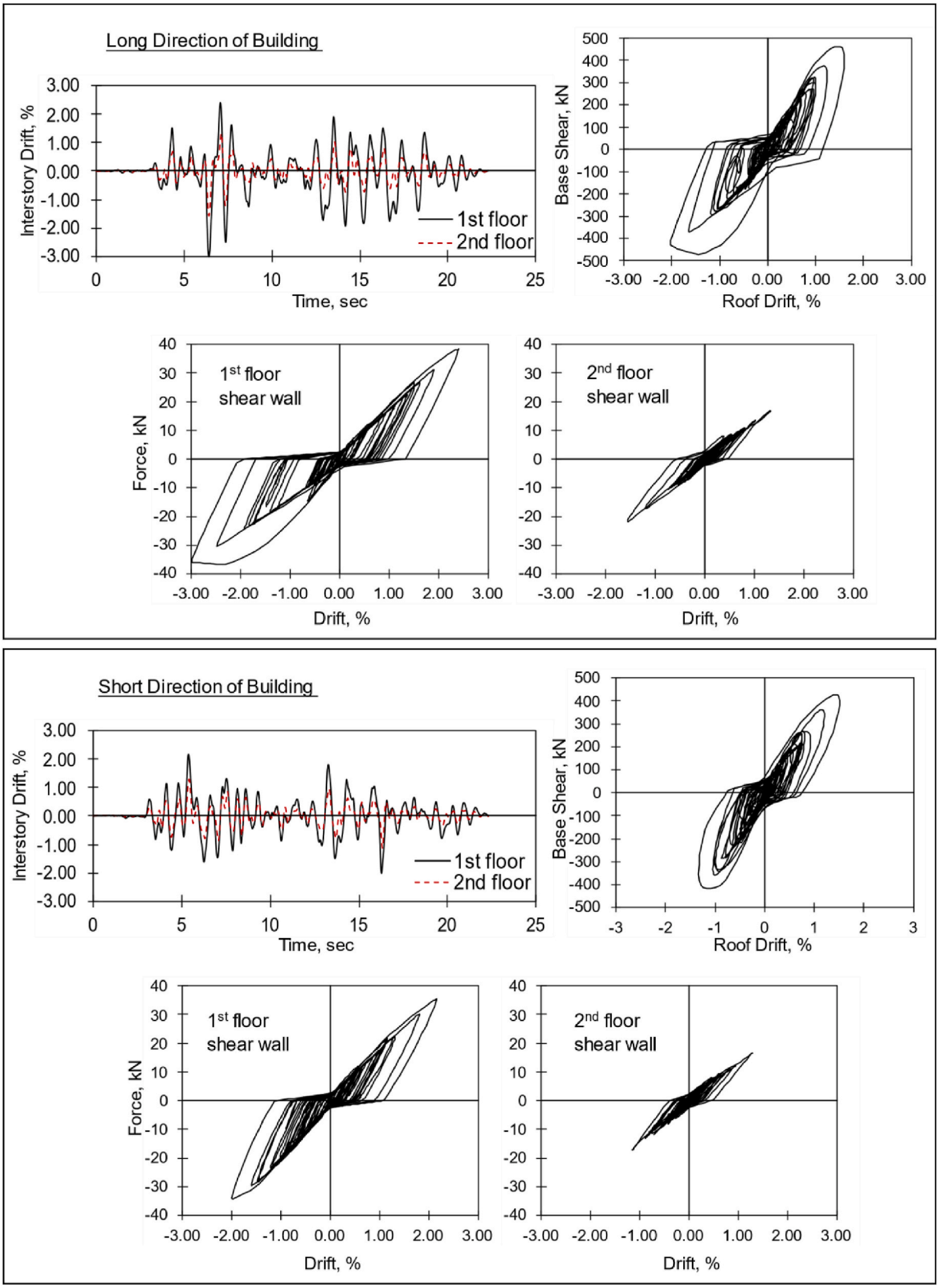

Fig. 11. Nonlinear time history analysis results for archetype O6.5W2.

overall base shear versus roof drift response of buildings, as well as the force versus drift response of individual shear walls in both principal directions are presented in Figs. 11 and 12, respectively for archetypes O6.5W2 and R3S-6.5W2. For both archetypes in both directions, first floor experienced higher seismic demand compared to the other floors. Maximum recorded interstory drift ratios are 3.0% and 2.1% for office and residential archetypes, respectively. The response of the building in the long and short directions varies since ground motion records consist of two horizontal components with different response spectra. It should be mentioned that, even though the building responses shown in Figs. 11

