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Research Article

Canadian seismic design coefficients for coupled composite plate shear wall/concrete filled (CC-PSW/CF)

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Abstract

Concrete-filled coupled composite plate shear walls (also known as as SpeedCore walls) are gaining acceptance for construction in seismic region throughout North America. Design provisions for this lateral load-resisting system have already been added to ASCE 7-22 and to the American Institute of Steel Construction Seismic Design Provisions. Adequacy of the seismic design parameters used for this structural purpose has been validated in the USA using the FEMA P695 methodology. An interest was expressed by the practicing engineering community to use these walls in Canada, which requires demonstration of satisfactory seismic performance within the Canadian context. As such, new analyses are needed using Canadian-specific sets of ground motions to confirm the adequacy of the seismic design parameters proposed for implementation of these composite walls in the National Building Code of Canada. This paper presents the results of these analyses, showing that the proposed seismic performance factors are appropriate for this structural system in Canada.

Key words: coupled composite plate shear walls, composite plate shear walls, seismic performance factors, FEMA P695 methodology, subduction earthquakes

1. Introduction

Composite plate shear walls-concrete filled (C-PSW/CF) are an efficient structural force resisting system (SFRS) that consist of two steel plates connected together with tie bars having concrete infill. It is highly ductile and stable as the steel plates act as the reinforcement and stay-in-place framework for the infilled concrete while the concrete prevents inward local buckling of the steel plate. These walls can be planar or have various cross-sections (such as C-shaped, T-shaped, and I-shaped) with steel closure plates at the ends or with circular/semi-circular concrete-filled steel tubes as boundary elements as shown in Fig. 1 (Bruneau et al. 2021).

A coupled composite plate shear walls-concrete filled (CC-PSW/CF) is created when two C-PSW/CF are connected at floor levels by coupling beams that consist of concrete-filled steel box sections, as shown in Fig. 2 (Kizilarslan et al. 2021*a*). The 850 ft (259 m) tall 58-story Rainier Square Tower in Seattle is the first building to have used CC-PSW/CF. A notable advantage of CC-PSW/CF for that project is that it reduced the construction period by eight-month compared to a typical reinforced concrete core (AISC 2021). The structural system is also gaining acceptance for mid-rise construction, with completion of the 200 Park building in San Jose (AISC 2021). At the time of this writing, other projects are in the planning stage (AISC 2021).

Design provisions for CC-PSW/CF were added to ASCE 7-22 and to the American Institute of Steel Construction (AISC) Seismic Design Provisions (AISC-341) for the following seismic design parameters: a response modification factor (R) of 8, an over-strength factor (Ω_0) of 2.5, and a deflection amplification factor (C_d) of 5.5 (Kizilarslan et al. 2021*a*). These parameters predict the inelastic response of SFRSs. Using these seismic design parameters, design can be done accounting for the ductile response of structural systems, without the need for inelastic nonlinear inelastic analysis (ASCE 2022).

An interest was expressed by the practicing engineering community to use C-PSW/CF and CC-PSW/CF in Canada. In parallel, as this is becoming a mainstream structural system, there is an interest in implementing this structural system in the Canadian Standard Associations (CSA) standard for Design of Steel Structures (CSA-S16) and the National Building Code of Canada (NBCC). However, at this juncture, adoption of new structural systems in the NBCC requires demonstration of satisfactory seismic performance within the Canadian context. At the time of this writing, a Canadian methodology similar to the FEMA P695 (FEMA 2009) methodology is being developed by the National Research Council of Canada (NRC) for the benefit of NBCC (Fazileh et al. 2023), but it has not been finalized and as it is still being verified and validated for the SFRSs currently defined in NBCC. Simplified equivalent procedures are also being investigated to help streamline the process (Fazileh et al. 2023). A major difference between the FEMA P695 and NRC methodologies lies in the different seismic scenarios being considered for Canada, particularly

Fig. 1. Cross-sections of composite plate shear walls-concrete filled with tie bars having planar rectangle wall with (*a*) flange plates, (*b*) semi-circular boundary elements, (*c*) circular boundary elements, (*d*) C-shaped walls with flange plates, and (*e*) I-shaped walls with flanged plates.

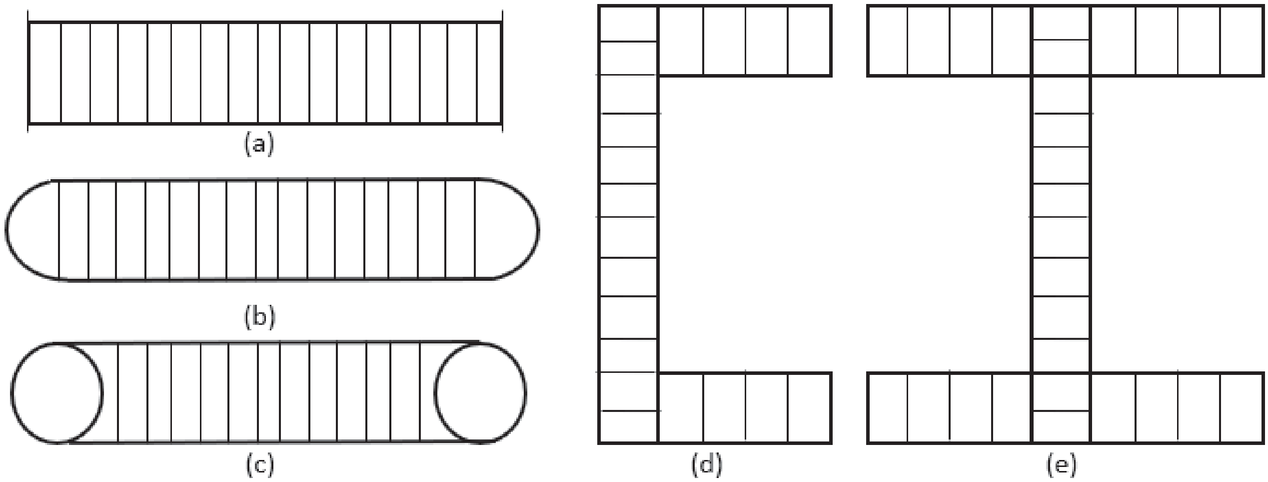
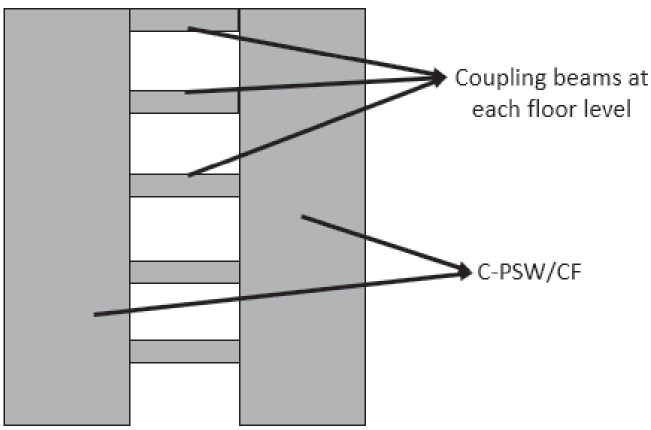


Fig. 2. Configuration of coupled composite plate shear walls-concrete filled (CC-PSW/CF).



due to the fact that, contrary to the FEMA P695 approach used in the USA, a series of subduction zone earthquakes are considered by the Canadian methodology. As such, the seismic design parameters needed for the implementation of the structural system in Canada cannot be taken directly as equal to those in ASCE 7 and new analyses were needed using Canadian-specific sets of ground motions to confirm the adequacy of the seismic design parameters proposed for implementation of these composite walls in the NBCC. This paper presents the results of these analyses conducted to validate the proposed seismic performance factors for CC-PSW/CF in Canada.

