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Preliminary Results from FEMA P695 Study on Coupled Composite Plate Shear Walls—Concrete-Filled

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Abstract

This paper presents preliminary findings from a study conducted to determine the R-factor (and other parameters, C_d and Ω_0) for Coupled CPSW/CF using the FEMA P-695 procedure, which is intended to ensure that an adequate margin against collapse exists for the maximum considered earthquake (MCE) hazard. This is established by performing incremental dynamic analysis for the 44 earthquake ground motion records specified by the FEMA P-695 methodology. The findings are intended to substantiate proposed response modification factor R; deflection amplification factor, C_d; and system over-strength factor, Ω_0) for these type of walls in subsequent editions of ASCE 7. A larger R factor is foreseen for coupled composite walls; more specifically, the current R-value of 6.5 specified in ASCE 7-16 for composite walls could potentially increase to a value as high as 8.

1. Introduction

Coupled Composite Plate Shear Walls-Concrete Filled (C-CPSW/CF) are a special lateral-force resisting systems consisting of two steel plates with concrete infill in between them. The steel plates are connected to each other using tie bars that are embedded in the concrete infill and, in some instances, steel-headed stud anchors. ASCE 7-16 specifies the seismic response parameters. However, the ASCE 7-16 specifications do not distinguish between coupled and non-coupled walls.

Nonlinear time history analysis is typically used to predict as accurately as possible the seismic response of a ductile structure. However, it is not typically suited for design. The most common approach in design is to perform (static or dynamic) elastic analysis considering lateral forces and displacement determined on the basis of an elastic design spectrum and modified to account for the ductile response of structures. More specifically, ASCE 7-16 defines three seismic performance factors to approximately predict the elastic-perfectly plastic response of a seismic resisting system; namely the response modification factor, R; deflection amplification factor, C_d; and the system over-strength factor, Ω_0. The research presented here only investigates the value of these factors for a special seismic-force resisting system defined as Coupled Composite Plate

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Shear Walls-Concrete Filled (C-CPSW/CF) by ASCE-7 and AISC-341. This system consists of two Composite Plate Shear Walls-Concrete Filled (CPSW/CF) and coupling beams (concrete-filled steel tubes) spanning between these walls in every floor. (CPSW/CF) consists of two steel plates with concrete infill in between them as in Fig. 1 and coupling beam are concrete-filled built-up box sections or rectangular HSS section. Even though there are specific factors for planar CPSW/CF in seismic regions, the ASCE-7 specifications do not differentiate between coupled and non-coupled walls. Therefore, FEMA P-695 procedure (FEMA 2009) has been used to establish a new R-factor (and other factors) for this lateral load resisting system. Preliminary representative findings from this research project are presented here.

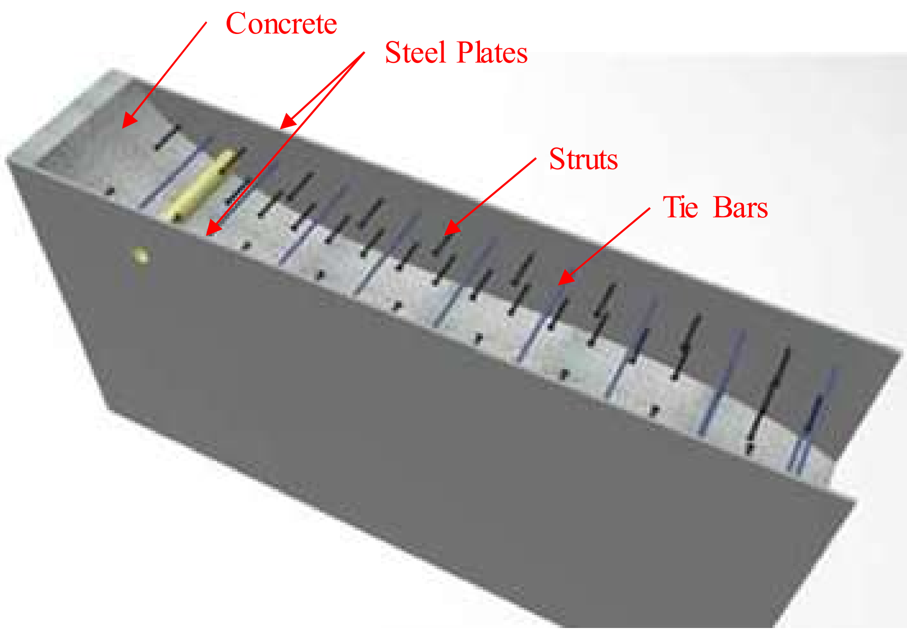

Figure 1: Schematic layout of CPSW/CF (Courtesy of Prof. Amit Varma, Purdue University)

