EXAMPLE APPLICATION OF THE FEMA P695 (ATC-63) METHODOLOGY FOR THE COLLAPSE PERFORMANCE EVALUATION OF REINFORCED CONCRETE SPECIAL MOMENT FRAME SYSTEMS

C.B. Haselton[1], A.B. Liel[2] and G.G. Deierlein[3]

ABSTRACT

This paper describes an example application of the newly developed FEMA P695 (ATC-63) methodology for assessing collapse performance. This methodology is applied to assessing the collapse performance of code-conforming reinforced concrete (RC) special moment frame (SMF) buildings. This example includes all of the aspects of the methodology, including definition of the representative structural designs, nonlinear dynamic simulation to collapse, calibration of the numerical model to test data, assessment of uncertainties, and the final probabilistic evaluation of collapse performance. This process shows that RC SMF buildings have pass the methodology and are deemed to have acceptable collapse safety. This finding has one caveat; this work found that use of the ASCE 7-05 provisions result in unacceptable collapse safety for some tall buildings. To remedy this, the minimum base shear requirement of ASCE 7-02 is reinstated so that acceptable collapse safety can be achieved. The example is fully consistent with the final version of the FEMA P695 Methodology and can be used by potential users to understand the steps and calculations needed to assess a new seismic resisting system according to this Methodology.

Introduction and Purpose

The Applied Technology Council, under the direction of the Federal Emergency Management Agency, has recently completed the FEMA P695 (ATC-63) document which contains a formalized assessment methodology for quantifying structural collapse safety under seismic loading. This methodology is described in related companion papers [Kircher and Heintz 2009].

This paper illustrates how the FEMA P695 (ATC-63) performance assessment methodology can be applied to the class of reinforced concrete (RC) special moment frames (SMFs). The purpose of this example is two-fold: (a) to illustrate an application of the methodology; and (b) to “benchmark” the methodology by checking the collapse performance of a code-approved structural system.

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[1]Assistant Professor, Dept. of Civil Engineering, California State University Chico, Chico, CA 95929-0930
[2]Assistant Professor, Dept. of Civil, Environmental and Architectural Engineering, University of Colorado, Boulder, CO, 80309
[3]Professor and Director of the Blume Earthquake Engineering Research Center, Dept. of Civil and Environmental Engineering, Stanford University, Stanford, CA 94305

Structural System Information

Design Requirements

This example assessment of RC SMFs is based on the ASCE 7-05 and ACI 318-05 design requirements; these are used in place of the requirements that would need to be developed for a newly proposed system. Based on these guidelines, the example tests if the current value of R = 8, along with the many other provisions that govern the strength and stiffness of the building, is appropriate for the design of RC SMF buildings. For the purpose of assessing uncertainty, these design requirements are categorized as "A-Superior," since they represent many years of development and include lessons learned from a number of major earthquakes.

Test Data

This example assessment relies on existing published test data in place of test data that would be developed for a newly proposed system. Test data is obtained from the Pacific Earthquake Engineering Research Center's Structural Performance Database that was developed by Berry, Parrish, and Eberhard (PEER 2009). To develop element models, the data are utilized from cyclic tests of 255 rectangular columns failing in flexure and flexure-shear (Haselton et al. 2008).

The quality of the test data is an important consideration when quantifying the uncertainty in the overall collapse assessment process. The test data used in this example cover a wide range of column design configurations and contain both monotonic and cyclic loading protocols. Even so, many of the loading protocols are not continued to deformations large enough for the capping point (which is discussed later in the modeling section) to be observed, and it is difficult to use such data to calibrate models for structural collapse assessment. These test data also do not include beam elements with attached slabs. Additionally, these data include no systematic test series that *both* (a) subject similar specimens to different loading protocols (e.g. monotonic and cyclic) and (b) continue the loading to deformations large enough for the capping behavior to be observed. Lastly, only column element tests were utilized when calibrating the element model, and sub-assemblage tests and/or full-scale tests were not used. Based on these observations, for the purpose of assessing uncertainty, this test data set is categorized as "B-Good."

Identification of RC SMF Archetype Configurations

Figure 1 shows the two-dimensional three-bay multi-story frame selected as the archetype configuration for RC frame buildings. This archetype model includes joint panels, beam and column elements, elastic foundation springs, and a leaning column to account for the P-Delta effect from loads on the gravity system. The gravity system is not modeled, according to the requirements of the FEMA P695 (ATC-63) methodology.

