Example Evaluation of the ATC-63 Methodology for Reinforced Concrete Special Moment Frame Buildings

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PURPOSE AND OBJECTIVES

The Applied Technology Council, under the direction of the Federal Emergency Management Agency, has recently completed the ATC-63 document which contains a codified assessment methodology for quantifying structural collapse safety under seismic loading. This methodology is described in related companion papers [Kircher and Heintz 2008, Deierlein and Kircher 2008].

This paper illustrates how the 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 showing that it concludes that code-conforming RC SMF buildings, designed by recent code provisions, pass the method 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.

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. This example uses these guidelines and 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. Specifically, this relies on the Pacific Earthquake Engineering Research Center's Structural Performance Database that was developed by Berry, Parrish, and Eberhard [PEER 2006, Berry et al. 2004]. To develop element models, the data are utilized from cyclic and monotonic tests of 255 rectangular columns failing in flexure and flexure-shear [Haselton and Deierlein 2007, chapter 4].

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 that is considered an appropriate 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 ATC-63 methodology.

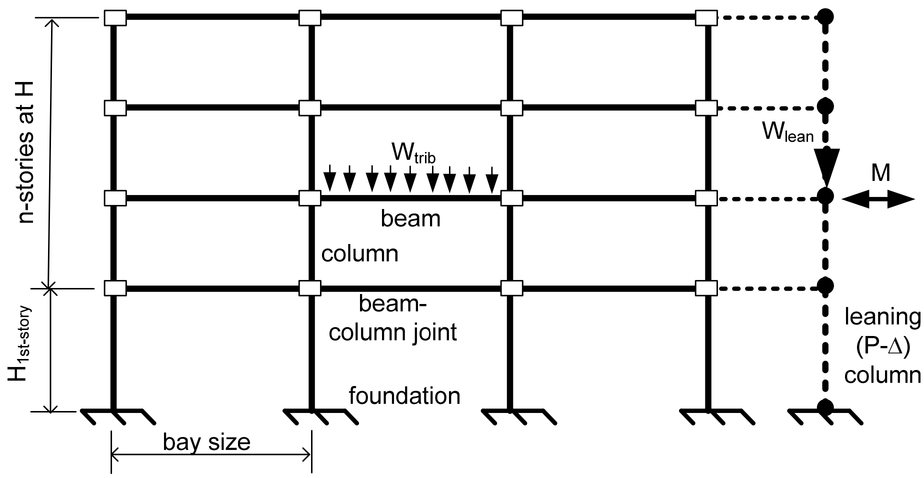

FIGURE 1 - ARCHETYPE ANALYSIS MODEL FOR MOMENT FRAME BUILDINGS

Using the above archetype model configuration, a set of structural archetype designs was developed to represent the archetype design space. Following the requirements of ATC-63, this set of designs was organized into four Performance Groups that represent the range of design ground motion intensities and the range of gravity loading conditions. Space and perimeter frame designs are used to represent high and low gravity loading, respectively.

- Maximum of highest SDC (SDC D_{max}), high gravity loading
- Maximum of highest SDC (SDC D_{max}), low gravity loading
- Minimum of highest SDC (SDC D_{min}), high gravity loading
- Minimum of highest SDC (SDC D_{min}), low gravity loading

Within each Performance Group, buildings are designed within a range of expected bay spacings (20' span and 30' span), and a range building heights (six heights between 1-story and 20-stories, as expected for RC frames buildings with no walls). To represent these four Performance Groups and ranges of design parameters, 48 archetypes could have been used to evaluate the RC SMF system (four seismic/gravity combinations, two bay spacings, and six heights). Instead of designing and assessing all 48 buildings, 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, use of 18 archetypes 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, bay spacings, etc.).

Table 1 shows the properties for each of these designs. The high- and low-seismic demands are represented by the minimum and maximum demands possible in Seismic Design Category (SDC) D. The archetypes are designed for a soil site (Site Class D) conditions and design lateral loads of $S_s = 1.5g$ and $S_1 = 0.6g$ for SDC D_{max} and $S_s = 0.55g$ and $S_1 = 0.13g$ for SDC D_{min}[1].

