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EXAMPLE APPLICATION OF THE FEMA P695 (ATC-63) METHODOLOGY FOR THE COLLAPSE PERFORMANCE EVALUATION OF REINFORCED MASONRY SHEAR WALL STRUCTURES

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ABSTRACT

This paper presents a trial application of the FEMA P695 (ATC-63) Methodology for the collapse performance evaluation of ordinary and special load bearing reinforced masonry shear wall systems designed according to current code provisions. The ordinary walls are designed for Seismic Design Category (SDC) C with partial grouting, while the special walls are for SDC D with full grouting. A total of forty wall archetypes are considered with the design variables including the number of stories, wall aspect ratio, and level of gravity load. Neither the special nor the ordinary wall systems pass the acceptance test of the Methodology. However, the performance of the special walls would most likely be acceptable if the assessment of the ductility demands and collapse conditions for the low-rise walls could be improved. In general, the ordinary walls have worse performance largely due to partial grouting. The system overstrength factors obtained for both wall types are comparable to those specified in ASCE/SEI 7-05.

Introduction

This paper presents a trial application of the FEMA P695 (ATC-63) Methodology (ATC 2009) for the collapse performance evaluation of ordinary and special load bearing reinforced masonry shear wall (RMSW) systems as defined in ASCE/SEI 7-05 (ASCE/SEI 2005) and the MSJC code (MSJC 2008). In particular, it is to investigate if walls designed according to these provisions and with the values of the structural response modification coefficient, *R*, specified in ASCE/SEI 7-05 would meet the performance objective of the Methodology, and to determine the values of the resulting system overstrength factor, Ω_o. The ordinary walls are designed for Seismic Design Category (SDC) C_{max} and C_{min}, as defined in the Methodology, with the use of partial grouting, which is most common for such walls, while the special walls are for SDC D_{max} and D_{min} with full grouting, which is a common practice in the western US. While reinforced masonry shear wall systems can have a number of different configurations, including perforated wall systems with regularly or irregularly arranged openings, cantilever walls with strong or weak coupling beams, and walls with flanged or rectangular cross sections, this study focuses on

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weakly coupled rectangular cantilever walls so that a broad range of design variables can be examined with a reasonable number of wall archetypes. A total of forty archetypes are considered with the design variables including the number of stories, wall aspect ratio, and level of gravity load. The wall systems considered here are designed and detailed according to the strength design requirements of the MSJC code. Hence, results of this study should be interpreted with the adopted design method as well as the respective grouting methods used for the ordinary and special walls in mind. Details of this study can be found in the ATC-76 project report (NCJV 2009).

Archetype Design and Configurations

The values of the *R* factor used for the design of the special and ordinary load bearing RMSW systems are 5 and 2, respectively, according to the specification of ASCE 7-05. The corresponding values of the displacement amplification factor, C_d, in the specification are 3.5 and 1.75. However, the Methodology stipulates that C_d be equal to *R* for this type of structural systems. In the analysis of a 12-story special RMSW considered herein, it has been found that when the equivalent lateral force (ELF) procedure is used to calculate the elastic story drift, the use of $C_d = R$ will result in a design that is governed by the story drift, and, thereby, more conservative than what would have been obtained with the C_d value given in ASCE/SEI 7-05. Nevertheless, if the response spectrum analysis (RSA) method is used, the drift of the structure will be much smaller and will not be a governing factor. For the shorter walls and ordinary RMSW's, drift does not appear to be an issue regardless of the analysis method used. Hence, to avoid the design being too conservative, the RSA method is used for the 12-story case to find the elastic drift.

The walls are designed and detailed using the strength design requirements of the MSCJ code. However, to avoid excessively large compressive strains that could be induced by the combined axial load and flexure, the wall designs are checked against the allowable stress design requirements of the MSJC code and are modified if necessary to ensure that the maximum compressive stress limit of $f'_m / 3$ is not violated. It has been found that this allowable stress requirement tends to govern the design of the high-rise partially-grouted walls.