Using this archetype model configuration, a set of structural archetype designs was developed to represent the archetype design space. Following the requirements of FEMA P695, there are 16 archetype performance groups that must be considered for this system, as shown in Table 1. Based on pilot studies, it was found that only six full peformance groups (instead of 16) were required to find the critical performance cases for RC SMF buildings, as show in Table 1.

Instead of designing and assessing buildings for all 16 performance groups, initial pilot studies were used to find the more critical design cases, which were high-seismic designs with 20' bay spacing. By utilizing these pilot studies and then focusing on the critical design cases,

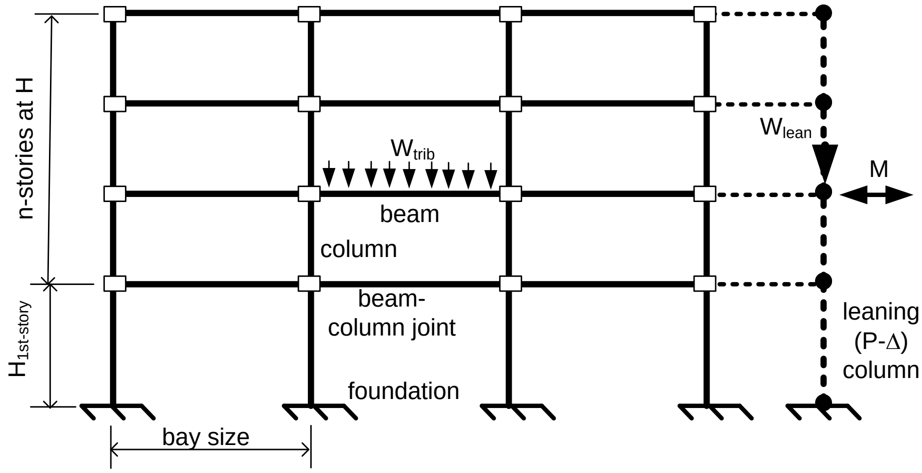

Figure 1. Archetype analysis model for moment frame buildings

Table 1. Performance groups for evaluation of RC special moment frame archetypes.

Performance Group Summary					
Group No.	Grouping Criteria				Number of Archetypes
	Basic Config.	Design Load Level		Period Domain	
		Gravity	Seismic		
PG-1	20-Foot Bay Width	High (Space Frame)	SDC D_{max}	Short	2+1[1]
PG-2				Long	4
PG-3			SDC D_{min}	Short	0
PG-4				Long	1[2]
PG-5		Low (Perimeter Frame)	SDC D_{max}	Short	2+1[1]
PG-6				Long	4
PG-7			SDC D_{min}	Short	0
PG-8				Long	3[2]
PG-9	30-Foot Bay Width	High (Space Frame)	SDC D_{max}	Short	0
PG-10				Long	1[3]
PG-11			SDC D_{min}	Short	0
PG-12				Long	
PG-13		Low (Perimeter Frame)	SDC D_{max}	Short	0
PG-14				Long	1[3]
PG-15			SDC D_{min}	Short	0
PG-16				Long	

1. Example includes only two archetypes for each short-period Performance Group (PG-1 and PG-5); full implementation of the Methodology requires a total of 3 archetypes in each Performance Group.
2. Example evaluates a selected number of low seismic (SDC D_{min}) archetypes to determine that high seismic (SDC D_{max}) archetypes control the *R* factor.
3. Example evaluates two 30-foot bay width archetypes to determine that 30' bay width archetypes do not control performance.

use of 20 archetypes (in eight performance groups) were found to be sufficient. Focusing on these critical design cases may add conservatism to the assessment process, but also allows for consideration of a wider range of design variants (more heights, etc.). Table 2 shows the properties for each of these designs. The archetypes are designed for a soil site (Site Class D) conditions and design lateral loads of Ss = 1.5g and S_1 = 0.6g for SDC D_{max} and Ss = 0.55g and S_1 = 0.13g for SDC D_{min}[2].