2. Material Models and Element Type

Due to the intense computational requirements required to executes the nonlinear analyses required by the FEMA P695 procedure, OpenSees (McKenna et al. 2016) has been selected as it allows to model nonlinear behavior using macro fiber models with relatively rapid execution time compared to the 2-D or 3-D finite element approaches based on continuum mechanics available in other software platforms. Moreover, OpenSees provides a wide range of constitutive models, elements and solution algorithms, and fast execution time.

The material called Reinforcing Steel Material in OpenSees was selected here to model the steel fibers of the cross-section. This model was developed by Kunnath et al. (2009). The advantage of this particular material model is that it is the only model in the OpenSees material library that allow simulating both buckling and fracture. This material is defined by backbone curve parameters, buckling parameters, fatigue parameters, Menegotto-Pinto curve parameters and hardening constant. More details can be obtained from the OpenSees library.

For the results presented here, the *Concrete02* material model in OpenSees was used for concrete because it accounts for the effect of confinement, degradation of stiffness, tension-stiffening, and concrete crushing. Only seven parameters are required to define the *Concrete02* model in OpenSees. Since the concrete inside composite walls is confined to some degree by the steel plates, the uniaxial stress-strain model for confined concrete in steel tubes developed by Susantha et al. (2001) was used to determine the values for the confined concrete parameters of the *Concrete02* material model in OpenSees.

Based on experimental observations, the shear deformation has not been observed to be significant for these type of walls, and hence, 2D beam element types are selected here to describe the hysteretic behavior. There are two types of nonlinear beam elements in OpenSees, namely force-based and displacement-based elements. Displacement-based elements for plastic hinge region were chosen as they observed to be more stable under earthquake loading. The rest was modeled with elastic beam-column elements.

3. Verification

Before performing IDA, the material models were needed to be verified by test done for the components of this system. This section presents the verification material model and size of element chosen for calibration of the components of this system.

3.1 Planar Composite Plate Shear Walls-Concrete Filled

Five planar CPSW/CF walls, tested in the Bowen laboratory of Purdue University, were used for calibrating the models of planar walls. In this test series, both axial and cyclic loading were applied at the top of the specimens. The backbone curves of the steel material was defined from the results of coupon test. The buckling and fatigue parameters were empirically obtained after many trials and default values for the Menegotto-Pinto curve parameters and hardening constant were used. Parameters of confined concrete at the two ends of cross-section were calculated from the equation for rectangular section from the study of Susantha et al. (2001) whereas parameters of the concrete between the two ends were obtained from strength of unconfined cylinder test.

Displacement-based nonlinear elements were only assigned to the plastic hinge region of the walls and elastic elements were used for the rest of the wall specimens. The plastic hinge region were modeled with three 6 in. elements based on the distance calculated from yield and plastic moment capacity of the specimen's cross-section.

Fig. 2 shows the comparison between the experimental results of CW-42-55-10-T specimen with its OpenSees models and a good correlation was obtained. This specimen had a height of 108 inches, length and width of 36 and 9 inches respectively, 0.1875-inch-thick steel plates, and ties spaces at 9 inches.