The archetypes considered in this study are divided into 16 performance groups with eight for special and eight for ordinary RMSW's, as shown in Tables 1 and 2. The archetypes are grouped according to the SDC, gravity loads carried, and code-based periods of the structures. All the one-story buildings have similar plan layouts that represent department store-type buildings with large window areas and minimum wall enclosure on the exterior. Figure 1 shows the plan view of a typical single-story building, which is designed for SDC D_{max}. All the shear walls are on the exterior faces of a building, and are 12 ft.-tall and 24-ft. long. The number of walls and plan dimensions can vary from one building to another depending on the seismic design category. To have a lower-bound design without excessive overstrength, the total length of the shear walls in each direction is kept to the minimum required. The interior and exterior columns share the gravity load according to the tributary roof area.

All the multi-story buildings have similar plan layouts representing a broad class of buildings for condominiums, hotels, and college dormitories. For all these buildings, the story height is 10 ft. and the walls are all 32-ft. long. Hence, the wall aspect ratio is proportional to the number of stories. Figure 2 shows the plan view of a 12-story building, which is designed for

SDC D_{max}. It has twelve shear walls in each direction. The walls in the two directions are not structurally connected. Except for the corridor area, the floor and roof systems consist of 8-in.-thick precast hollow core planks with a 2-in. cast-in-place concrete topping. They are supported by the east-west (EW) walls. The corridor area has cast-in-place concrete slabs supported by the north-south (NS) walls. The coupling moments and shear forces introduced by the floor and roof slabs into the wall systems are neglected in the design and analysis. Merryman et al. (1990) and Kingsley et al. (1994) have shown that the axial loads introduced into the walls by the coupling action of the slabs can significantly increase the lateral resistance of a wall system. However, the degree of coupling depends on the continuity of the bottom reinforcement in the slabs and the orientation of the planks at the wall-to-slab connections. Such conditions can vary from one building to another. Therefore, it is appropriate to ignore the coupling action.

Table 1. Special reinforced-masonry shear wall archetype design variables.

Arch. Design ID No.	Design Parameters								
	No. of Stories	Wall Height/ Length (ft.)	Gravity Loads	SDC	No. of Walls in Each Direct.	Roof Weight Per Wall (kips)	Floor Weight Per Wall (kips)	*T* [sec]	T_1 [sec]
Performance Group No. PG-1S and PG-5S									
S1/S11	1	12/24	High/Low	D_{max}	2	646	NA	0.25/0.25	0.10/0.10
S2/S12	2	20/32	High/Low	D_{max}	4	516	617	0.26/0.26	0.13/0.13
S3/S13	4	40/32	High/Low	D_{max}	8	231	286	0.45/0.45	0.21/0.26
Performance Group No. PG-2S and PG-6S									
			High/Low	D_{max}					
S4/S14	8	80/32	High/Low	D_{max}	12	115	151	0.75/0.75	0.55/0.61
S5/S15	12	120/32	High/Low	D_{max}	12	126	174	1.02/1.02	0.93/0.93
Performance Group No. PG-3S and PG-7S									
S6/S16	1	12/24	High/Low	D_{min}	2	1,756	NA	0.25/0.25	0.14/0.14
S7/S17	2	20/32	High/Low	D_{min}	4	893	1,053	0.28/0.28	0.19/0.21
			High/Low	D_{min}					
Performance Group No. PG-4S and PG-8S									
S8/S18	4	40/32	High/Low	D_{min}	4	642	762	0.48/0.48	0.35/0.43
S9/S19	8	80/32	High/Low	D_{min}	4	390	464	0.80/0.80	1.12/1.16
S10/S20	12	120/32	High/Low	D_{min}	4	402	493	1.09/1.09	1.74/1.94

The plan layouts for all the multi-story buildings are similar to that shown in Figure 2 with the plan width fixed at 74 ft. However, the number of walls and the plan length may vary depending on the number of stories and the seismic design category to have a lower-bound design that barely meets the strength requirements. Shear walls are replaced by reinforced concrete gravity frames when they are not needed for the seismic load so that the tributary floor area for gravity load remains the same for each wall. However, the number of walls for the NS direction is always is equal to that for the EW direction.