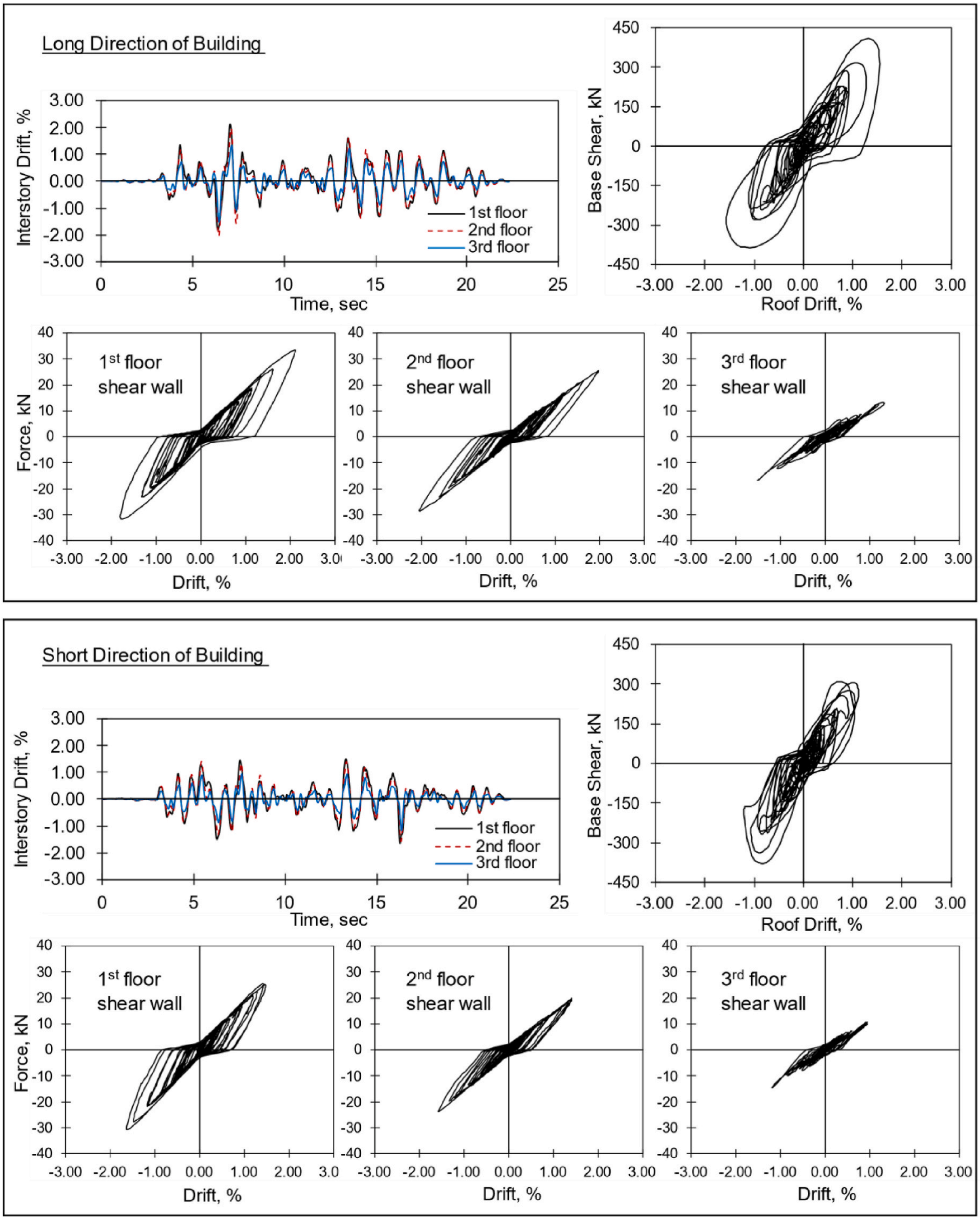

Fig. 12. Nonlinear time history analysis results for archetype R3S-6.5W2.

and 12 represent one horizontal component of ground motion #17 acting in the long direction of archetype and the other component acting on the short direction, same results were obtained when two components of the ground motion changed directions, given that the same number of shear walls were provided for both long and short directions of each archetype as explained before.