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.)	V/W (g)	
Maximum Seismic (D_{max}) and Low Gravity (Perimeter Frame) Designs, 20' Bay Width							
2069	1	P	D_{max}	8	0.26	0.125	1.50
2064	2	P	D_{max}	8	0.45	0.125	1.50
1003	4	P	D_{max}	8	0.81	0.092	1.11
1011	8	P	D_{max}	8	1.49	0.050	0.60
5013	12	P	D_{max}	8	2.13	0.035	0.42
5020	20	P	D_{max}	8	3.36	0.022	0.27
Maximum Seismic (D_{max}) and High Gravity (Space Frame) Designs, 20' Bay Width							
2061	1	S	D_{max}	8	0.26	0.125	1.50
1001	2	S	D_{max}	8	0.45	0.125	1.50
1008	4	S	D_{max}	8	0.81	0.092	1.11
1012	8	S	D_{max}	8	1.49	0.050	0.60
5014	12	S	D_{max}	8	2.13	0.035	0.42
5021	20	S	D_{max}	8	3.36	0.022	0.27
Minimum Seismic (SDC D_{min}) Designs, 20' Bay Width							
6011	8	P	D_{min}	8	1.49	0.013	0.16
6013	12	P	D_{min}	8	2.13	0.010	0.11
6020	20	P	D_{min}	8	3.36	0.010	0.07
6021	20	S	D_{min}	8	3.36	0.010	0.07
30-Foot Bay Width Designs (SDC D_{max})							
1009	4	P	D_{max}	8	0.81	0.092	1.11
1010	4	S	D_{max}	8	0.81	0.092	1.11

TABLE 1 - ARCHETYPE STRUCTURAL DESIGN PROPERTIES

DEVELOPMENT OF NONLINEAR STRUCTURAL ARCHETYPE MODELS

The core of this 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 [ATC 2007, Appendix E; Haselton and Deierlein 2007, chapter 4] provides additional detail regarding nonlinear structural modeling of RC SMF buildings. The system-level modeling uses the three-bay multi-story frame configuration shown in Figure 1.

[1] This class of buildings was designed for $S_s = 0.38g$ and $S_1 = 0.1g$, which differs slightly from the $S_s = 0.55g$ 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.

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. [Haselton and Deierlein, 2007]. An important aspect of this model is the "capping point," where monotonic strength loss begins, and the post-capping negative stiffnessm 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.

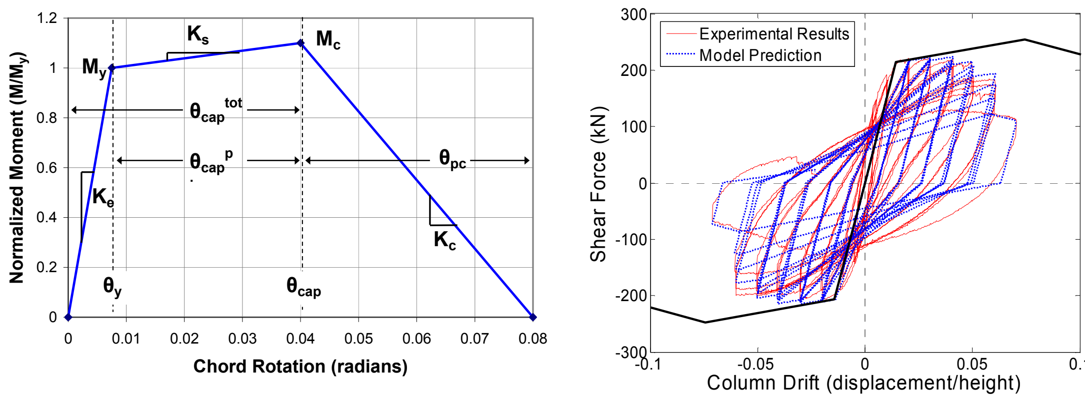

FIGURE 2 - ILLUSTRATION OF EXPERIMENTAL AND CALIBRATED ELEMENT RESPONSE [SAATCIOLGU AND GRIRA, 1999, SPECIMEN BG-6] [AFTER HASELTON ET AL. 2007]. THE SOLID BLACK LINE SHOWS THE CALIBRATED MONOTONIC BACKBONE.

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). A rating of "A-Superior" by no means indicates that this modeling approach is perfect, but the rating does recognize 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 2006], 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 element model (called "Clough" and documented in [Altoontash 2002]), 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, standard monotonic static pushover analysis is used, with the lateral load pattern prescribed in ASCE7-05 [ASCE 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 ATC-63 methodology. Note that the ATC-63 methodology does not require a full IDA, but rather only a simplified version, which has the goal of quantifying the median collapse capacity of each archetype model (i.e. not needing to estimate the record-to-record variability).

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 ATC-63.