2. FEMA P695 methodology

FEMA P695 is a systematic methodology that has been developed to rigorously quantify seismic performance factors for different structural systems. It involves the development of detailed system design information with the probabilistic assessment of collapse risk using incremental dynamic analysis (IDA) and then comparing their response with maximum considered earthquake (MCE). The key steps of this procedure to validate a given structural system include: (1) development of detailed design requirements for the structural system under consideration; (2) collecting information and results from various tests substantiating these design requirements; (3) characterizing the system behavior by designing a number of archetypes; (4) developing computational models that capture the physical behaviors observed experimentally; (5) evaluating the seismic performance of the archetypes using IDA; and (6) computing the margin against collapse provided by the design procedure and comparing this margin against a set acceptance criteria. This methodology was followed by Kizilarslan et al. (2021*a*) to validate the seismic performance factors for CC-PSW/CF that were subsequently implemented in ASCE 7 for seismic design in the USA, namely the values of $R = 8$, $\Omega_0 = 2.5$, and $C_d = 5.5$ mentioned previously.

Procedurally, the proposed equivalent Canadian NRC methodology is similar. First, a location in Canada would have to be selected and new archetype models would have to be designed for that location, using proposed Canadian R factors, and then nonlinear static and IDA would be performed using series of Canadian earthquakes. However, in absence of a finalized Canadian methodology, and to expedite the process (because designing a new set of archetypes for a new location is extremely time consuming), it was decided to instead take the same archetype models considered by Kizilarslan et al. (2021*a*), and pick a location in Canada such that the proposed R factor would have resulted in the same archetypes. This valid simplification allowed to focus on establishing the appropriate "conversion" between US and Canadian seismic performance factors and then running the IDA analysis for the series of Canadian earthquakes.

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Table 1. Three-story archetypes (note 1 in. = 25.4 mm).

Case	No. stories	Coupled wall length, in.	Wall thickness, t_{sc}, in.	Plate thickness, t_p, in.	CB length, in.	CB section, in.	Design coupling ratio, %	Performance group
1	3	120	12	1/8	120	12 × 24x 1/2(f), 3/8(w)	63.1	5
2		120	12	3/16	120	12 × 24 3/8(f), 3/8(w)	47.6	5

Table 2. Eight-story archetypes (note 1 in. = 25.4 mm).

Case	No. stories	L/d	C_s	Coupled wall length, in.	Wall thickness, t_{sc}, in.	Plate thickness, t_p, in.	CB length, in.	CB section, in.	Uncoupled wall length, in.	Performance group
PG-1A	8	3	0.076	144	20	9/16	72	20 × 24 × 3/8(f), 3/8(w)	252	1
PG-1C		5		120	24	5/8	120	24 × 24 × 1/2(f), 3/8(w)	240	1

Table 3. Eighteen-story archetypes (note 1 in. = 25.4 mm).

Case	No. stories	L/d	C_s	C wall depth, in. (c-c)	C wall width, in. (c-e)	$t_{sc.f}$, in.	$t_{sc.w}$, in.	$t_{p.bot}$, in.	$t_{p.top}$, in.	CB length, in.	CB section, in.	Performance group
PG-3A	18	3	0.042	360	180	18	14	1/2	5/16	72	18 × 24 × 5/16(f), 3/8(w)	3
PG-3C		5		360	156	26	16	9/16	5/16	120	26 × 24 × 1/2(f), 3/8(w)	3

3. Archetypes

CC-PSW/CF archetypes used in this study consisted of uncoupled planar walls (Type I) for lower rise buildings, coupled C-shaped walls (Type II) for the taller ones. The coupling beams (CB) were composite box cross sections. The maximum considered seismic demand for this system was seismic design category (SDC) D. On the maximum seismic design parameters (D_{max} for which design spectral accelerations are SDS = 1.0 *g* and SD1 = 0.6 *g*) were evaluated per the FEMA P695 procedure. The height of the structure influenced both the period and the wall configuration of the archetypes. Properties of the archetypes selected for the FEMA P695 study are listed in Tables 1–3.

Reinforcing steel (McKenna 2016) and Concrete02 material models in OpenSees library were used for steel and concrete fibers in the cross-sections of planar and C-shaped walls. Note that this steel inelastic hysteretic model used in the fiber analyses was selected because of its ability to track cyclic strain demands and remove fibers when their low-cycle fatigue life is reached, which is the most important way by which strength degradation is achieved in this type of composite wall, as was observed experimentally by Kenarangi et al. (2020) and Kizilarslan (2023), and verified analytically by Kizilarslan et al. (2021*b*). Details can be found in Kizilarslan et al. (2021*a*). For walls, the nonlinear beam–column elements were only assigned to the first floor of the walls and the rest of the floors were modeled using elastic beam–column elements having effective stiffness per AISC341 eq. I2-12, whereas the coupling beams were modeled using only nonlinear beam–column elements. Leaning columns of insignificant flexural stiffness were added to the structural model to capture the P-Δ effects in each given story due to gravity loads that are not located on the CC-PSW/CF system itself (1440 kips (6405 kN)). Tributary loads coming to the C-PSW/CF walls (72 kips (320 kN) per floor for planar walls and 144 kips (640 kN) for C-Shaped walls) were applied to the wall in each floor. Rigid links were assigned between the C-PSW/CF wall center of gravity and the point where the coupling beams frame into the walls, and rigid beams were used to connect the leaning column and C-PSW/CF wall at every floor. No seismic mass was assigned to the leaning column; seismic masses were applied to the C-PSW/CF walls and distributed equally to its left and right joints at every story.

4. Canadian earthquakes

The ground motion records used in the FEMA P695 procedure, similarly to the seismic data for the USA, are dominated by shallow crustal earthquakes and do not include deep subduction earthquakes. As a significant part of the Canadian infrastructure exposed to seismic hazards is located along the pacific coast of British Colombia, which is exposed to the risk of deep subduction earthquakes, it was deemed necessary to validate the seismic design parameters proposed for CC-PSW/CF with series of Canadian earthquakes.