The recorded drift profiles of office and residential archetypes at the instant of peak roof displacement are given, respectively in Fig. 14(a) and (b). It is evident that an irregular drift profile occurs with lateral displacement localizing at the first floor when shear wall SW1 is utilized in both floors. In cases where SW2 or SW3 type shear walls are used at first floor and SW1 type shear wall is used at second floor, significantly larger drifts occur at second floor than at first floor. On the other hand, utilizing SW2 or SW3 type shear walls either alone or in combination results in regular distribution of lateral displacements among floors. Displacement profiles for office archetypes utilizing all three types of shear walls and designed with R values of 6.5, 5 and 4 are given in Fig. 15. Irrespective of the R value adopted in design, lateral displacement is localized at first floor with SW1 type shear wall, while a more favorable drift distribution is obtained with shear walls SW2 and SW3.

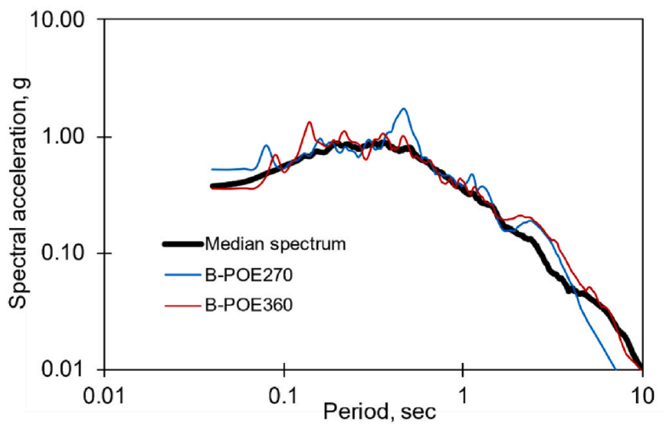

Fig. 13. Comparison of the response spectrum of records from ground motion #17 and the median spectrum of the far-field record set.

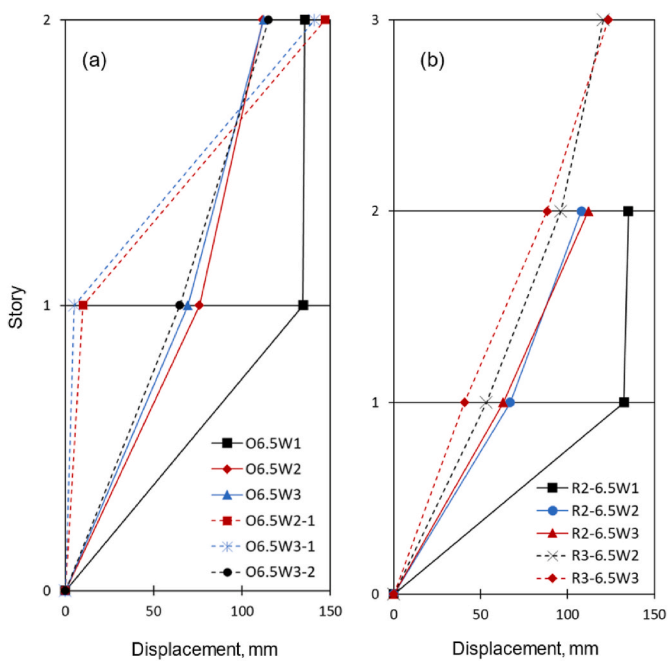

Fig. 14. Displacement profiles for archetypes: (a) office buildings; (b) residential buildings, under ground motion #17.

Detrimental effect of using SW1 type shear wall in second floor of an office archetype is shown in Fig. 16. Significantly large lateral displacements experienced by these walls led to collapse of the second floor, indicated by large residual interstory drifts.