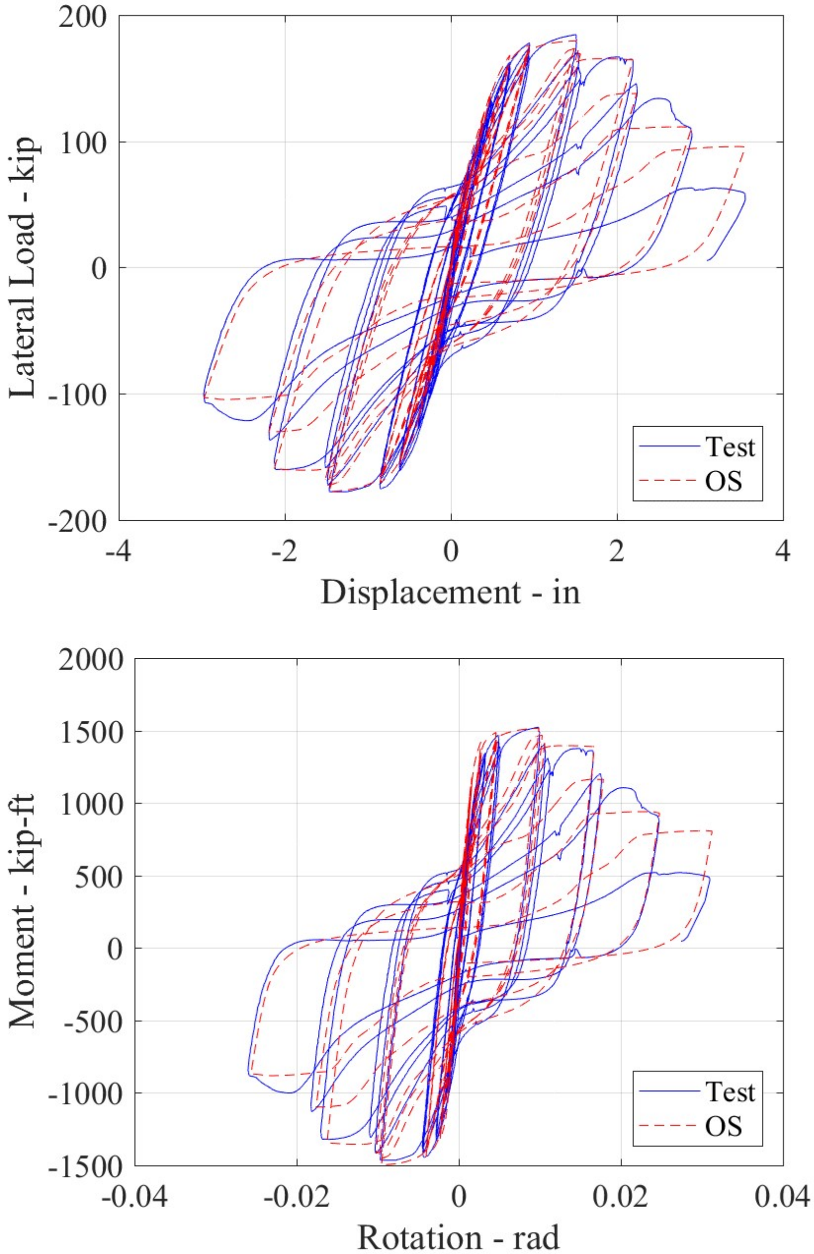

Figure 2. Base shear-displacement (top) and moment-rotation (bottom) comparisons of OpenSees models with test data for CW-42-55-10-T.

3.2 Coupling Beam Tests by Nie et al. (2014)

The calibration of the OpenSees models of the coupling beams was done using experimental results from the study conducted by Nie et al. (2014) on six coupling beam specimens connected to two shear wall piers and subjected to cyclic loading. The coupling beam width and height were 5.91 in. (150 mm.) and 11.81 in. (300 mm.), respectively. The thickness (t) and the length of the specimen (l_b) varied from one specimen to the other. The coupling beams in the archetype of the C-CPSW/CF system considered for the current FEMA P-695 study is purposely designed to fail in a flexure-governed mode. Therefore, the results from coupling beams that has flexuredominant yielding were considered more representative and used here to calibrate the coupling beam models in OpenSees.

The backbone parameters are taken from coupon test reported by Nie et al (2014). Buckling and fatigue parameters were empirically chosen until good correlation for the observed hysteretic behavior of the tested specimens was obtained. Displacement-based nonlinear elements were also assigned to the coupling beam specimens. In addition, the wall piers to which the coupling beams were attached had some flexibility that resulted in some additional rotations at the specimens ends as they were cycled. To account for that effect, two rotational spring, whose stiffness was determined by trial-and-error, were added to the ends of the beams. Then, the shear force (in kN) versus chord rotation hysteretic relationship for the coupling beam specimens were compared with OpenSees model of corresponding coupling beam, as shown in Fig. 3. The chord rotation was calculated by dividing the relative displacements between two ends of coupling beam of the specimens by the length of the coupling beam part of the specimen. Fig. 3 shows that the test results agreed with results from the OpenSees model within acceptable tolerance for both coupling beams.