As the procedure used for this research was reversed from that of FEMA P695, the archetypes were designed first which was taken from Kizilarslan et al. (2021*a*) and then the study location for this was sited at Burnaby, British Colombia, Canada. Two suites of 20 ground motion records have been prepared for this site: one suite of 20 records from shallow crustal shown in Table 4 and in-slab earthquakes shown in Table 5 and another suite of 20 records from interface subduction earthquakes shown in Table 6.

The selection of ground motions was performed in accordance with the procedure specified in the NBCC. The resulting comparison with the design spectra shows adequate scatter to prevent bias in the results of the study. Also,

Table 4. Suite of 12 records from shallow crustal earthquakes.

S.N.	NGA-West2 RSN	Event date and name	Mag.	Station name	V_s, 30 (m/s)	R_{rup} (km)	Comp.
1	4866	2007 Chuetsu-oki, Japan	6.8	Kawanishi Izumozaki	338	12	EW
2	960	1994 Northridge	6.69	Canyon Country—W Lost Cany	326	12	270°
3	1082	1994 Northridge	6.69	Sun Valley—Roscoe Blvd	321	10	0°
4	1003	1994 Northridge	6.8	LA - Saturn St	309	27	110°
5	4886	2007 Chuetsu-oki, Japan	6.69	Tamati Yone Izumozaki	338	11	NS
6	313	1981 Corinth, Greece	6.8	Corinth	361	10	T
7	995	1994 Northridge	6.6	LA—Hollywood Stor FF	316	24	360°
8	725	1987 Superstition Hills-02	6.69	Poe Road (temp)	317	11	360°
9	987	1994 Northridge	6.54	LA—Centinela St	322	28	245°
10	1042	1994 Northridge	6.69	N Hollywood—Coldwater Can	326	12	270°
11	3749	1992 Cape Mendocino	7.01	Fortuna Fire Station	355	20	360°
12	953	1994 Northridge	6.69	Beverly Hills—14145 Mulhol	356	17	279°
			6.7		332	16	

Table 5. Suite of eight records from shallow in-slab earthquakes.

S.N.	No.	Event date and name	Mag.	Station name	Site cl.	R_{hyp}	Comp
1	N001001	2001 El Salvador	7.7	San Pedro Nonualco	D	91	0°
2	EHM003	2001 Geiyo, Japan	6.8	Tohyo	D	63	EW
3	1416a	2001 Nisqually	6.8	West Seattle, Fire Station 29	D	76	125°
4	EHM015	2001 Geiyo, Japan	6.8	Nagahama	D	77	EW
5	4355a	2001 El Salvador	7.7	Santiago de Maria	C/D	95	90°
6	HRS014	2001 Geiyo, Japan	6.8	Ohno	D	63	NS
7	Za01003	2001 El Salvador	7.7	Zacatecoluca	C/D	84	360°
8	HRS0190	2001 Geiyo, Japan	6.8	Kure	D	50	EW
			7.1			73	

Table 6. Suite of 20 records from subduction interface earthquakes.

S.N.	Event date and name	Mag.	Station name	Site Cl.	R_{cd} (km)	Comp.
1	2003 Tokachi-oki, Japan	8.3	Monbetsu-W	D	106	EW
2	2003 Tokachi-oki, Japan	8.3	Monbetsu	C	104	EW
3	2003 Tokachi-oki, Japan	8.3	Biratori-W	C	106	NS
4	2011 Tohoku, Japan	9.1	Towadako-E	D	145	NS
5	2011 Tohoku, Japan	9.1	Iwaki-E	D	112	NS
6	2011 Tohoku, Japan	9.1	Hasunuma	D	164	NS
7	2003 Tokachi-oki, Japan	8.3	Biratori	D	104	NS
8	2011 Tohoku, Japan	9.1	Shimodate	C	161	NS
9	2003 Tokachi-oki, Japan	9.1	Kuriyama	D	151	NS
10	2011 Tohoku, Japan	9.1	Towada	D	143	NS
11	2011 Tohoku, Japan	9.1	Nakoso	D	119	EW
12	2011 Tohoku, Japan	9.1	Yaita	D	162	NS
13	2011 Tohoku, Japan	9.1	Onoda	D	125	EW
14	2011 Tohoku, Japan	9.1	Iwanuma	D	115	NS
15	2011 Tohoku, Japan	9.1	Kanegasaki	C	120	NS
16	2011 Tohoku, Japan	9.1	Naruko	D	137	EW
17	2011 Tohoku, Japan	9.1	Hisaki-2	D	145	EW
18	2011 Tohoku, Japan	9.1	Yaita	C	168	NS
19	2011 Tohoku, Japan	9.1	Yohkaichiba	D	156	NS
20	2003 Tokachi-oki, Japan	8.3	Shihoro	D	98	NS
		8.9			132	

Fig. 3. Scaled ground motion for (*a*) 20 shallow crustal and in-slab earthquakes; and (*b*) 20 subduction interface earthquakes.

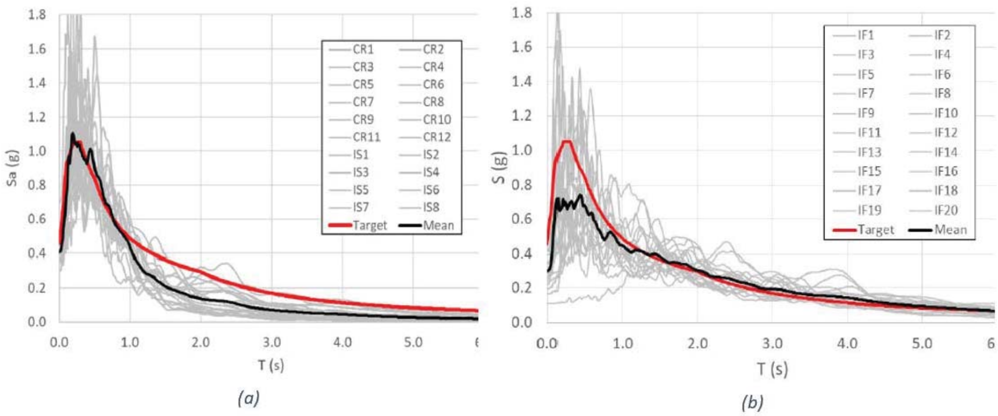
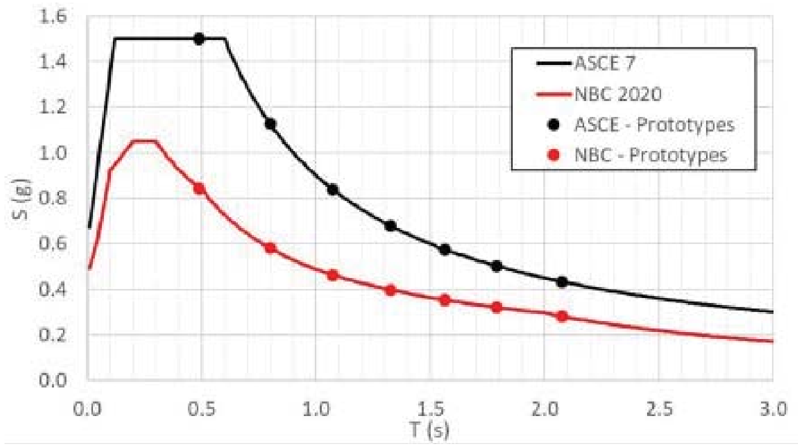


Fig. 4. Comparing and scaling 2%/50 years spectra.