Table 2. Archetype structural design properties

Archetype Design ID Number	No. of Stories	Key Archetype Design Parameters						
		Framing (Gravity Loads)	Seismic Design Criteria					$S_{MT}(T)$ [g]
			SDC	*R*	*T* [sec]	T_1 [sec]	*V/W* [g]	
Performance Group No. PG-5 (Short Period, 20' Bay Width Configuration)								
2069	1	P	D_{max}	8	0.26	0.71	0.125	1.50
2064	2	P	D_{max}	8	0.45	0.66	0.125	1.50
--	3	P	D_{max}	8	0.63	--	0.119	1.43
Performance Group No. PG-6 (Long Period, 20' Bay Width Configuration)								
1003	4	P	D_{max}	8	0.81	1.12	0.092	1.11
1011	8	P	D_{max}	8	1.49	1.71	0.050	0.60
5013	12	P	D_{max}	8	2.13	2.01	0.035	0.42
5020	20	P	D_{max}	8	3.36	2.63	0.022	0.27
Performance Group No. PG-1 (Short Period, 20' Bay Width Configuration)								
2061	1	S	D_{max}	8	0.26	0.42	0.125	1.50
1001	2	S	D_{max}	8	0.45	0.63	0.125	1.50
--	3	S	D_{max}	8	0.63	--	0.119	1.43
Performance Group No. PG-3 (Long Period, 20' Bay Width Configuration)								
1008	4	S	D_{max}	8	0.81	0.94	0.092	1.11
1012	8	S	D_{max}	8	1.49	1.80	0.050	0.60
5014	12	S	D_{max}	8	2.13	2.14	0.035	0.42
5021	20	S	D_{max}	8	3.36	2.36	0.022	0.27
Selected Archetypes - Performance Group Nos. PG-4 and PG-8 (20' Bay Width)								
6011	8	P	D_{min}	8	1.60	3.00	0.013	0.15
6013	12	P	D_{min}	8	2.28	3.35	0.010	0.10
6020	20	P	D_{min}	8	3.60	4.08	0.010	0.065
6021	20	S	D_{min}	8	3.60	4.03	0.010	0.065
Selected Archetypes - Performance Group Nos. PG-10 and PG-14 (30' Bay Width)								
1009	4	P-30	D_{max}	8	1.03	1.16	0.092	1.03
1010	4	S-30	D_{max}	8	1.03	0.86	0.092	1.03

Development of Nonlinear Models of Structural Archetypes

The core of the FEMA P695 methodology is explicit modeling of structural collapse. This is a challenging task, but current research developments allow for collapse simulation of some structural systems. This section outlines these current developments, while (FEMA 2009 and Haselton et al. 2008) provide additional detail regarding nonlinear structural modeling of RC

[2] This class of buildings was designed for Ss = 0.38g and S_1 = 0.1g, which differs slightly from the Ss = 0.55g and S_1 = 0.13g required by these guidelines for SDC D_{min}. Even so, the results are normalized by the design level, so this still provides a reasonable check of the SDC D_{min} seismic loading conditions.

SMF buildings. The system-level modeling uses the three-bay multi-story frame configuration shown in Figure 1.

For element-level modeling, many RC element models exist, but most cannot be used to simulate structural collapse. Recent research by Ibarra, Medina, and Krawinkler (2005) has resulted in an element model that is capable of capturing the severe deterioration that precipitates sideway collapse. Figure 2 shows the tri-linear monotonic backbone curve and sample results of calibration which illustrates the hysteretic behavior of this model. This model was calibrated to over 255 experimental tests by Haselton et al. (2008). An important feature of this model is the "capping point" (identified by θ_{cap} on the figure) where monotonic strength loss begins, and the post-capping negative stiffness, which enables modeling of the strain softening behavior associated with concrete crushing, rebar buckling and fracture, or bond failure. Direct simulation of sidesway structural collapse is not possible without modeling this post-capping behavior.