6.2. Performance evaluation of archetypes

Result of each of the 22 time history analyses conducted on each archetype building with ground motions scaled up to ACMR20% level was evaluated by checking whether the analyzed archetype reached the failure condition. It should be noted that, due to the number of shear walls being the same in two principal directions, each ground motion component produces identical building responses in the long and short directions. For that reason, 22 time history analyses were performed for each archetype with each ground motion component acting in one principal horizontal direction. Previous studies in the literature on OSB sheathed CFS framed shear walls reported maximum drift ratios varying between 2 and 4%. A 4% drift ratio was achieved by individual shear walls utilizing 150 mm screw spacing in the study of Liu et al. [24]. Li [18] reported the drift level of 3% for shear walls with 50 mm of screw spacing. Test results provided in Fig. 3 also show that shear walls with single and double side sheathing and 50 mm screw spacing sustained drift values close to 4%. The same maximum drift ratio was also obtained in a previous study with double side sheathed wall panels [25]. Moreover, the 4% interstory drift ratio limit was also utilized in previous numerical studies for OSB sheathed CFS framed shear walls [10,11,14]. In order to be consistent with the current literature, the collapse criterion was adopted as the interstory drift ratio of 4% in this study. Tables 6 and 7 summarize the results of nonlinear dynamic analyses, respectively for office and residential archetypes utilizing different shear wall types and designed with different response modification coefficients. The tables present the number of ground motion pairs that caused collapse for each archetype building. According to the methodology, the archetype fails to satisfy the target performance level when at least half of the ground motions cause collapse. Therefore, the archetypes were considered to fail when the 4% interstory drift ratio limit is exceeded for at least 11 pairs of ground motions.

As evident in Tables 6 and 7, all archetype designs utilizing SW1 type shear walls failed to satisfy the performance criterion, irrespective of the response modification coefficient used at the design stage. This result is valid for both office and residential occupancies. As discussed earlier, these archetypes possess smaller overstrength than the rest of the buildings due to their design being governed by the lateral strength criterion as opposed to drift criterion. It is noteworthy that with the use of SW1 type shear walls as the lateral load resisting system, utilizing even a relatively small response modification coefficient of $R = 4$ results in inadequate design. Based on this observation, it can be stated that caution should be taken when using CFS shear walls sheathed with OSB panels on one side with 150 mm spaced fasteners as the sole lateral load resisting members in building systems. The unfavorable response of the SW1 type shear wall results from its relatively low lateral load capacity compared to its lateral stiffness in relation to the SW2 and SW3 type walls. Utilizing SW1 type shear walls also resulted in overstrength factors smaller than the code recommended value 3, which may prevent these buildings to achieve sufficient performance levels even with relatively small R values. The analysis results indicate that utilizing shear walls with smaller load capacity and stiffness at the second floor is generally an acceptable method. However, the use of SW1 type shear walls in combination with the other wall types adversely affected the building performance and the required collapse performance level was not satisfied for some of the investigated archetypes.

The results presented in Tables 6 and 7 indicate that for buildings utilizing SW2 or SW3 type shear walls, response modification coefficients of $R = 5$ and 6.5 seem to be appropriate with a deflection amplification factor of $C_d = 4$, respectively for the office and residential archetypes. However, as shown in Table 6, the archetype O6.5W2 experienced collapse under 12 ground motion records out of the complete set of 22 motions. The response of this office building under ground motion record pair #7 is further investigated in Fig. 17 in comparison with the companion residential building (R2S-6.5W2). For the ground motion component acting in the long direction, neither buildings experienced failure, and responses are quite similar. In the short direction analysis, up to the maximum interstory drift values close to approximately 3.5%, the two buildings exhibited very similar responses with the office building experiencing slightly larger lateral displacements than the residential building. Such a slight difference in the lateral displacements led to the interstory drifts remaining below the 4% limit in the residential building while the office building passed beyond this limit. As evident in Fig. 17(b), reaching the 4% interstory drift value resulted in shear wall collapse in the office building and from that point forward the building experienced increasing drifts as the lateral load resisting mechanism was basically lost. Because the office archetype O6.5W2 experienced very similar response as the companion residential archetype R2S-6.5W2 with a slight difference in story drifts, O6.5W2 can also be considered as a “near pass”. In that case, all office and