Bebamzadeh (2023) showed that seismic hazard disaggregation for long period structures in Vancouver for a 2% in 50-year probability of exceedance is driven by $M > 8.3$ earthquakes, with a median of M8.8. For the Cascadia subduction zone, only records from the Tohoku and the Tokachi-oki earthquakes meet this criterion with sufficient Arias intensity and duration to be deemed representative for this region.

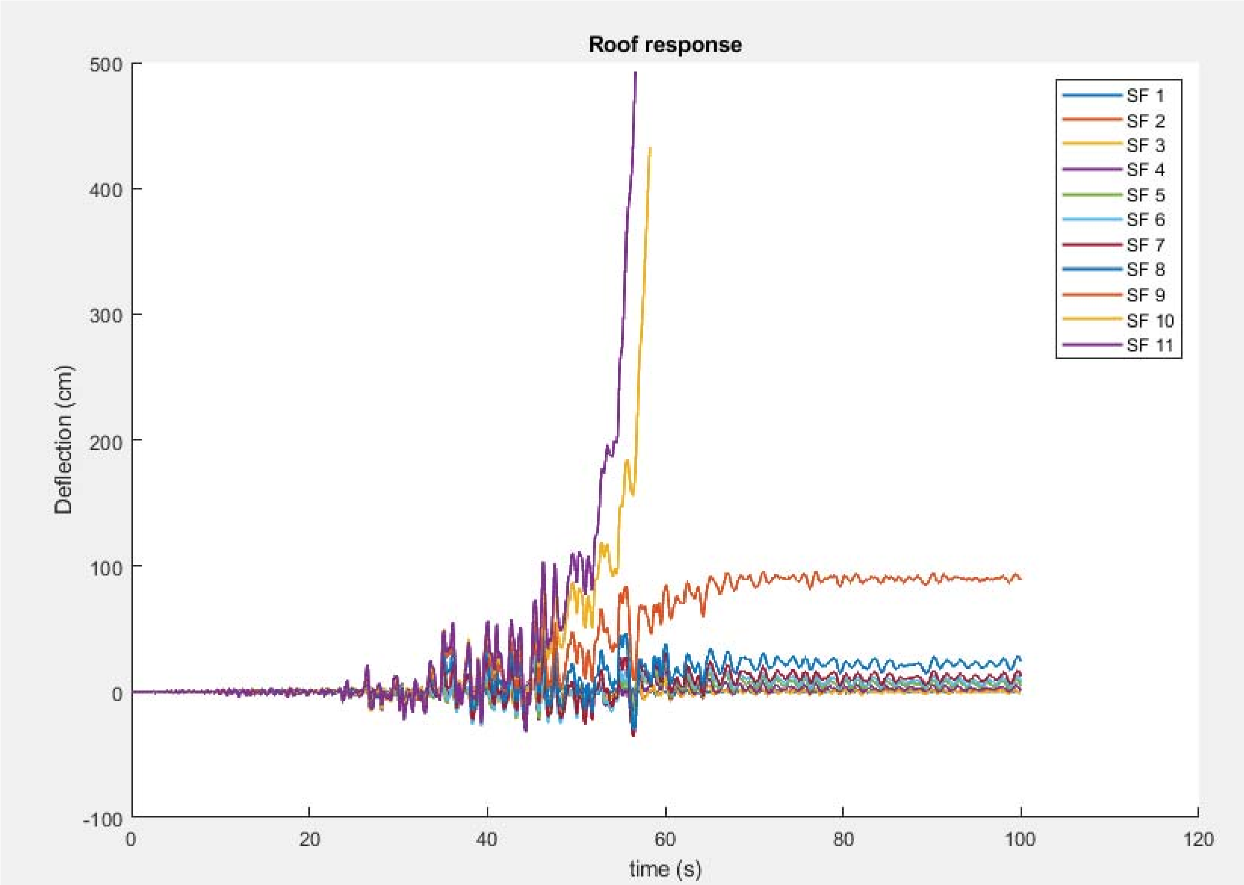
The records were selected and scaled to match the 2020 NBCC 2%/50 years uniform hazard spectrum at the site over the following period ranges: 0.15–1.0 s for the first suite; and 1.0–6.0 s for the second suite, as shown in Fig. 3.

The records were further scaled up such that the imposed seismic demand is consistent with the ASCE-7 MCE hazard level used for the design of the prototypes as shown in Fig. 4. These scaling factors are 1.84 for the GM records from shallow crustal and in-slab earthquakes (i.e., average for the archetype periods over the range 0.15–1.0 s) and 1.65 for the GM records from subduction interface (Cascadia) earthquakes (i.e., average for the archetype periods over the range 1.0–6.0 s) to define this MCE design level. From there, the ASCE factor $R = 8$ was deemed to be equivalent to NBCC factors of $R_\mathrm{d} = 5.0$, $R_\mathrm{o} = 1.6$, as the product of these two parameters is 8.0. This approach is reasonable and expected to lead to no significant differences in results given that the methodology relies on IDA to obtain collapse margin ratio (CMR), and given that the design procedure in a new Annex N concurrently proposed for CSA S16-2024 is similar to the design requirements in its AISC-341-22 counterpart. Also, note that while the objective here is to demonstrate that archetypes with a product of $R_\mathrm{d}R_\mathrm{o}$ as large as 8 would exhibit satisfactory seismic performance, this ensures satisfactory performance if lower values of that product were eventually adopted by the NBCC (making the study presented here more conservative in this regard).

5. Incremental dynamic analysis (IDA) of archetypes

Two different computer models were developed by Kizilarslan et al. (2021*b*) for the analysis of archetypes, namely a distributed plasticity model and a concentrated plasticity model. As the coupling beams in distributed plasticity model were able to explicitly account the effects of cycling loading history and plastic strain accumulation, this model was selected here to perform the IDA using Canadian ground motions. While Kizilarslan et al. (2021*a*) considered 16 different archetype structures to validate the seismic parameters

Fig. 5. Typical response of roof for incremental dynamic analysis showing collapse (SF 1–11 corresponding to "scale factor" levels).



proposed and adopted by ASCE-7, here only six archetypes were selected for the analyses, namely selecting for each of the three-story, eight-story, and eighteen-story archetypes one that Kizilarslan et al. (2021*a*) reported having the lowest CMR, and one that had a higher CMR within the same performance group (PG). The archetype names reflect a number corresponding to the number of stories of the archetypes, followed by the letters "A" or "C" which refer to the length to depth ratio of coupling beams (L/d) being 3 and 5, respectively. The difference in the design of archetypes within the same PG lies in the length of walls and the L/d ratio of the coupling beams. The total length of two walls and coupling beam between them were kept the same within PG but the length of coupling beam was changed to test different coupling ratio of the system.