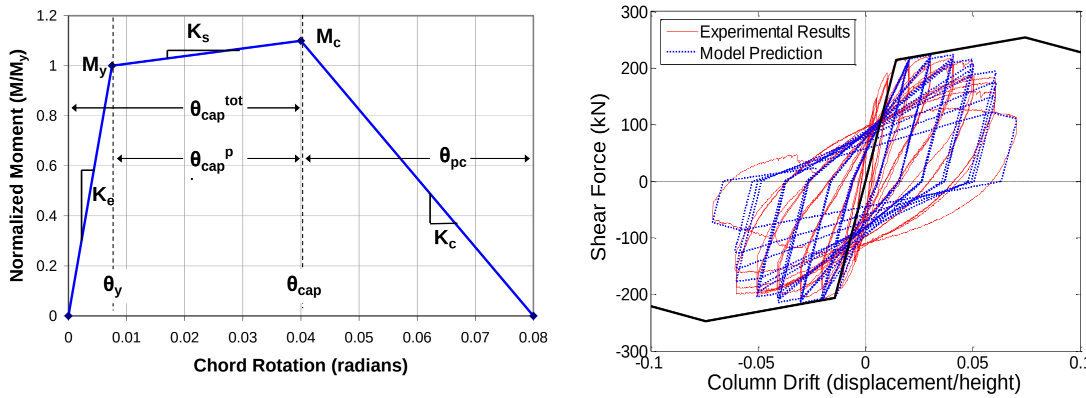

Figure 2. Illustration of experimental and calibrated element response [Saatciolgu and Grira, 1999, spec. BG-6]. The solid black line shows the calibrated monotonic backbone.

Uncertainty due to Model Quality

For the purpose of assessing uncertainty, this modeling is rated as "A-Superior." This rating is based on SMF buildings, and their elements, being controlled by the many detailing and capacity design requirements of the building code, which limit possible failure modes. The primary expected failure mode is flexural hinging leading to sidesway collapse, which the modeling approach can simulate reasonably well by capturing post-peak degrading response (under both monotonic and cyclic loading). The "A-Superior" rating does not indicate that this modeling approach is perfect, but signifies that the modeling approach is able to directly simulate structural response up to collapse (simulating all expected modes of damage that could lead to collapse), and is well-calibrated to the available test data.

Nonlinear Structural Analyses

The structural analysis software utilized for this example is the Open Systems for Earthquake Engineering Simulation (OpenSees 2009), which has been developed by the Pacific Earthquake Engineering Research (PEER) Center. This software includes the modeling aspects required for collapse simulation of RC SMF buildings: the Ibarra et al. element model, joint models, a large deformation geometric transformation, and several robust numerical algorithms for solving the systems of equations associated with nonlinear dynamic and static analyses.

To compute the system overstrength factor (Ω_0) and to help verify the structural model, monotonic static pushover analysis is used, with the lateral load pattern of ASCE7-05 (2005).

To compute the collapse capacity for each archetype design, the Incremental Dynamic Analysis (IDA) approach is used (Vamvatsikos and Cornell, 2002), with the 44 records of the far-field ground motion set which is required by the FEMA P695 methodology. Note that the goal of IDA within the context of FEMA P695 is to quantify the median collapse capacity of each archetype model (estimating the record-to-record variability is not required).

In this example, it is assumed that RC SMF buildings only collapse in a sideway mechanism, which can be directly simulated using the structural analysis model. This assertion is made due to the many detailing, continuity, and capacity design provisions preventing other collapse modes. For structural systems where some collapse modes are not simulated by the structural model, these additional modes must be accounted for using the method required by FEMA P695.

Table 3 later summarizes the results of these analyses. The collapse capacity is expressed in terms of the collapse margin ratio (CMR) which is the ratio between the median collapse capacity (S_{CT}), and the maximum considered earthquake demand level (S_{MT}). These IDA results shows that the average CMR values are 1.23-1.34 for the sets of perimeter frames and 1.56-2.01 for the sets of space frames, but these values have not yet been adjusted for the beneficial effects of spectral shape. The results in Table 3 verify that the buildings designed in low-seismic regions, and those with 30' bay width, have higher CMR (lower collapse risk) compared with the sets of buildings designed with 20' bay width and in high-seismic regions. Therefore, the remaining assessment focuses on these 14 more critical archetype buildings.

Performance Evaluation

The CMR computed in the last section does not account for unique spectral shape of rare ground motions, which has a large impact and increases the collapse capacity (Baker and Cornell, 2006). To account for spectral shape, the FEMA P695 (ATC-63) document includes simplified spectral shape factors (SSFs) which depend on the building period, building ductility capacity, and the properties of nearby faults (approximately quantified by the seismic design category). For SDC B or C, the SSF values range from 1.0 to 1.37, and the values range from 1.0 to 1.61 for SDC D. The adjusted collapse margin ratio (ACMR) is computed as the multiple of the SSF and the CMR. Table 3 presents the SSF and ACMR values for each of the archetype buildings included in this assessment.