Tables 1 and 2 summarize the design variables for the archetypes, including the number of stories, the wall geometry, the number of walls in each direction, and the seismic weight per story shared by each wall. In the tables, T is the code-based period calculated according to ASCE/SEI 7-05 and the Methodology, while T_1 is the fundamental period calculated with an eigenvalue analysis using an analytical model. A shaded row in a performance group reflects the fact that the group is short of one archetype to meet the minimum number required by the Methodology.

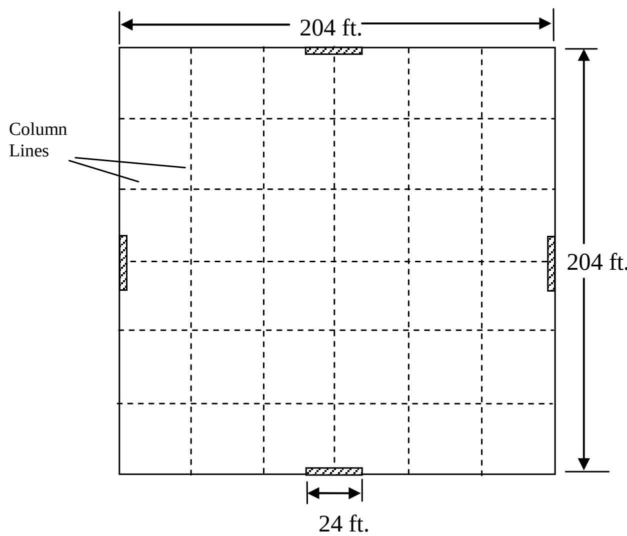

Figure 1. Plan view of single-story building for SDC D_{max}.

Table 2. Ordinary reinforced-masonry shear wall archetype design variables.

Arch. Design ID No.	Design Parameters								
	No. of Stories	Wall Height/ Length (ft.)	Gravity Loads	SDC	No. of Walls in Each Direct.	Roof Weight Per Wall (kips)	Floor Weight Per Wall (kips)	T [sec]	T_1 [sec]
Performance Group No. PG-1O and PG-5O									
O1/O11	1	12/24	High/Low	C_{max}	4	318	NA	0.25/0.25	0.10/0.10
O2/O12	2	20/32	High/Low	C_{max}	4	381	450	0.28/0.28	0.19/0.17
			High/Low	C_{max}					
Performance Group No. PG-2O and PG-6O									
O3/O13	4	40/32	High/Low	C_{max}	8	158	193	0.48/0.48	0.28/0.28
O4/O14	8	80/32	High/Low	C_{max}	12	107	135	0.80/0.80	0.59/0.59
O5/O15	12	120/32	High/Low	C_{max}	12	111	144	1.09/1.09	1.00/1.04
Performance Group No. PG-3O and PG-7O									
O6/O16	1	12/24	High/Low	C_{min}	4	873	NA	0.25/0.25	0.16/0.16
O7/O17	2	20/32	High/Low	C_{min}	4	631	740	0.31/0.31	0.20/0.20
			High/Low	C_{min}					
Performance Group No. PG-4O and PG-8O									
O8/O18	4	40/32	High/Low	C_{min}	4	319	374	0.52/0.52	0.43/0.42
O9/O19	8	80/32	High/Low	C_{min}	8	192	232	0.87/0.87	0.72/0.82