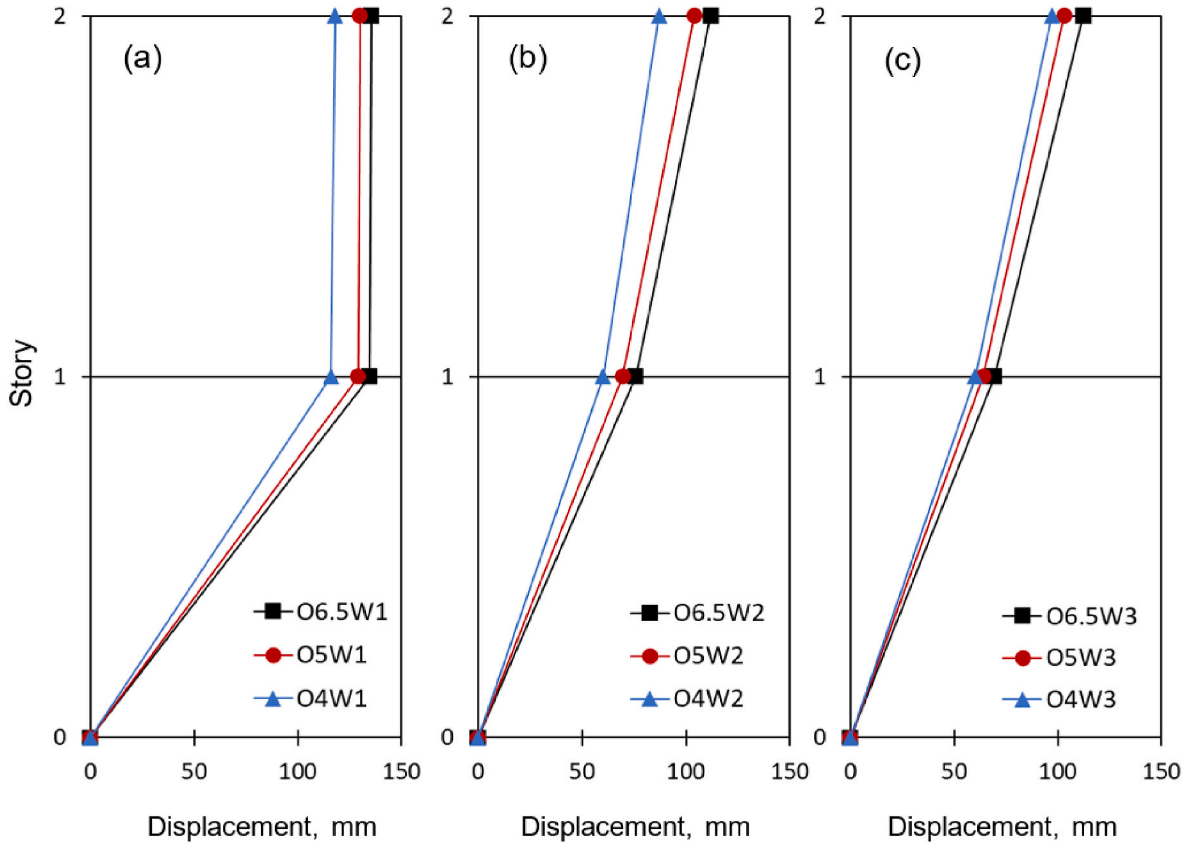

Fig. 15. Displacement profiles for office archetypes under ground motion #17: (a) archetypes with shear wall SW1; (b) archetypes with shear wall SW2; (c) archetypes with shear wall SW3.