In addition to quantifying the ACMR, the acceptance criteria are also needed, which depend on both the allowable conditional collapse probability and the overall uncertainty in the assessment process. Regarding allowable collapse probabilities, the FEMA P695 method requires that, given that a MCE ground motion (rare motion, commonly with 2475 year return period) has occurred, the average conditional collapse probability be $\leq$ 10% for each Performance Group. In addition, FEMA P695 requires that no one archetype model have a conditional collapse probability > 20%. For the uncertainties in this example, the composite uncertainty (β_{TOT}) in collapse capacity is 0.50; this accounts for the variability between ground motion records of a given spectral intensity (assumed to be constant for all structural systems, β_{RTR} = 0.40), the quality of the test data used to calibrate the element models, and the quality of the structural system design requirements. Assuming a lognormal distribution for the collapse capacity, these values result in a required average ACMR of $\geq$1.90 for each Performance Group, and required individual values of the ACMR of $\geq$1.52 for each archetype building.

Table 3 presents the final results and acceptance criteria for each of the archetype designs. This shows the collapse margin ratio computed directly from IDA (CMR), the SSF, and the final adjusted collapse margin ratio (ACMR). The acceptable margins are then shown and each individual archetype and each performance group are shown to either pass of fail the acceptance criteria.

Table 3. Summary of final collapse margins and comparison to acceptance criteria

Arch. Design ID No.	Design Configuration			Computed Overstrength and Collapse Margin Parameters					Acceptance Check	
	No. of Stories	Framing (Gravity Loads)	SDC	Static Ω	*CMR*	μ_T	*SSF*	*ACMR*	Accept. *ACMR*	Pass/ Fail
Performance Group No. PG-5 (Short Period, 20' Bay Width Configuration)										
2069	1	P	Dmax	1.6	1.18	14.0	1.33	1.57	1.52	Pass
2064	2	P	Dmax	1.8	1.50	19.6	1.33	2.00	1.52	Pass
--	3	P	Dmax	1.7*	--	--	--	2.13*	--	--
Mean of Performance Group:				**1.7***	**1.34**	**16.8**	**1.33**	**1.90***	**1.90**	**Pass**
Performance Group No. PG-6 (Long Period, 20' Beam Span Configuration)										
1003	4	P	Dmax	1.6	1.61	10.9	1.41	2.27	1.52	Pass
1011	8	P	Dmax	1.6	1.25	9.8	1.61	2.01	1.52	Pass
5013	12	P	Dmax	1.7	1.22	7.4	1.58	1.93	1.52	Pass
5020	20	P	Dmax	2.6	0.82	4.1	1.40	1.15	1.52	**Fail**
Mean of Performance Group:				**1.9**	**1.23**	**8.1**	**1.50**	**1.84**	**1.90**	**Fail**
Performance Group No. PG-1 (Short Period, 20' Beam Span Configuration)										
2061	1	S	Dmax	4.0	1.96	16.1	1.33	2.61	1.52	Pass
1001	2	S	Dmax	3.5	2.06	14.0	1.33	2.74	1.52	Pass
--	3	S	Dmax	3.1*	--	--	--	2.63*	--	--
Mean of Performance Group:				**3.5***	**2.01**	**15.0**	**1.33**	**2.66***	**1.90**	**Pass**
Performance Group No. PG-3 (Long Period, 20' Beam Span Configuration)										
1008	4	S	Dmax	2.7	1.78	11.3	1.41	2.51	1.52	Pass
1012	8	S	Dmax	2.3	1.63	7.5	1.58	2.58	1.52	Pass
5014	12	S	Dmax	2.8	1.59	8.6	1.61	2.56	1.52	Pass
5021	20	S	Dmax	3.5	1.25	4.4	1.42	1.78	1.52	Pass
Mean of Performance Group:				**2.8**	**1.56**	**8.0**	**1.51**	**2.36**	**1.90**	**Pass**
Selected Archetypes - Performance Group Nos. PG-4 and PG-8 (20' Bay Width)										
6011	8	P	Dmin	1.8	2.12	3.0	1.21	2.56	1.52	Pass
6013	12	P	Dmin	1.8	2.00	3.7	1.24	2.47	1.52	Pass
6020	20	P	Dmin	1.8	1.73	2.8	1.20	2.08	1.52	Pass
6021	20	S	Dmin	3.4	3.70	3.3	1.22	4.51	1.52	Pass
Selected Archetypes - Performance Group Nos. PG-10 and PG-14 (30' Bay Width)										
1009	4	P-30	Dmax	1.6	1.98	13.4	1.41	2.79	1.52	Pass
1010	4	S-30	Dmax	3.3	2.50	13.2	1.41	3.53	1.52	Pass

* For completeness, RC SMF example assumes values of static overstrength and *ACMR* of missing 3-story archetypes (based on the average of respective 2-and 4-story values).