O10/O20	12	120/32	High/Low	C_{min}	8	163	200	1.18/1.18	1.19/1.24

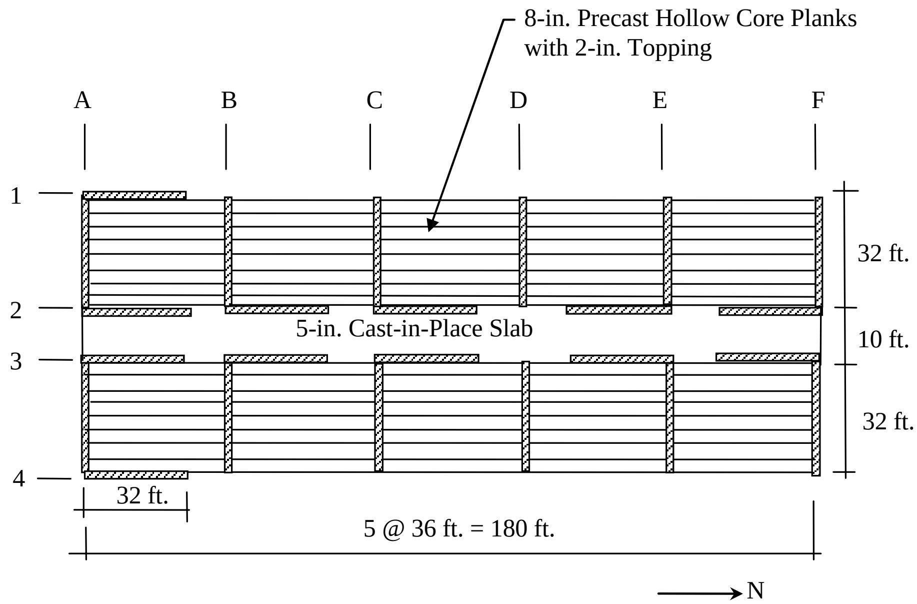

Figure 2. Plan view of 12-story building for SDC D_{max}.

For the multi-story walls, the reinforcement quantities and nominal masonry design strength are changed every other story whenever practical to have an economical design. However, the wall thickness remains the same for the entire building height. Since the wind load varies with the geographic location, it is ignored in the design so that the archetypes in lower seismic design categories will not be over-designed with respect to the seismic load. The out-of-plane seismic load is found to be not critical for the unsupported wall heights considered here.

Nonlinear Static and Dynamic Analyses

For nonlinear analysis, each archetype is idealized as an uncoupled cantilever wall with appropriate gravity load and seismic mass determined from the respective tributary areas. The analyses are conducted with OpenSees developed by the PEER Center (http://opensees.berkeley.edu/index.php) using displacement-based fiber-section beam-column elements to model the flexural behavior of a wall. The shear deformation of a wall is modeled with zero-length elastic springs. Hence, inelastic shear behavior is not accounted for and shear failure is treated as a non-simulated mode. One shear spring is used for each story, and its stiffness is equal to the elastic shear stiffness of the story below. The discretization scheme adopted for each archetype model, i.e., the number and lengths of beam-column elements used, depends on the effective plastic-hinge length at the wall base and the height of the bottom story. The plastic-hinge length of a wall is assumed to be 20% of the total wall height based on the experimental data examined. If the plastic-hinge length is close to or larger than the story height, each story of the archetype is represented by one beam-column element. Otherwise, the bottom story is modeled with two beam-column elements and the length of the element closest to the base is equal to the effective plastic-hinge length, while each of the upper stories is represented by one element. The P-Δ effect is accounted for by using the co-rotational transformation in the beam-column element. However, this influence is found to be insignificant to the in-plane response of a wall.

The Kent-Park model (Kent and Park 1971) for concrete, which is available in OpenSees, is adopted to model the compressive behavior of masonry. The model assumes zero tensile strength and exhibits stiffness degradation in compressive unloading and reloading. The expected compressive strength f'_m of masonry is assumed to be 1.25 times the nominal strength chosen for design. This is based on the prism test data provided in the Commentary of the MSJC code (MSJC 2008).