Archetype Design ID Number	Design Configuration			Pushover and IDA Results			
	No. of Stories	Framing / Gravity Loads	Seismic SDC	Static Ω_O	S_{MT} [T] (g)	S_{CT} [T] (g)	CMR
Maximum Seismic (D_{max}) and Low Gravity (Perimeter Frame) Designs, 20' Bay Width							
2069	1	P	D_{max}	1.6	1.50	1.77	1.18
2064	2	P	D_{max}	1.8	1.50	2.25	1.50
1003	4	P	D_{max}	1.6	1.11	1.79	1.61
1011	8	P	D_{max}	1.6	0.60	0.76	1.25
5013	12	P	D_{max}	1.7	0.42	0.51	1.22
5020	20	P	D_{max}	2.6	0.27	0.22	0.82
Mean:	--	--	--	1.8	--	--	1.26
Maximum Seismic (D_{max}) and High Gravity (Space Frame) Designs, 20' Bay Width							
2061	1	S	D_{max}	4.0	1.50	2.94	1.96
1001	2	S	D_{max}	3.5	1.50	3.09	2.06
1008	4	S	D_{max}	2.7	1.11	1.97	1.78
1012	8	S	D_{max}	2.3	0.60	0.98	1.63
5014	12	S	D_{max}	2.8	0.42	0.67	1.59
5021	20	S	D_{max}	3.5	0.27	0.34	1.25
Mean:	--	--	--	3.1	--	--	1.71
Comparison of Results – SDC D_{max} & D_{min} Seismic Design Conditions, 20' Bay Width							
1011	8	P	D_{max}	1.6	0.60	0.76	1.25
6011	8	P	D_{min}	1.8	0.16	0.34	2.12
5013	12	P	D_{max}	1.7	0.38	0.46	1.22
6013	12	P	D_{min}	1.8	0.11	0.23	2.00
5020	20	P	D_{max}	2.6	0.24	0.20	0.82
6020	20	P	D_{min}	1.8	0.07	0.26	3.70
5021	20	S	D_{max}	3.5	0.26	0.33	1.25
6021	20	S	D_{min}	3.4	0.07	0.12	1.73
Comparison of Results - 20-Foot and 30-Foot Bay Width Designs (SDC D_{max})							
1003	4	P-20	D_{max}	1.6	1.11	1.79	1.61
1009	4	P-30	D_{max}	1.6	1.11	2.20	1.98
1008	4	S-20	D_{max}	2.7	1.11	1.97	1.78
1010	4	S-30	D_{max}	3.3	1.11	2.77	2.50

TABLE 2 - SUMMARY OF COLLAPSE RESULTS FOR ARCHETYPE DESIGNS

Static pushover analyses were completed and the IDA method was applied to each of the 18 archetype designs; Table 2 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 is 1.26 for perimeter frames and 1.71 for space frames, but these values have not yet been adjusted for the beneficial effects of spectral shape. The results in Table 2 verify that the buildings designed in low-seismic regions, and those with 30' bay width, both 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 12 more critical archetype buildings.

EVALUATION OF ADJUSTED COLLAPSE MARGIN RATIO AND ACCEPTANCE CRITERIA

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; Haselton and Baker, 2006]. To remedy this, the 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 1.38, and the values range from 1.0 to 1.65 for SDC D. The adjusted collapse margin ratio (ACMR) is computed as the multiple of the SSF and the CMR (from Table 2). 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 ATC-63 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, ATC-63 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 0.60; this accounts for the variability between ground motion records of a given spectral intensity (assumed to be constant for all structural systems, $\beta_{RTR} = 0.40$), 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, the above values result in a requirement that the average ACMR be at least 2.16 for each Performance Group, and that the individual values of the ACMR be at least 1.66 for each archetype building.

Table 3 presents the final results and acceptance criteria for each of the 18 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 archetype is shown to either pass of fail the acceptance criteria. Average margin results are also shown for the two groups of six high-seismic designs with 20' bay spacing; these sets of buildings represent the critical design conditions of the two high-seismic Performance Groups.

The comparisons in the second half of the table show that the high-seismic designs control and have lower ACMR. Therefore, the low-seismic Performance Groups were not focused on in this safety assessment. Also, the 20' bay width designs have lower ACMR than the 30' bay width designs, and control the reaminder of the assessment.

When focusing on the critical subsets of the two controlling Performance Groups, namely perimeter- and space-frame buildings designed for SDC D_{max} and having 20' bay spacing, this table shows that the majority of the archetype buildings have acceptable ACMR, but a disturbing trend becomes evident. For space- and perimeter-frame buildings taller than four-stories, the ACMR decreases substantially with increased building height. This causes the 20-story perimeter frame building to have an unacceptable ACMR and causes the average ACMR of the perimeter frame buildings to also be unacceptable. In addition, the one-story perimeter frame building has an ACMR that is slightly below the acceptable value.