When focusing on the four controlling Performance Groups, this table shows that the majority of the archetype buildings have acceptable ACMR, but a disturbing trend becomes evident. For buildings taller than four-stories, the ACMR decreases substantially with increased building height. This causes the 20-story perimeter frame to have an unacceptable ACMR and causes the average ACMR of the perimeter frame PG to also be unacceptable.

As a result, the "newly proposed" RC SMF system does not attain the collapse performance required by the FEMA P695 methodology and could not be added as a "new system" in the building code provisions. To address the problem of the ACMR decreasing for taller buildings, one alternative would be to limit the proposed system to a maximum height of 12-stories (or 160'). Even with such a height limit, the average ACMR for the first Performance Group would not meet the required value of 1.90, so additional design requirement adjustments would be needed to improve the overall performance of the perimeter frame buildings.

This example assessment assumes that a 12-story height limit is not desirable, so the next section discusses one possible approach to modify the design requirements, so the "newly proposed" RC SMF system (up to 20-stories) will have acceptable collapse performance and be recommended for inclusion into the provisions.

Iteration: Alteration of Design Requirements to Meet Collapse Performance Goals

The last section concluded that the RC SMF system did not meet the performance criteria, at least not with the initially proposed set of design requirements. For the initial assessment presented in the previous sections, the ASCE 7-05 and ACI 318-05 design requirements were used along with R = 8, $C_d = 5.5$, an inter-story drift limit of 2%, and the minimum design base shear requirement from ASCE 7-05. These requirements must now be modified in such a way that the RC SMF collapse performance will improve and meet the performance criteria.

When deciding how to change the design requirements, the specific performance problems should be addressed to the extent possible. Table 3 showed that the adjusted collapse margin ratio (ACMR) decreased (higher collapse risk) with increasing height. This issue could be addressed in various ways. For example, height limits could be established (mentioned previously), more restrictive drift limits required, or, possibly, more conservative beam-column strength ratios developed for taller buildings. In this example, the minimum design base shear is increased in the effort to solve this problem. Specifically, the ASCE 7-05 minimum base shear requirement (ASCE 2005, equation 12.8-5) is replaced by the ASCE 7-02 requirement (ASCE 2002, equation 9.5.5.2.1-3, which is $C_s \geq 0.044 S_{DS} I$).

The building designs were revised to include this new minimum base shear requirement, and the collapse assessments were completed with the revised designs. This minimum base shear requirement changes the design base shear substantially for the taller buildings, with the largest changes being for the 20-story building in SDC D_{max} (design base shear coefficient from 0.022 to 0.044). Table 4 shows the updated collapse performance results. This table shows that each archetype building meets the performance requirement of ACMR $\geq$ 1.52 (i.e. 20% conditional collapse probability) and the average (ACMR) $\geq$ 1.90 for each Performance Group (i.e. 10% conditional collapse probability). This shows that after modifying the minimum design base shear requirement, the "newly proposed" RC SMF system attains the required collapse performance and could be added as a "new system" in the building code provisions.

Table 4. Summary of Final Collapse Margins and Comparison to Acceptance Criteria, for Buildings Redesigned with an Updated Minimum Base Shear Requirement