The envelope of the reference stress-strain relation selected for the reinforcing bars is shown in Figure 3. Grade-60 steel is chosen. The expected yield and tensile strengths of a bar are assumed to be 1.13 times the nominal strengths based on the study of Nowak et al. (2008). This results in 68 and 102 ksi for the yield and tensile strengths. The steel models in OpenSees cannot simulate the buckling or rupture of a bar, which is important for assessing the collapse capacity of an archetype. Hence, the stress-strain relation used in this study is obtained from appropriate calibration of the general hysteretic model in OpenSees. It is assumed that the tensile rupture of a steel bar occurs at a strain of 0.05, which is about one half of the strain at which a bar reaches its peak tensile strength, to account for the low-cycle fatigue phenomenon. After this, the tensile strength of the bar decreases linearly and reach zero at a tensile strain of 0.10. A bar will buckle when the masonry around the bar spalls significantly. This is simulated in the user-defined model by introducing a compressive strain softening, which starts at a compressive strain of 0.0083. This is the strain level at which the masonry compressive strength drops to 40% of the peak value, signifying the occurrence of severe spalling. After buckling, it is assumed that the compressive strength of a bar will drop to 10% of the yield strength when its strain reaches 0.016, after which the residual strength remains constant.

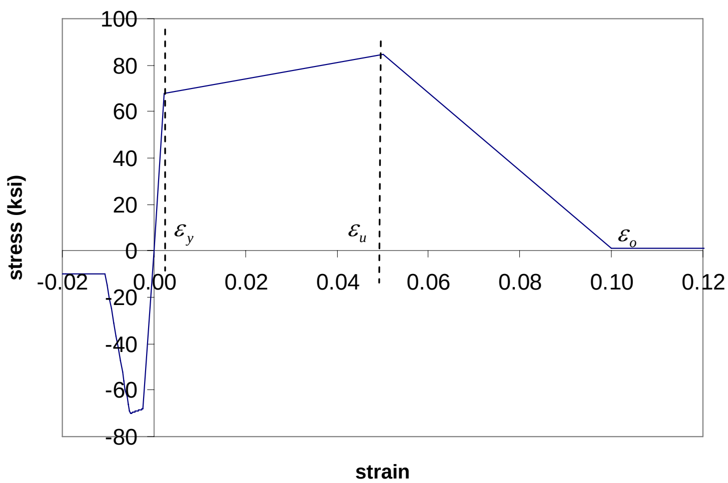

Figure 3. Reference stress-strain relation for steel.

When using displacement-based beam-column elements with distributed plasticity and strain-softening material laws to model a shear wall, inelastic deformation tends to concentrate in a single element at the wall base while rest of the model remains elastic. This phenomenon, termed strain localization leads to numerical results that are sensitive to the length of the element in which the plastic strain is localized. To circumvent this problem, the material stress-strain relations have to be modified when the element size is different from the plastic-hinge length to have the realistic total fracture-energy dissipation. The regularization method used here is based on that proposed by Coleman and Spacone (2001) for force-based beam elements.

For each archetype, a pushover analysis is conducted with a lateral load distribution that corresponds to the fundamental mode shape and mass distribution of the structure. In general, it can be observed that that walls subjected to higher axial loads are less ductile because they develop more severe toe crushing, and that the low-rise walls are less ductile than the high-rise walls. Partially-grouted walls are in general less ductile as expected. Results of the pushover analyses have been used to establish the flexural collapse criteria employed in the nonlinear dynamic analyses. Based on the post-peak response observed from the base shear-vs.-roof displacement curves, as shown for two cases in Fig. 4, it is assumed that flexural collapse is triggered by either one of the following two conditions: (a) excessive crushing in the wall cross section, which is defined as the condition that 30% of the cross section has reached the end of the softening branch of the stress-strain relation for masonry; and (b) excessive reinforcing bar fracture or buckling, which is defined as the condition that 30% or more of the bars at a wall cross section has either lost their tensile resistance due to rupture or reached their residual compressive resistance due to buckling. Using bar rupture as the collapse criterion is conservative for the one-story and two-story archetypes as it may not induce immediate collapse.