IDA was performed with these six archetypes by subjecting them to the Canadian ground motions, progressively scaling up of the ground motion until collapse of each structure occurred. The same increments in scaling of ground motions were used as in Kizilarslan et al. (2021*a*). Generally, in IDA, collapse of a structure is considered to have occurred when inter-story drift deformations massively increase for a small increase in the ground motion intensity, eventually to become unbounded. However, as described in Kizilarslan et al. (2021*a*), CC-PSW/CF proved to be structurally stable up to drifts much greater than 5%. Therefore, as excessive drifts are problematic for buildings, the point at which a 5% drift was reached was also defined as corresponding to "collapse" for the purpose of the IDA analyses (except for those few cases where collapse actually occurred a lower drift value where collapse was taken as the last point before unbounded roof response occurred in Fig. 5). A typical response of an archetype for incremental ground motion reaching collapse with increasing scale factor (SF) is shown in Fig. 5.

For all the archetypes, the 90% mass participation ratio occurred in the first four modes of vibrations and the coupled walls started to behave as individual walls with a larger period of vibration after the fracture of nearly all coupling beams (which typically occurred at 5% drift or larger). Thus, to prevent overdamping of the structure after the fracture of the coupling beams, damping properties of the system were reduced by selecting the anchoring periods for the Rayleigh damping coefficients as equal to five times the first period and the fourth period of vibration of the structure (five times the first period being greater than or equal to the period of vibration of the individual walls after fracture of the coupling beams). Note that rigorous application of the FEMA P695 procedure would only require to consider the first and fourth period in this case, so the approach taken here is more conservative as it results in lower damping values (on the order of 1%) at the first period of vibration.

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Fig. 6. Incremental dynamic analysis results for (*a*) PG-ThreeStory, (*b*) PG-ThreeStory2, (*c*) PG-1A, (*d*) PG-1C, (*e*) PG-3A, and (*f*) PG-3C. ACMR, adjusted collapse margin ratio; CMR, collapse margin ratio; SSF, spectral shape factor.

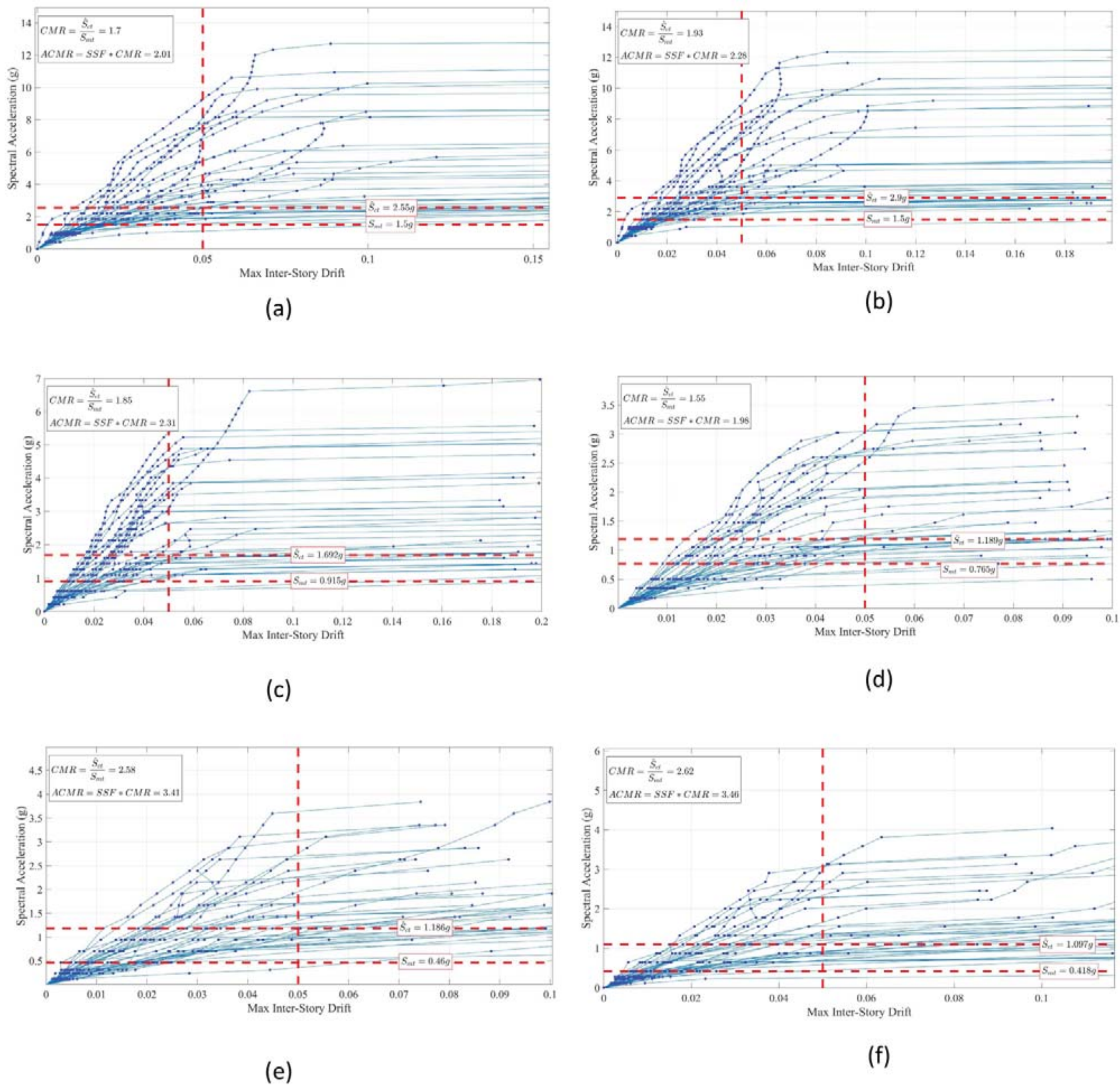

Table 7. Results of incremental dynamic analysis.

Group	Archetype	$\hat{S}$CT (g)	S_{MT} (g)	CMR	SSF	ACMR	Pass/fail	$ACMR_{ave}$	Pass/fail
3 Story	PG-ThreeStory	2.55	1.5	1.70	1.18	2.01	Pass	2.15	Pass
	PG-ThreeStory2	2.9	1.5	1.93	1.18	2.28	Pass		
8 Story	PG-1A	1.692	0.915	1.85	1.25	2.31	Pass	2.15	Pass
	PG-1C	1.189	0.765	1.55	1.28	1.98	Pass		
18 Story	PG-3A	1.186	0.460	2.58	1.32	3.41	Pass	3.44	Pass
	PG-3C	1.097	0.418	2.62	1.32	3.46	Pass		

Note: ACMR, adjusted collapse margin ratio; CMR, collapse margin ratio; SCT, spectral acceleration intensity; SSF, spectral shape factor.