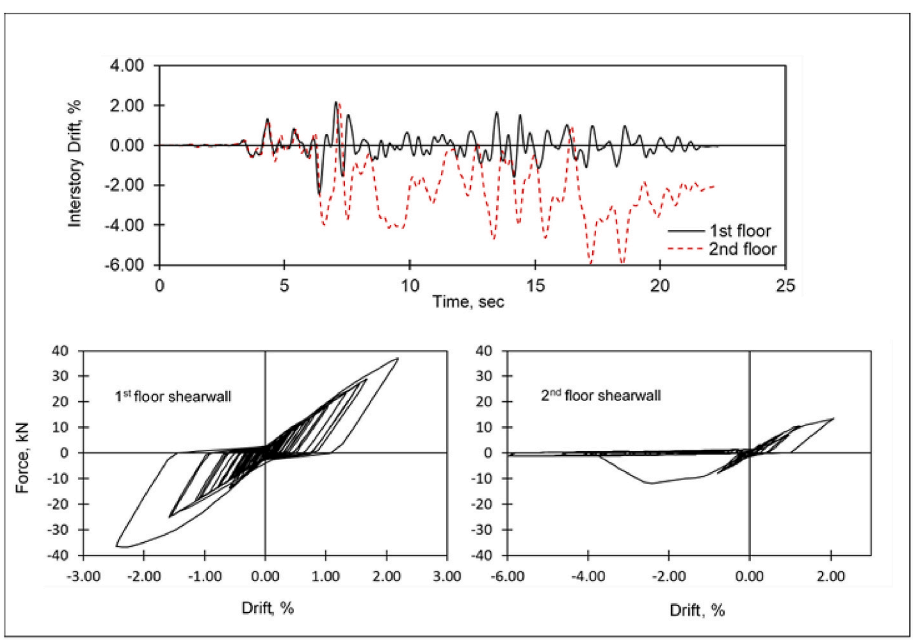

Fig. 16. Nonlinear time history analysis results for archetype O6.5W2-1.

residential archetypes designed with a response modification coefficient of $R = 6.5$ by utilizing SW2 and SW3 types of shear walls satisfy the FEMA P695 acceptability criteria.

It should be noted that the consideration of the deflection amplification factor C_d and the inelastic drift limit for buildings affected the results obtained in the present study. For archetype buildings where the number of wall panels is governed by drift limitation instead of strength requirement, overstrength values range between 3 and 3.75 and the average value is 3.3. This value is in good accordance with the code specified system overstrength value of $\Omega_o = 3.0$.

Table 6
Nonlinear dynamic analysis results for office archetypes.

Office Structure	SW1	SW2	SW3	SW2&1	SW3&1	SW3&2
***R* = 6.5** ***C_d* = 4**	**O6.5W1** **18/22 - FAIL**	**O6.5W2** **12/22 - FAIL**	**O6.5W3** 5/22 - PASS	**O6.5W2-1** **13/22 - FAIL**	**O6.5W3-1** **15/22 - FAIL**	**O6.5W3-2** 5/22 - PASS
***R* = 5** ***C_d* = 4**	**O5W1** **14/22 - FAIL**	**O5W2** 5/22 - PASS	**O5W3** 3/22 - PASS	**O5W2-1** 11/22 - PASS	**O5W3-1** **12/22 - FAIL**	**O5W3-2** 3/22 - PASS
***R* = 4** ***C_d* = 4**	**O4W1** **13/22 - FAIL**	**O4W2** 2/22 - PASS	**O4W3** 2/22 - PASS	**O4W2-1** 4/22 - PASS	**O4W3-1** 8/22 - PASS	**O4W3-2** 2/22 - PASS

Table 7
Nonlinear dynamic analysis results for residential archetypes.

Residential Structure	2 - Story			3 - Story	
	SW1	SW2	SW3	SW2	SW3
***R* = 6.5** ***C_d* = 4**	**R2S-6.5W1** **19/22 - FAIL**	**R2S-6.5W2** 6/22 - PASS	**R2S-6.5W3** 4/22 - PASS	**R3S-6.5W2** 4/22 - PASS	**R3S-6.5W3** 3/22 - PASS
***R* = 4** ***C_d* = 4**	**R2S-4W1** **12/22 FAIL**	**R2S-4W2** 2/22 - PASS	**R2S-4W3** 1/22 - PASS	**R3S-4W2** 1/22 - PASS	**R3S-4W3** 4/22 - PASS

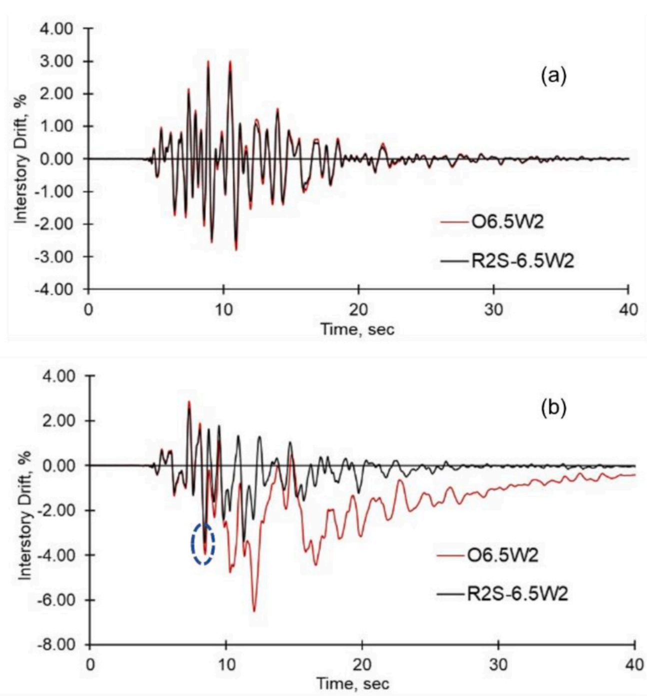

Fig. 17. Nonlinear time history analysis results for archetypes O6.5W2 and R2S-6.5W2 under ground motion #7: (a) long direction; (b) short direction.