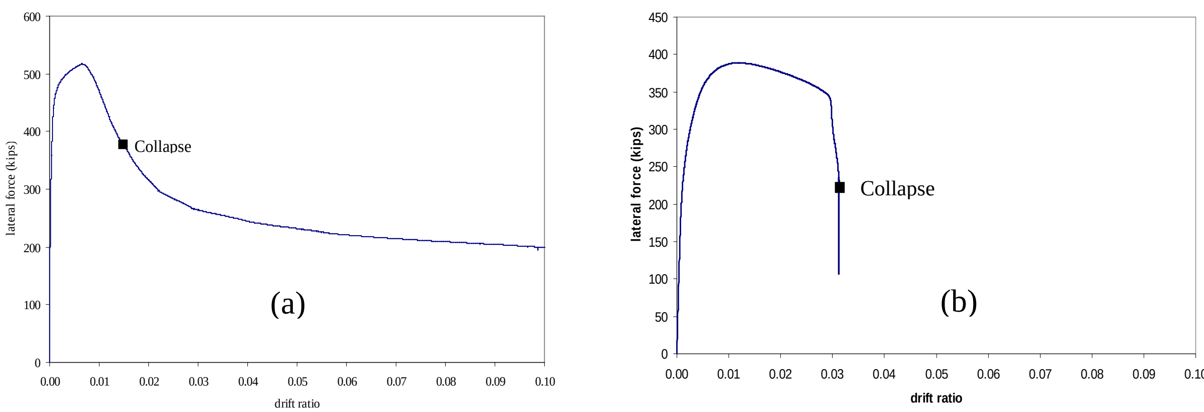

Figure 4. Pushover curves and detection of collapse for (a) a 2-story building, and (b) a 12-story building, both designed for SDC D_{max} with high axial loads.

Collapse due to diagonal shear cracks is considered to occur when the shear force in a wall exceeds the nominal shear strength calculated with the formula in the MSJC provisions. This is a non-simulated collapse mode. In the 8-story and 12-story fully-grouted archetypes with low axial loads, shear collapse occurs in many cases due to the higher-mode effect. Shear collapse also occurs in the low-rise, partially-grouted walls due to the absence of a capacity design requirement for the ordinary walls.

The collapse performance is evaluated with a simplified Incremental Dynamic Analysis (IDA) method. Each archetype is subjected to a set of 44 far-field records provided in the FEMA P695 Methodology (ATC 2009). The records are scaled up gradually by the same factor each time until collapse was obtained for 50% of the record set. At this point, the median spectral acceleration of the record set at the structural period *T* is taken as the median collapse spectral intensity S_{CT}. The ratio of S_{CT} to S_{MT}, which is the MCE-level spectral acceleration at the same structural period, is considered as the collapse margin ratio (*CMR*).

Performance Evaluation

According to the Methodology, the performance of a structural system will be acceptable provided that the average value of the adjusted collapse margin ratio (*ACMR*) for each performance group exceeds a value (*ACMR10%)* that corresponds to a probability of collapse no greater than 10% for MCE-level ground motions, and that the *ACMR* of each index archetype within a performance group exceeds a value (*ACMR20%*) that corresponds to a probability of collapse no greater than 20%. The *ACMR*'s are obtained by multiplying the *CMR*'s computed from the nonlinear time history analyses by the respective spectral shape factors *SSF*. The *SSF* for each index archetype is determined according to its code-based fundamental period *T* and period-based ductility μ_T using Table 7-1 in the ATC-63 project report (ATC 2009). The values of *ACMR10%* and *ACMR20%* are determined from the values of the total system uncertainly parameter β_{TOT} using Table 7-3 in the ATC-63 report. The β_{TOT} value is a function of the quality ratings of the nonlinear models, test data, and design requirements as well as the record-to-record uncertainty. The β_{TOT} values are determined with either Equation (7-5) or Table 7-2 in the ATC-63 report. The quality ratings determined for the special and ordinary RMSW's are summarized in Table 3.