Archetype Design ID Number	Design Configuration			Computed Collapse Margin				Acceptance Check	
	No. Stories	Framing / Gravity Loads	Seismic SDC	CMR	μ_c	SSF	ACMR	Acceptable ACMR	Pass/Fail
Maximum Seismic (D_{max}) and Low Gravity (Perimeter Frame) Designs, 20' Bay Width									
2069	1	P	D_{max}	1.18	16.1	1.34	1.58	1.66	Near Pass
2064	2	P	D_{max}	1.50	19.5	1.34	2.01	1.66	Pass
1003	4	P	D_{max}	1.61	9.2	1.42	2.29	1.66	Pass
1011	8	P	D_{max}	1.25	7.9	1.62	2.02	1.66	Pass
5013	12	P	D_{max}	1.22	5.9	1.54	1.88	1.66	Pass
5020	20	P	D_{max}	0.82	3.3	1.40	1.14	1.66	**FAIL**
Mean/Acceptable:		--	--	--		--	1.82	2.16	**FAIL**
Maximum Seismic (D_{max}) and High Gravity (Space Frame) Designs, 20' Bay Width									
2061	1	S	D_{max}	1.96	16.1	1.34	2.62	1.66	Pass
1001	2	S	D_{max}	2.06	14.3	1.34	2.76	1.66	Pass
1008	4	S	D_{max}	1.78	9.6	1.42	2.53	1.66	Pass
1012	8	S	D_{max}	1.63	6.2	1.55	2.52	1.66	Pass
5014	12	S	D_{max}	1.59	6.9	1.58	2.51	1.66	Pass
5021	20	S	D_{max}	1.25	3.5	1.41	1.76	1.66	Pass
Mean/Acceptable:		--	--	--		--	2.45	2.16	Pass
Comparison of Results - SDC D_{max} & D_{min} Seismic Design Conditions, 20' Bay Width									
1011	8	P	D_{max}	1.25	7.9	1.62	2.02	1.66	Pass
6011	8	P	D_{min}	2.12	2.3	1.20	2.55	1.66	Pass
5013	12	P	D_{max}	1.22	5.9	1.54	1.88	1.66	Pass
6013	12	P	D_{min}	2.00	2.5	1.22	2.43	1.66	Pass
5020	20	P	D_{max}	0.82	3.3	1.40	1.14	1.66	**FAIL**
6020	20	P	D_{min}	3.70	1.8	1.17	4.32	1.66	Pass
5021	20	S	D_{max}	1.25	3.5	1.41	1.76	1.66	Pass
6021	20	S	D_{min}	1.73	2.4	1.21	2.09	1.66	Pass
Comparison of Results - 20-Foot and 30-Foot Bay Width Designs (SDC D_{max})									
1003	4	P-20	D_{max}	1.61	9.2	1.42	2.29	1.66	Pass
1009	4	P-30	D_{max}	1.98	11.4	1.42	2.81	1.66	Pass
1008	4	S-20	D_{max}	1.78	9.6	1.42	2.53	1.66	Pass
1010	4	S-30	D_{max}	2.5	11.0	1.42	3.55	1.66	Pass

TABLE 3 - SUMMARY OF FINAL COLLAPSE MARGINS AND COMPARISON TO ACCEPTANCE CRITERIA

At this point, the "newly proposed" RC SMF system does not attain the collapse performance required by the ATC-63 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 2.16, 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. For the initial RC SMF assessment, Table 3 showed that the one-story perimeter frame building does not meet the ACMR requirement of 1.66; this deficiency would need to be addressed for this RC SMF system to attain the collapse performance required by this document. This could be addressed in many ways, such a general decrease in the R-fator, a more targeted added strength requirement for short-period structures, or an added requirement for perimeter frame buildings (which tend to have low Ω). For brevity, this example does not address this issue[2].

Error! Reference source not found. also showed a more important and disturbing trend, 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. Table 4 shows the updated collapse performance results, with the bold italic lines highlighting the designs that were affected by the change to the minimum base shear requirement. To give an idea of how this minimum base shear requirement changed the design base shear for the taller buildings, the largest changes was the design base shear coefficient (V/W) increasing from 0.022 to 0.044 for the 20-story building in SDC D_{max}. This table shows that each archetype building meets the performance requirement of ACMR ≥ 1.66 (i.e. 20% conditional collapse probability) and the average(ACMR) ≥ 2.16 for each Performance Group (i.e. 10% conditional collapse probability)[2]. 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[2].