4. Archetype Generation

FEMA P695 requires example structures of the system under consideration, called archetypes, in order to perform incremental dynamic analysis (IDA). Therefore, the design space chosen for this study is the core walls of low to mid-rise (8-22 story) commercial buildings. The core walls consist of two concrete-filled composite plate shear walls (CPSW/CF) coupled with concrete filled tubes. This design space was then broken down into performance groups based on basic configuration, design load level, and structure period. Two structural configurations, under two seismic load levels were evaluated. These parameters correspond to four performance groups (PG) with up to 16 archetypes designed and analyzed. These archetypes were designed by Jungil Seo, Morgan Broberg, Soheil Shafei, under the direction of Professor Amit Varma at Purdue University, as part of this Pankow/AISC collaborative project.

Table 1 summarizes properties of selected archetype, for which results are presented in this paper. Although two structural configurations of coupled walls were addressed in this study, namely planar walls and C-shaped walls, only the result for coupled planar walls presented in this paper.

Table 1. Selected 8- and 12-story archetype structures.

Name	No. of Stories	L/d	Seismic Design Category (SDC)	C_s	Coupled Wall Length (in)	Wall Thickness (in)	Plate Thickness (in)	Coupling Beam (CB) Length (in)	CB Section (in)
PG-1A	8	3	D_{max}	0.076	144	20	9/16	72	20x24x 3/8(f), 3/8(w)
PG-1D	12	3	D_{max}	0.057	204	18	9/16	72	18x24x 5/16(f), 3/8(w)