Table 3. Summary of quality ratings.

	Design Requirements	Test Data	Nonlinear Models
Special RMSW's	B	B	B
Ordinary RMSW's	C	C	B

The values of μ_T, *SSF*, *ACMR*, and acceptable *ACMR* (which is *ACMR20%* for each index archetype and *ACMR10%* for each performance group) determined for the special and ordinary RMSW archetypes can be found in Chapter 3 of the ATC-76 project report (NCJV 2009). For the special walls, all the one-story walls and one two-story wall fail the acceptance tests, while the rest pass the test. The relatively poor performance of the low-rise walls can be attributed to two factors. One is the higher ductility demand of the earthquake ground motions on structures of shorter periods, which is typical. The second is the lower ductility capacity of the shorter walls. The ductility demand and capacity for low-rise walls require additional considerations. Low-rise walls are generally stiffer, and the ductility demands on these structures can be more significantly influenced (lowered) by the soil-structure interaction effect, which has not been accounted for in the analyses performed here. However, soil-structure interaction may reduce the ductility capacity because of the increase in yield drift. Furthermore, the sudden load drop and collapse of the special reinforced masonry low-rise walls in the analysis is triggered by the rupture of the vertical reinforcement, which might not necessarily lead to collapse for low-rise structures. For low-rise squat walls, a more likely failure scenario is base sliding. In either case, a low-rise shear wall may not loose stability because of severe rocking or base sliding. Rather, collapse is expected to occur only when the drift is so significant that other gravity-load carrying systems, such as gravity frames and walls subjected to out-of-plane deformation, in the structure become unstable. In spite of the above considerations, the appropriateness of assigning the same *R* value to low-rise and high-rise walls deserves further study given the fact that the ductility demands for the two cases could be very different.

For the ordinary RMSW archetypes, all the performance groups except PG-6O and PG-8O fail the acceptance test. The two performance groups that pass the test consist of 4 to12-story walls. The poor performance of the ordinary walls can be attributed to two factors. One is the quality ratings, which are lower than those of the special walls, and the other is the lower ductility of the ordinary walls, which affect the values of the *ACMR*. The latter is, however, the main factor. For the one- and two-story partially-grouted walls, collapse is caused by diagonal shear failures, while the taller walls fail with excessive masonry crushing.

Even though both the special and the ordinary wall systems do not meet the acceptance criteria of the Methodology, the performance of the special walls would most likely be acceptable if the assessment of the ductility demands and collapse conditions for the low-rise walls could be improved with the development of more appropriate and consistent guidelines for such analysis. However, it should be emphasized that the different performances of the special and ordinary wall systems can be largely attributed to the grouting methods. If the special walls were partially-grouted, which is allowed by the code but uncommon in the West Coast, their performance would be less desirable.

The system overstrength factor Ω_o is the largest average value of the overstrength factor Ω computed for each performance group. The system overstrength factor Ω_o calculated for the special RMSW archetypes is 2.12, while that for the ordinary RMSW archetypes is 2.08. They are smaller than the value of 2.5 given in ASCE/SEI 7-05 for both systems.