The reduction of the damping ratio ($\xi_{critical}$) depends upon height of the story (H) and was calculated using eq. 1 from Pacific Earthquake Engineering Research Center (PEER 2017) as part of the Tall Buildings Initiative section 4.2.7.

$$\xi_{critical} = \frac{0.36}{\sqrt{H}} \leq 0.05 \quad (1)$$

Table 8. Incremental dynamic analysis (spectral shape factor (SSF) = 1 for subduction earthquakes).

Group	Archetype	$\hat{S}$CT (g)	S_{MT} (g)	CMR	SSF	ACMR	Pass/fail	$ACMR_{ave}$	Pass/fail
3 Story	PG-ThreeStory	2.55	1.5	1.70	(mixed)	2.006	Pass	2.046	Pass
	PG-ThreeStory2	2.9	1.5	1.93	(mixed)	2.086	Pass		
8 Story	PG-1A	1.692	0.915	1.85	(mixed)	2.069	Pass	1.996	Pass
	PG-1C	1.189	0.765	1.55	(mixed)	1.924	Pass		
18 Story	PG-3A	1.186	0.460	2.58	(mixed)	3.41	Pass	3.28	Pass
	PG-3C	1.097	0.418	2.62	(mixed)	3.46	Pass		

Note: ACMR, adjusted collapse margin ratio; CMR, collapse margin ratio; SCT, spectral acceleration intensity.

As per eq. 1, the damping ratios for three, eight, and eighteen stories calculated as 0.05, 0.0336, and 0.0225, respectively.

The results for the IDA performed for the six archetypes and 40 Canadian ground motions considered are presented in Fig. 6. In each case, the median collapse spectral acceleration intensity ($\hat{S}$CT), corresponding to a 50% probability of collapse of the structural system, was taken for each archetype and divided by the response spectrum value for the MCE ground motions at the fundamental period of the archetype, S_{MT}, to obtain the CMR of the archetype, as follows:

$$\text{CMR} = \frac{\widehat{S}_{CT}}{\widehat{S}_{MT}} \quad (2)$$

These CMRs were then multiplied by the spectral shape factor (SSF) recommended by FEMA P695 to obtain adjusted collapse margin ratio (ACMR), where SSF is a function of the fundamental period of vibration (T), the SDC, and period-based ductility (μT) of each archetype. The period-based ductility was conservatively taken as 3 for all archetypes based on observed behavior in experimentally obtained cyclic hysteretic curves even though the nonlinear pushover analysis of archetypes proved that the ductility is more than 3 for all archetypes. Note that the SSF factors provided in FEMA P695 were developed for far-field earthquake records. The same SSFs were used here because the period-based ductility is unchanged here since the same archetypes are used, and also because the Canadian earthquakes, while including subduction zone earthquakes being longer in duration, remain coastal pacific and far field in nature (this could be revisited in future research). Consequently, the acceptable AMCR values (namely, the ACMR10% and ACMR20%) were also taken to be the same as the ones recommended by FEMA P695 and used by Kizilarslan et al. (2021*a*). For the assumed R factor to be deemed satisfactory, per FEMA P695, the ACMR of individual archetype and the average ACMR of a group of archetypes should be greater than the 20% (ACMR20%) and 10% (ACMR10%) collapse probability under MCE, respectively. The values of ACMR20% and ACMR10% are given as 1.56 and 1.96, respectively.

The results of IDA shown above in Table 7 verify that R value of 8 (per ASCE-7) is acceptable, which corresponds to possible Canadian values of $R_d = 5.0$, $R_o = 1.6$.

Note that the obtained CMR values increased for taller buildings, which is consistent with what was observed in Kizilarslan et al. (2021a). Currently, NBCC 2020 does not impose height limits to systems comparable to CC-PSW/CF, such as ductile reinforced concrete walls and reinforced concrete coupled walls in seismic zones, contrary for what is done in ASCE-7. CMRs obtained from Kizilarslan et al. (2021*a*) compare favorably with those obtained for concrete walls studies by Tauberg et al. (2019) and consequently, for consistency, no height limits are recommended for concrete-filled composite plate steel walls in NBCC.

Although not typically required by a FEMA P695 analysis, the IDA results allowed to determine the expected drift of CC-PSW/CF at the design basis earthquake, taken here for expediency as equal to 2/3 of the MCE level (similarly to ASCE-7 practice). Corresponding median values of 0.89%, 1.41%, and 0.75% drift were obtained at the DBE level for the 3-story (PG-ThreeStory), 8-story (PG-1 C), and 18-story archetypes (PG-3 A) archetypes (i.e, those having the lowest CMRs in each group).

Note that, alternatively, it can be logically argued that the SSF for the subduction interface earthquakes should be 1.0. Therefore, to take this into account, an additional comparison was done by using an SSF value of 1.0 for these earthquakes, and the FEMA P695 SSF values for all the others. As such, the ACMR was calculated for each individual earthquake using the appropriate SSF value, and the resulting mean ACMR value is then obtained. Results using this approach are presented in Table 8, and again indicate that the structural system meets the target value (i.e, "pass").

6. Results for different types of earthquakes

The FEMA P695 procedure predominantly focuses on shallow earthquake and the research done to date using that procedure have therefore typically focused on the behavior of structures subjected to those type of ground motions. In validating the proposed seismic parameters for CC-PSW/CF for Canada, determining how the structural system would perform when subjected to deep subduction zone earthquakes was a primary reason for this research. It has been shown above that the ACMR is satisfied globally for the group of 40 earthquakes considered as a whole, and application of the FEMA P695 (or equivalent) procedure effectively ends there. Nothing in the methodologies available to date requires that results be disaggregated to individually present the results for the 20 shallow (in-plate and crustal) earthquakes and the 20 subduction earthquakes. However, even though there is no set of guidelines or requirements on validating the response of SFRS individually for these shallow and subduction