Using a relatively large *R* value, such as 6.5 as used in North America, is expected to result in a drift governed design, which eventually leads to higher overstrength. In a study by Shakeel et al. [11], where OSB sheathed wall panels are used as lateral load resisting mechanism for CFS framed buildings similar to the present study, the reported overstrength values for archetypes that were designed with a behavior factor 2.5 range between 1.06 and 1.33. The discrepancy in the overstrength values from the current study and those reported by Shakeel et al. [11] is due to the adaptation of drift requirement in the current study. The number of shear walls in some of the archetypes had to be increased due to the imposed drift requirement, which in turn led to higher overstrength levels. The *R* factor of 6.5 together with an overstrength of 3 indicates a ductility based response modification factor of 6.5/3 = 2.16. In the study by Shakeel et al. [11] the corresponding ductility based response modification factor is 2.5/1.2 = 2.08. These results indicate that this type of structural system possess a ductility based response modification factor of approximately 2.0.

7. Conclusions

A numerical study on the seismic response of CFS structures with OSB sheathed shear walls was conducted. Twenty-eight archetype buildings were designed with various shear wall types and response modification coefficients. The archetypes consisted of office and residential type buildings and were designed by utilizing three shear wall types and three response modification coefficients. Each shear wall represented a different strength level; single side OSB sheathed shear wall SW1 with 150 mm screw spacing, single side OSB sheathed shear wall SW2 with 50 mm screw spacing and double side OSB sheathed shear wall SW3 with 50 mm screw spacing. Response modification coefficients of *R*=6.5, 5 and 4 were adopted in design along with the deflection amplification factor of $C_d = 4$. The following conclusions can be drawn for archetype buildings designed according to the North American specifications:

- Other than archetypes in which shear wall type SW1 was used, interstory drift limit governs the structural design. In these cases, the number of shear walls needs to be increased beyond what is required for strength requirement.
- For archetype buildings with the number of wall panels is governed by drift limitation instead of strength requirement the overstrength values ranged between 3 and 3.8 with an average value of 3.3.
- Buildings designed with shear wall SW1, which represents a wall with single-sided OSB sheathing with 150 mm screw spacing at panel edges, fail to satisfy the performance criteria regardless of the response modification coefficient used for design.
- For buildings with shear walls SW2 (i.e., wall with single-sided OSB sheathing with 50 mm screw spacing) and SW3 (i.e., wall with double-sided OSB sheathing with 50 mm screw spacing) a response modification coefficient of *R*=6.5 is appropriate for the studied office and residential buildings. This justifies the response modification coefficient specified in the American specifications, and indicates that those specified in the Canadian and Turkish specifications are conservative.
- Irregular drift profile occurs with lateral displacement localizing at the first floor when shear wall SW1 is utilized in both floors. In cases where SW2 or SW3 type shear walls are used at first floor and SW1 type shear wall is used at second floor,

significantly larger drifts occur at second floor than at first floor. On the other hand, utilizing SW2 or SW3 type shear walls either alone or in combination results in regular distribution of lateral displacements among floors.

Based on the above conclusions it is recommended to design CFS buildings with SW2 (single-sided OSB sheathing with 50 mm screw spacing) or SW3 (double-sided OSB sheathing with 50 mm screw spacing) type OSB sheathed shear walls with a response modification coefficient of R=6.5 along with the deflection amplification factor of C_d = 4. On the other hand, buildings designed with shear wall type SW1, featuring single-sided OSB sheathing with 150 mm screw spacing at panel edges, fail to meet the performance criteria regardless of the response modification coefficient used. Therefore, it is recommended to employ a more strict interstory drift limitation for buildings utilizing SW1 type shear walls.

Author statement

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Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

Data will be made available on request.

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