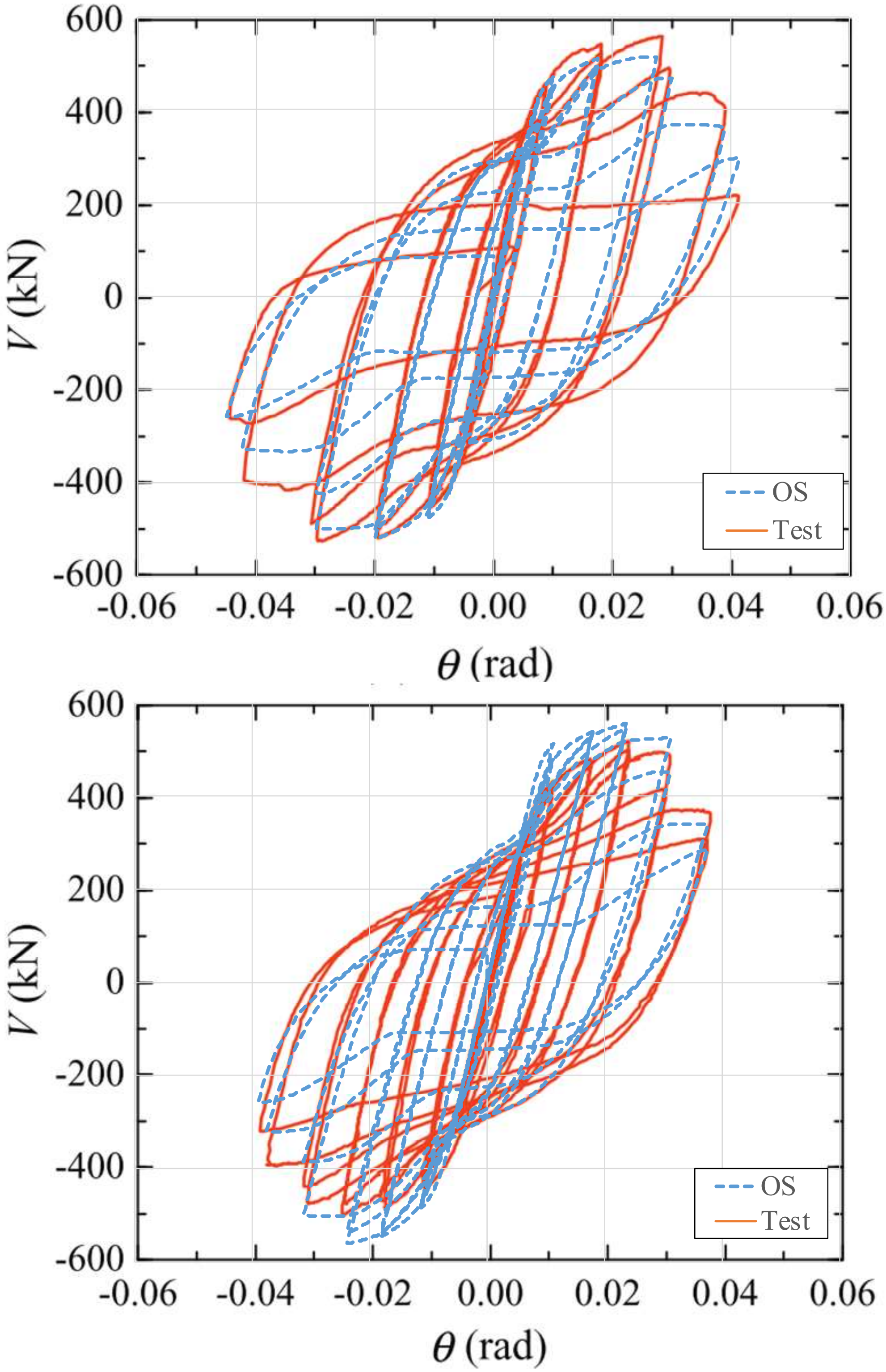

Figure 3. The comparison between OpenSees and Test results for CFSCB-1 (top) and CFSCB-3 (bottom) specimens.

5. Results of Incremental Dynamic Analyses

Incremental dynamic analysis (IDA) consists of a series of successive time history analyses performed for a given structural model, for which the intensity of ground motions specified is gradually scaled up from low to high magnitude until global collapse is observed in the structure. The 44 far-field ground motions from Pacific Earthquake Engineering Research Center (PEER 2017) specified in the FEMA P695 were used for IDA. For the eight- and twelve-story archetypes considered here, each ground motion was gradually scaled up in steps equal to four-tenth and six-tenth of the MCE level (i.e., 0.4Sa-MCE and 0.6Sa-MCE), respectively, up to the intensity that caused structural collapse for each individual ground motion.

Fig. 4 presents the IDA results obtained for representative archetypes, namely for the PG-1A (8 story) and PG-1D (12 story) archetypes. The median collapse spectral acceleration intensity at 5% maximum inter-story drift, $\hat{S}_{CT}$, and the median spectral acceleration, S_{MT} are also shown in this figure. Here, 5% drift was chosen to be the limiting collapse definition as real buildings would suffer an extensive amount of non-structural damage at that point even if not experiencing global collapse. Hence, for the IDA, the Collapse Margin Ratio (CMR) was calculated based on response values obtained at 5% drift for each archetype, which is the ratio between $\hat{S}_{CT}$ and S_{MT}.