Conclusions

In this study, neither the special nor the ordinary wall systems pass the acceptance test of the Methodology. For the special wall systems, all the one-story walls and one two-story wall fail the tests, while the rest pass the test. This raises an issue as to whether the low-rise walls should have the same *R* factor as the high-rise walls, as well as whether the same collapse criteria could be used and the soil-structure interaction effect should be considered in the dynamic analysis. The performance of the special walls would most likely be acceptable if the assessment of the ductility demands and collapse conditions for the low-rise walls could be improved with the development of more appropriate and consistent guidelines for such analysis. For the ordinary wall systems, the tall walls also have a problem in passing the test. In general, the ordinary walls have worse performance largely due to partial grouting. For the less ductile systems, the Methodology seems to have a more severe performance standard than the current approach for the determination of the *R* factor. It requires the consideration of the performance of a structure under earthquake ground motions way exceeding the maximum considered earthquake (MCE). This is especially demanding for the less ductile systems that are not appropriately detailed for severe seismic loads. However, the stringent criteria can be justified to assure a uniform risk for collapse. The system overstrength factors obtained in this study are comparable to that specified in ASCE/SEI 7-05. It should be noted that the wall systems considered here are designed and detailed according to the strength design requirements of the MSJC code. Results of this study should be interpreted with these as well as the grouting conditions in mind.

This study has been focused on rectangular cantilever wall systems. However, masonry buildings may have different wall configurations, such as flanged walls and strongly coupled walls. Many masonry buildings are low-rise box systems with perforated shear walls having various opening sizes and configurations. All these systems need to be studied in the future to fully characterize the seismic performance of reinforced masonry structures and identify their appropriate seismic performance factors. Current code provisions do not adequately distinguish the wide range of performance characteristics of different masonry wall systems.

Additional experimental data are needed to calibrate analytical models for different wall systems. The reliability of an analytical model depends highly on a good estimation of the effective plastic-hinge length in a flexure-dominated wall. Experimental data on this are limited, especially for walls with height/length ratios greater than one. The modeling of perforated wall systems presents a major challenge. Experimental data for this type of systems are extremely limited. Improved analytical models are also needed to simulate the diagonal shear failure of a wall system.

Acknowledgments

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References

ASCE/SEI, 2005. *Minimum Design Loads for Buildings and Other Structures (ASCE/SEI 7-05)*, American Society of Civil Engineers/Structural Engineering Institute, Reston, VA.

ATC, 2009. *Quantification of Building Seismic Performance Factors*, FEMA P695, ATC-63 Project Report, Applied Technology Council, Redwood City, CA.

Coleman, J. and Spacone, E., 2001. Localization Issues in Forced-Based Frame Elements, J*ournal of Structural Engineering* 127(11), 1257-1265.

Kent, D.C. and Park, R., 1971. Flexural Members with Confined Concrete, *Journal of the Structural Division* 97(7), American Society of Civil Engineers, 1969-1990.

Kingsley, G.R., Seible, F., Priestley, M.J.N., and Hegemier, G.A., 1994. The U.S.-TCCMAR Full-Scale Five-Story Reinforced Masonry Research Building Test: Part II, Design, Construction, and Testing, *Report No. 9.4-2*, US-Japan Coordinated Program for Masonry Building Research, University of California at San Diego, La Jolla, CA.

Merryman, K.M., Leiva, G., Antrobus, N., and Klingner, R.E., 1990. In-plane Seismic Resistance of Two-Story Concrete Masonry Coupled Shear Walls, *Report No. 3.1(c)-1*, US-Japan Coordinated Program for Masonry Building Research, University of Texas at Austin, Austin, TX.

MSJC, 2008. *Building Code Requirements for Masonry Structures (TMS 402-08 / ACI-530-08 / ASCE 5-08)*, The masonry Society, American Concrete Institute, and ASCE/Structural Engineering Institute.

NCJV, 2009. *Evaluation of the FEMA P695 Methodology for Quantification of Building Seismic Performance Factors*, ATC-76-1/4 Project Report – 90% Draft, NEHRP Consultants Joint Venture, Applied technology Council and the Consortium of Universities for Research in Earthquake Engineering.

Nowak, A.S., Szersen, M.M., Szeliga, E.K., Szwed, A., and Podhorecki, P.J., 2008. Reliability-Based Calibration for Structural Concrete, Phase 3, *PCA R&D Serial No. 2849*, Portland Cement Association, Skokie, IL.