Arch. Design ID No.	Design Configuration			Computed Overstrength and Collapse Margin Parameters					Acceptance Check	
	No. of Stories	Framing (Gravity Loads)	SDC	Static Ω	*CMR*	μ_T	*SSF*	*ACMR*	Accept. *ACMR*	Pass/ Fail
Performance Group No. PG-5 (Short Period, 20' Bay Width Configuration)										
2069	1	P	Dmax	1.6	1.18	14.0	1.33	1.57	1.52	Pass
2064	2	P	Dmax	1.8	1.50	19.6	1.33	2.00	1.52	Pass
--	3	P	Dmax	1.7*	--	--	--	2.13*	--	--
Mean of Performance Group:				**1.7***	**1.34**	**16.8**	**1.33**	**1.90***	**1.90**	**Pass**
Performance Group No. PG-6 (Long Period, 20' Bay Width Configuration)										
1003	4	P	Dmax	1.6	1.61	10.9	1.41	2.27	1.52	Pass
1011	8	P	Dmax	1.6	1.25	9.8	1.61	2.01	1.52	Pass
1013	*12*	*P*	Dmax	*1.7*	*1.45*	*11.4*	*1.61*	*2.33*	*1.52*	*Pass*
1020	*20*	*P*	Dmax	*1.6*	*1.66*	*5.6*	*1.49*	*2.47*	*1.52*	*Pass*
Mean of Performance Group:				**1.6**	**1.49**	**9.4**	**1.53**	**2.27**	**1.90**	**Pass**
Performance Group No. PG-1 (Short Period, 20' Bay Width Configuration)										
2061	1	S	Dmax	4.0	1.96	16.1	1.33	2.61	1.52	Pass
1001	2	S	Dmax	3.5	2.06	14.0	1.33	2.74	1.52	Pass
--	3	S	Dmax	3.1*	--	--	--	2.63*	--	--
Mean of Performance Group:				**3.5***	**2.01**	**15.0**	**1.33**	**2.66***	**1.90**	**Pass**
Performance Group No. PG-3 (Long Period, 20' Bay Width Configuration)										
1008	4	S	Dmax	2.7	1.78	11.3	1.41	2.51	1.52	Pass
1012	8	S	Dmax	2.3	1.63	7.5	1.58	2.58	1.52	Pass
1014	*12*	*S*	Dmax	*2.1*	*1.59*	*7.7*	*1.60*	*2.54*	*1.52*	*Pass*
1021	*20*	*S*	Dmax	*2.0*	*1.98*	*5.7*	*1.50*	*2.96*	*1.52*	*Pass*
Mean of Performance Group:				**2.3**	**1.75**	**8.1**	**1.52**	**2.65**	**1.90**	**Pass**
Selected Archetypes - Performance Group Nos. PG-4 and PG-8 (20' Beam Span)										
4011	*8*	*P*	Dmin	*1.8*	*1.93*	*3.6*	*1.23*	*2.38*	*1.52*	*Pass*
4013	*12*	*P*	Dmin	*1.8*	*2.29*	*4.3*	*1.26*	*2.89*	*1.52*	*Pass*
4020	*20*	*P*	Dmin	*1.8*	*2.36*	*3.9*	*1.24*	*2.94*	*1.52*	*Pass*
4021	*20*	*S*	Dmin	*2.8*	*3.87*	*3.8*	*1.24*	*4.81*	*1.52*	*Pass*
Selected Archetypes - Performance Group Nos. PG-10 and PG-14 (30' Beam Span)										
1009	4	P-30	Dmax	1.6	1.98	13.4	1.41	2.79	1.52	Pass
1010	4	S-30	Dmax	3.3	2.50	13.2	1.41	3.53	1.52	Pass

* For completeness, RC SMF example assumes values of static overstrength and *ACMR* of missing 3-story archetypes (based on the average of respective 2-and 4-story values).

Calculation of Ω_0 using Set of Archetype Designs

At this point, the Ω_0 value can be established for use in the proposed design provisions, as specified by the FEMA P695 methodology. For this example, the upper-bound value of $\Omega_o = 3.0$ is warranted, due to the average Ω value of 3.5 for performance group PG-1.

Summary, Conclusions, and Observations

This paper used the RC SMF system and, for purposes of illustration, considered it to be a new system being proposed for inclusion in the building code provisions. This RC SMF example illustrated how the FEMA P695 methodology can be applied iteratively to develop system design provisions that result in acceptable collapse safety of a newly proposed structural system.

In additional to this RC SMF example providing an illustration of how to apply this assessment methodology, this example also demonstrates that this methodology is calibrated reasonably by showing that the recent building code design provisions lead to acceptable collapse safety. Acceptable collapse safety can be accomplished through use of the ACI 318-05 and ASCE 7-05 provisions, with an important modification of instead using the minimum base shear requirement of the ASCE 7-02 provisions (equation 9.5.5.2.1-3). This example also shows that this methodology is useful for testing possible changes to design requirements and informing building code decisions.

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