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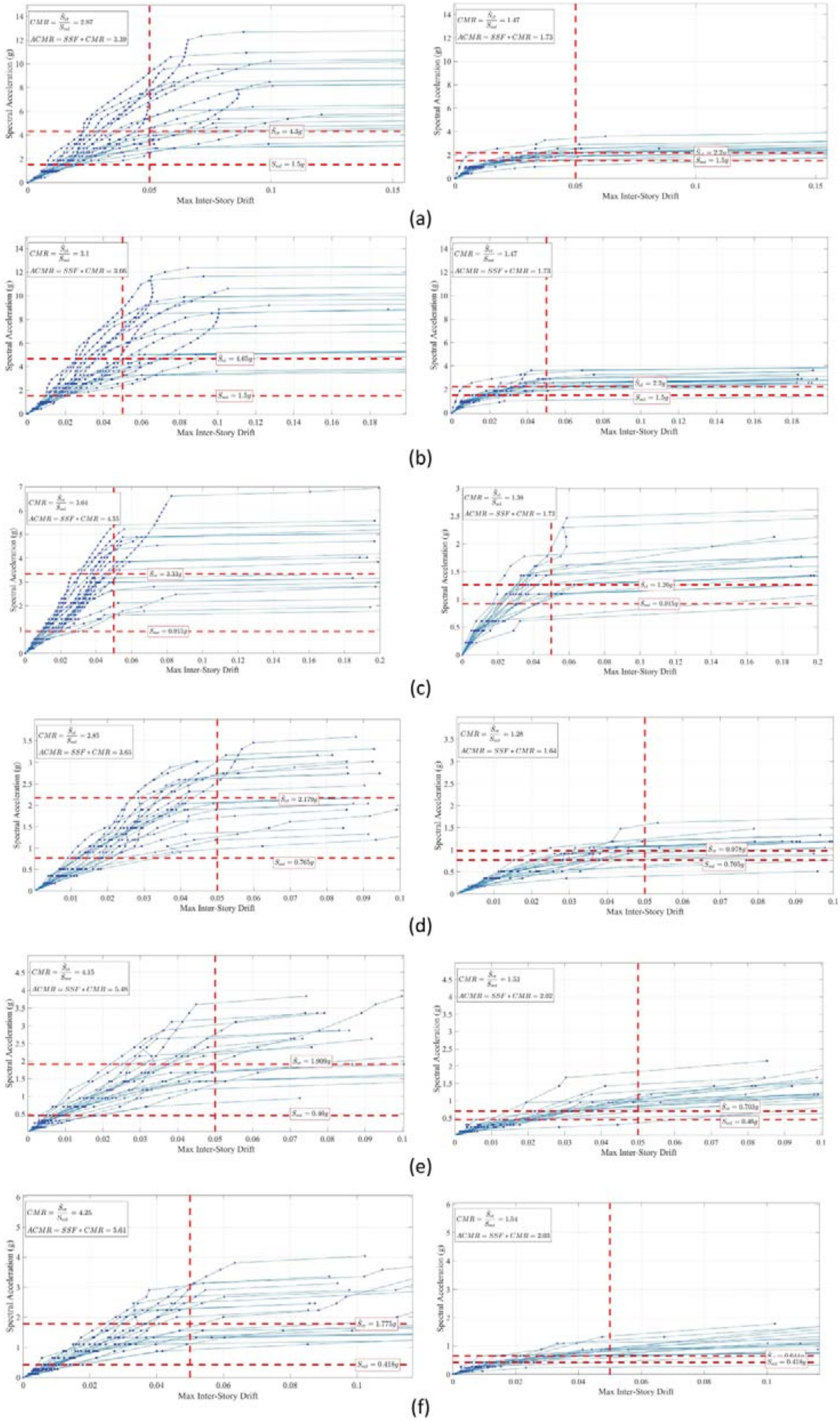
Fig. 7. Incremental dynamic analysis for shallow (left) and subduction (right) zone earthquakes of (*a*) PG-ThreeStory, (*b*) PG-ThreeStory2, (*c*) PG-1 A, (*d*) PG-1 C, (*e*) PG-3 A, and (*f*) PG-3 C. ACMR, adjusted collapse margin ratio; CMR, collapse margin ratio; SSF, spectral shape factor.



zone earthquakes, this separate data are presented here and examined against the FEMA P695 acceptance criteria, as it can provide some insights on the respective contributions of each set of earthquakes and be helpful for future reference.

As shown in Fig. 7, and Table 9 and 10, as expected, CC-PSW/CF behaved well in shallow earthquake. In fact, resulting ACMR were on the same order of magnitudes as those obtained by Kizilarslan et al. (2021*a*). However, for the

Table 9. Incremental dynamic analysis results for shallow earthquakes only.

Group	Archetype	$\hat{S}$CT (g)	S_{MT} (g)	CMR	SSF	ACMR	Pass/fail	$ACMR_{ave}$	Pass/fail
3 Story	PG-ThreeStory	4.3	1.5	2.87	1.18	3.39	Pass	3.52	Pass
	PG-ThreeStory2	4.65	1.5	3.10	1.18	3.66	Pass		
8 Story	PG-1A	3.245	0.915	3.64	1.25	4.55	Pass	4.04	Pass
	PG-1C	2.179	0.765	2.85	1.28	3.65	Pass		
18 Story	PG-3A	1.910	0.460	4.15	1.32	5.48	Pass	5.54	Pass
	PG-3C	1.775	0.418	4.25	1.32	5.61	Pass		

Note: ACMR, adjusted collapse margin ratio; CMR, collapse margin ratio; SCT, spectral acceleration intensity; SSF, spectral shape factor.

Table 10. Incremental dynamic analysis results for subduction earthquakes only.

Group	Archetype	$\hat{S}$CT (g)	S_{MT} (g)	CMR	SSF	ACMR	Pass/fail	$ACMR_{ave}$	Pass/fail
3 Story	PG-ThreeStory	2.2	1.5	1.47	1.18	1.73	Pass	1.73	Fail
	PG-ThreeStory2	2.2	1.5	1.47	1.18	1.73	Pass		
8 Story	PG-1A	1.26	0.915	1.38	1.25	1.73	Pass	1.68	Fail
	PG-1C	0.978	0.765	1.28	1.28	1.64	Pass		
18 Story	PG-3A	0.703	0.460	1.53	1.32	2.02	Pass	2.03	Pass
	PG-3C	0.644	0.418	1.54	1.32	2.03	Pass		

Note: ACMR, adjusted collapse margin ratio; CMR, collapse margin ratio; SCT, spectral acceleration intensity; SSF, spectral shape factor.

Table 11. Incremental dynamic analysis results showing failure conditions.

	Number of earthquakes causing failure of archetype			
	Before 5% drift		After 5% drift	
Archetypes	Shallow	Subduction	Shallow	Subduction
PG-ThreeStory	0	13	20	7
PG-ThreeStory2	0	13	20	7
PG-1A	4	13	16	7
PG-1C	12	18	8	2
PG-3A	17	18	3	2
PG-3C	16	18	4	2

subduction zone earthquakes, while the individual ACMR passed the $ACMR_{20\%}$ criteria, the average ACMR marginally failed to satisfy the $ACMR_{10\%}$ criteria in two of the three cases considered. ACMR values of 1.73 and 1.68 were 11.7% and 14.3% lower than the $ACMR_{10\%}$ of 1.96 prescribed by FEMA P695. This indicates that under subduction type earthquakes, slightly more than 10% of the archetypes would collapse. In these cases, 15.8% and 17% actually collapsed. Evidently, the 10% threshold is a somewhat arbitrary reflection of what is deemed acceptable by the profession. As other studies have also typically found code-compliant structures to have collapse probabilities greatly exceeding 10% when subjected to subduction earthquakes (e.g., Nasser et al. 2019), it is foreseeable that this will be the subject of future discussions. Note that the acceptable probability of collapse under subduction earthquakes has never been established and should be the subject of future deliberations. The numbers here provide a preliminary basis of comparison to fuel such discussions. Finally, additional data showing the failure of archetypes below and above 5% drift is presented in Table 11. Results indicate that failure generally occurred at smaller drifts for subduction zone earthquakes.