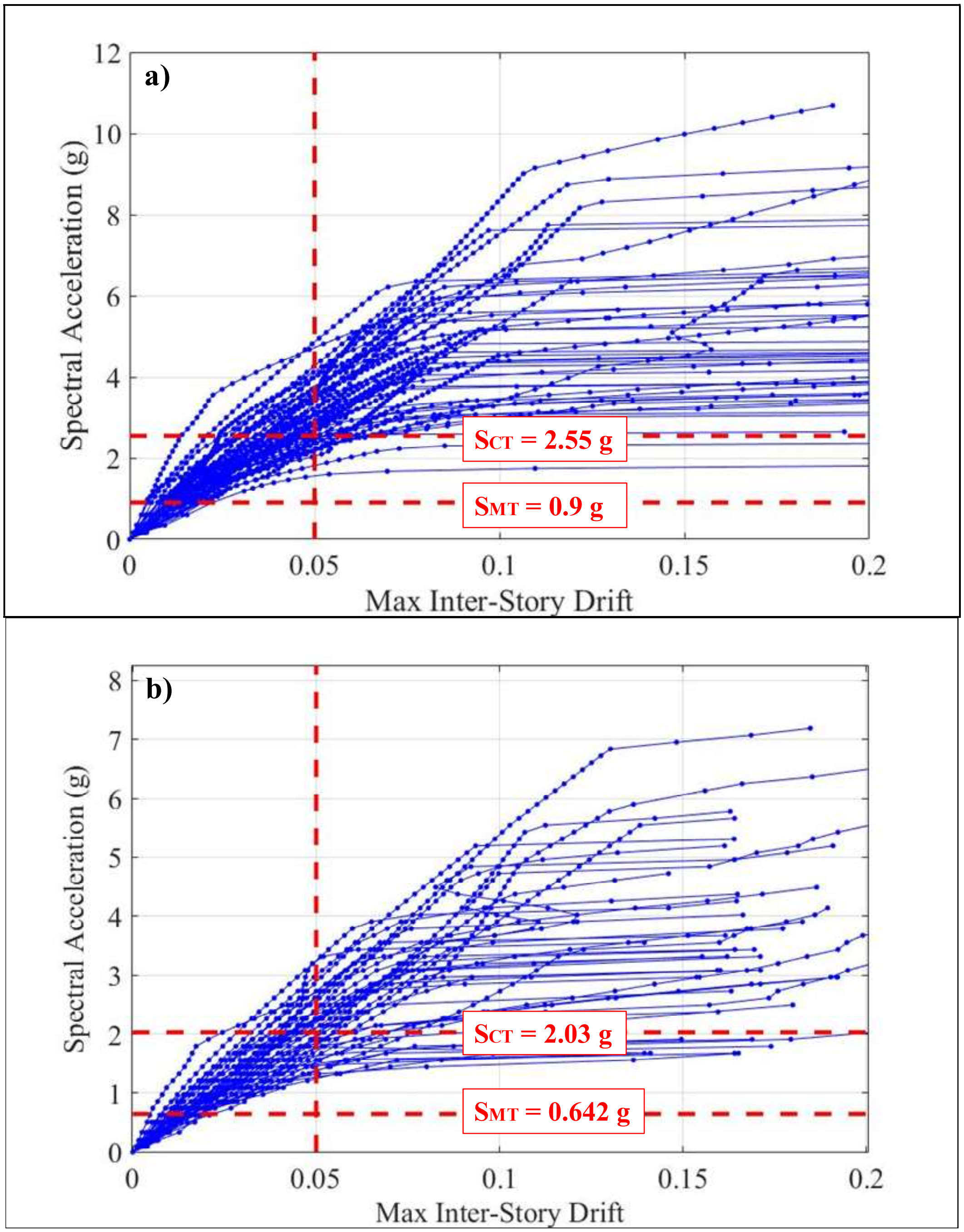

Figure 4. Incremental Dynamic Analysis (IDA) Results for (a) PG-1A and (b) PG-1D.

6. Performance Evaluation

Table 2 provides collapse performance evaluation for the 8- and 12-Story archetypes, by summarizing design information, nonlinear static and dynamic analyses results, and evaluation of the seismic performance factors used for the original design. The initial step of the performance evaluation is to adjust the CMR value obtained from the IDA to take into account the frequency content of the selected ground motion records (i.e., the effect of spectral shape). The spectral shape factor (SSF) values that are used to modify the CMR to the adjusted collapse margin ratio (ACMR) are a function of the archetype fundamental period, the applicable Seismic Design Category (SDC) and period-based ductility (μ_T) attained from nonlinear pushover analysis. The fundamental period (T) of the archetypes were used instead of the maximum period for which archetypes are designed (note that this does not make a significant difference on the results). The period-based ductility (μ_T) 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. The value of SSF are obtained from Tables 7-1a and 7-1b (depending on SDC) in the FEMA P695 document for both archetypes. Accordingly, the ACMR are obtained by multiplying the CMR by the SSF value.