Comparing the drift of CC-PSW/CF at the DBE obtained in each case for the separate analyses above, it is found that median drift values were 0.89% and 0.83% for the 3-story (PG-ThreeStory) archetype, 1.12% and 1.64% for the 8-story (PG-1 C) archetype, and 0.51% and 1.05% for 18-story archetypes (PG-3 A) archetypes, for the cases of shallow earthquakes and subduction earthquakes, respectively. This indicates that for CC-PSW/CF (for the limited cases considered), DBE drifts for subduction zone earthquakes could be approximately the same or up to 45% larger in subduction earthquake compared to what is expected in analyses using shallow earthquakes.

7. Effects of damping on disaggregated results

When considering the above disaggregated data, keep in mind that the analysis was conducted using damping values lower than what is rigorously required by the FEMA P695 procedure. Rayleigh damping coefficient were defined by anchoring the damping values from eq. 1 at five times the first period and the fourth period, whereas it is actually prescribed by the procedure to anchor the values at the first and fourth period. For example, considering archetype PG-1 C, which among all the archetypes had the lowest ACMR, considering the extreme anchoring period of five times the first period

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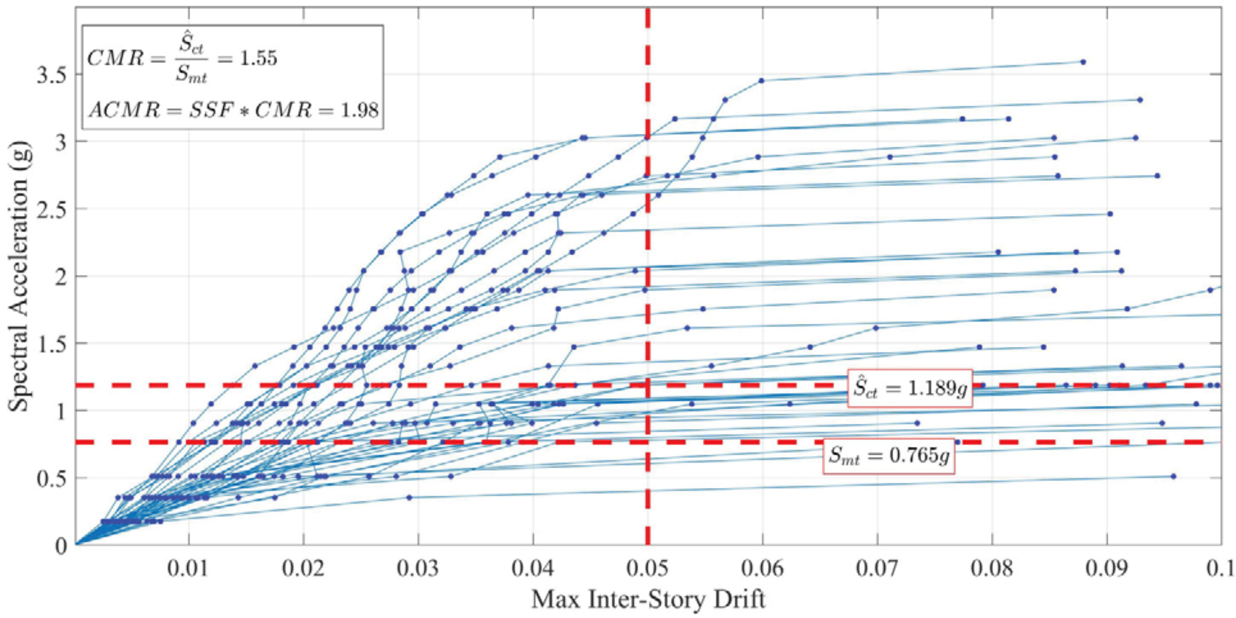
Fig. 8. Incremental dynamic analysis of PG-1 C (with 5% damping anchored at first and fourth period) for subduction earthquakes only. ACMR, adjusted collapse margin ratio; CMR, collapse margin ratio; SSF, spectral shape factor.



results in less than 1% damping (instead of 3.3%) at the first period of vibration for PG-1 C, which contributes to more than 80% of the total modal participation factor (Bruneau et al. 2019).

Thus, for comparison, a complementary set of analyses were performed for this archetype by taking Rayleigh damping of 5% anchored between the first period and the fourth period of vibration. The resulting ŜCT was 1.189, giving a CMR of 1.55 and an ACMR of 1.99 as shown in Fig. 8, which is greater than the required ACMR10% of 1.96 prescribed by FEMA (meaning results in Table 10 now pass, instead of marginally failing, which shows that the reduced damping approach adopted in this study (as described earlier) is a more conservative approach than what is prescribed by the FEMA P695 procedure).

8. Conclusion

This study investigated the adequacy of proposed seismic performance factors of concrete-filled coupled composite plate steel walls, using the FEMA P695 methodology but ground motions deemed to be more appropriate to demonstrate satisfactory seismic performance within the Canadian context. Most significantly, this differed from previous studies in that a series of subduction zone earthquakes were considered for this purpose. As far as results are concerned for possible adoption by the NBCC, it was shown using values of $R_d = 5.0$ and $R_o = 1.6$ would be adequate for design in the Canadian context (and, implicitly, lower values of the product R_dR_o would also be acceptable).

Drift at the design basis earthquake level for the archetypes considered ranged between 0.75% and 1.41% drift. For the set of archetypes considered, drifts calculated for the subduction earthquakes alone compared to the shallow (in-plate and crustal) earthquakes were either approximately the same or up to 45% larger.

For curiosity only (because it is not required by the methodology to certify seismic parameter values), results were "disaggregated" to presents separately results corresponding to only subduction zones earthquakes, and only shallow earthquakes. For the latter case, CMRs were comfortably large and on the same order of magnitudes as those obtained by Kizilarslan et al. (2021*a*). For the subduction zone earthquakes, CMRs were substantially smaller, as expected, but still considered satisfactory. Note that FEMA P695 does not define what should be the acceptable CMR for subduction earthquakes, and this should be the subject of future deliberations. The numbers here provide in this study provide a preliminary basis of comparison to fuel such discussions.

Finally, it is recommended that this new structural system be adopted in NBCC without height limits in seismic zones, for consistency with current practice for ductile reinforced concrete walls and reinforced concrete coupled walls, particularly as the seismic performance of CC-PSW/CF is known to compare favorably with its concrete counterparts.

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Data availability

Data generated or analyzed during this study are provided in full within the published article.

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