As such, total system collapse uncertainty (β_{TOT}) is required in order to calculate the acceptable ACMR value. The value of β_{TOT} is obtained by combining uncertainty factors related to record-to-record (β_{RTR}), design requirements (β_{DR}), test data (β_{TD}), and nonlinear modeling (β_{MDL}). For the selected ground motions used in the FEMA P695 methodology, a constant value of β_{RTR} equal to 0.4 is used, given that period-based ductility is larger than or equal to 3 ($\mu_T \geq 3$). However, here, all values of β_{DR}, β_{TD}, and β_{MDL} were taken as corresponding to the "poor" condition (β_{DR}, β_{TD}, and $\beta_{MDL} = 0.5$), only to see if satisfactory results could be obtained irrespective of these values (the authors believe that, arguably, all these values of uncertainties could be actually be taken as "good" here, rather than "poor"). The corresponding total system uncertainty is 0.954. The acceptable ACMR for 10% and 20% collapse probability under MCE ground motions (i.e., $ACMR_{10\%}$ and $ACMR_{20\%}$) for β_{TOT} of 0.954 are 3.38 and 2.22 from Table 9-7 in FEMA P695 document, respectively.

The FEMA P695 methodology specifies that $ACMR_{20\%}$ and $ACMR_{10\%}$ are the acceptable threshold values to evaluate performance of individual archetype and average performance of several archetypes in one performance group, respectively. Hence, all archetypes passed the performance requirement. Here, all 8- and 12-Story archetypes are considerably above the $ACMR_{20\%}$ threshold. Likewise, all archetypes also passed $ACMR_{10\%}$ threshold.

Results from the collapse performance evaluations of the 8- and 12-Story archetypes presented in Table 2 indicate that the initial R factor of 8 used to design the archetypes considered is adequate. Results also indicate that the system over-strength factor (Ω_o) could be specified as 2.0 (which is the minimum of all the values tabulated above) and that the deflection amplification factor (C_d) could be equal to 8, similar to the value of R factor.

Table 2. Summary of FEMA P695 Study of archetypes.

Parameter	PG-1A	PG-1D
1. Design Stage		
R	8	8
V_{design}, (kip)	879	979
2. Nonlinear Pushover Analysis		
V_{max}, (kip)	1953.1	2284
$\Omega = V_{max}/V_{design}$	2.22	2.33
$\delta_{y,eff}$	4.16	6.2
δ_u	29.26	36.53
$\mu_T = \delta_u/ \delta_{y,eff}$	7.03	5.89
3. Incremental Dynamic Analysis (IDA)		
S_{CT}	2.55	2.03
S_{MT}	0.9	0.642
$CMR = S_{CT}/S_{MT}$	2.83	3.16
4. Performance Evaluation		
T	1.00	1.38
SDC	D_{max}	D_{max}
$SSF(T,\mu_T,SDC)$	1.25	1.306
$ACMR = SSF(T,\mu_T)xCMR$	3.54	4.13
β_{RTR}	0.4	0.4
β_{DR}	0.5	0.5
β_{TD}	0.5	0.5
β_{MDL}	0.5	0.5
$\beta_{tot} = sqrt\ (\beta_{RTR}^2+\beta_{DR}^2+\beta_{TD}^2+\beta_{MDL}^2)$	0.954	0.954
$ACMR_{20\%}\ (\beta_{tot})$	2.22	2.22
$ACMR_{10\%}\ (\beta_{tot})$	3.38	3.38
$Status_i$	Pass	Pass
$Status_{PG}$	Pass	Pass
5. Final Results		
R	8	8
Ω	2.22	2.33
μ_T	7.03	5.89
C_d	8	8

7. Conclusions

Overall, Composite Plate Shear Walls-Concrete Filled (CPSW/CF) is a new lateral loading resisting system and its seismic response parameters is investigated in this paper as ASCE 7-16 specifications do not distinguish between coupled and non-coupled walls. For that purpose, material and element models for steel and concrete are chosen from OpenSees and their parameters are calibrated based on the behavior observed during tests of each component of this system. Then, the example structures, i.e. archetypes, are generated and nonlinear earthquake analysis were run gradually scaled up from low to high magnitude until collapse is observed. Finally, the collapse assessment analysis were performed by use of FEMA P695 document to conclude if new seismic response parameters can be established or not. Based on the preliminary results obtained to date for 8 and 12 story archetypes, the current R-value of 6.5 specified in ASCE 7-16 for composite walls could potentially increase to a value as high as 8.

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