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[2] The performance of the one-story building causes the ACMR values to be slightly below the required values for the perimeter frame SDC D_{max} Performance Group. For simplicity, this is not addressed in this example, and it is assumed that the SDC D_{max} perimeter frame Performance Group meets the performance requirements. In the context of a complete performance assessment, this deficiency would need to be addressed.

Arch. Design ID Number	Design Configuration				Computed Collapse Margin				Acceptance Check	
	No. of Stories	Framing / Gravity Loads	Seismic SDC	Static Ω_O	CMR	μ_c	SSF	ACMR	Acceptable ACMR	Pass/Fail
Maximum Seismic (D_{max}) and Low Gravity (Perimeter Frame) Designs, 20' Bay Width										
2069	1	P	D_{max}	1.6	1.18	16.1	1.34	1.58	1.66	Near Pass
2064	2	P	D_{max}	1.8	1.50	19.5	1.34	2.01	1.66	Pass
1003	4	P	D_{max}	1.6	1.61	9.2	1.42	2.29	1.66	Pass
1011	8	P	D_{max}	1.6	1.25	7.9	1.62	2.02	1.66	Pass
1013	***12***	***P***	D_{max}	***1.7***	***1.45***	***10.0***	***1.62***	***2.35***	***1.66***	***Pass***
1020	***20***	***P***	D_{max}	***1.6***	***1.66***	***7.2***	***1.59***	***2.64***	***1.66***	***Pass***
Mean/Acceptable:		--	--	***1.7***	--		--	***2.15***	***2.16***	***Near Pass***
Maximum Seismic (D_{max}) and High Gravity (Space Frame) Designs, 20' Bay Width										
2061	1	S	D_{max}	4.0	1.96	16.1	1.34	2.62	1.66	Pass
1001	2	S	D_{max}	3.5	2.06	14.3	1.34	2.76	1.66	Pass
1008	4	S	D_{max}	2.7	1.78	9.6	1.42	2.53	1.66	Pass
1012	8	S	D_{max}	2.3	1.63	6.2	1.55	2.52	1.66	Pass
1014	***12***	***S***	D_{max}	***2.1***	***1.59***	***5.8***	***1.53***	***2.44***	***1.66***	***Pass***
1021	***20***	***S***	D_{max}	***2.0***	***1.98***	***9.1***	***1.62***	***3.21***	***1.66***	***Pass***
Mean/Acceptable:		--	--	***2.8***	--		--	***2.68***	***2.16***	***Pass***
Comparison of Results - SDC D_{max} & D_{min} Seismic Design Conditions, 20' Bay Width										
1011	8	P	D_{max}	1.6	1.25	7.9	1.62	2.02	1.66	Pass
4011	***8***	***P***	D_{min}	***1.8***	***1.93***	***2.8***	1.23	***2.37***	***1.66***	***Pass***
1013	***12***	***P***	D_{max}	***1.7***	***1.45***	***10.0***	1.62	***2.35***	***1.66***	***Pass***
4013	***12***	***P***	D_{min}	***1.8***	***2.29***	***3.4***	1.25	***2.87***	***1.66***	***Pass***
1020	***20***	***P***	D_{max}	***1.6***	***1.66***	***7.2***	1.59	***2.64***	***1.66***	***Pass***
4020	***20***	***P***	D_{min}	***1.8***	***2.36***	***3.0***	1.24	***2.92***	***1.66***	***Pass***
1021	***20***	***S***	D_{max}	***2.0***	***1.98***	***9.1***	1.62	***3.21***	***1.66***	***Pass***
4021	***20***	***S***	D_{min}	***2.8***	***3.87***	***3.0***	1.24	***4.80***	***1.66***	***Pass***
Comparison of Results - 20-Foot and 30-Foot Bay Width Designs (SDC D_{max})										
1003	4	P-20	D_{max}	1.6	1.61	9.2	1.42	2.29	1.66	Pass
1009	4	P-30	D_{max}	1.6	1.98	11.4	1.42	2.81	1.66	Pass
1008	4	S-20	D_{max}	2.7	1.78	9.6	1.42	2.53	1.66	Pass
1010	4	S-30	D_{max}	3.3	2.50	11.0	1.42	3.55	1.66	Pass

TABLE 4 - SUMMARY OF FINAL COLLAPSE MARGINS AND COMPARISON TO ACCEPTANCE CRITERIA, FOR BUILDINGS REDESIGNED WITH AN UPDATED MINIMUM BASE SHEAR REQUIREMENT

CALCULATION OF Ω_0 USING FINAL 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 ATC-63 methodology. For this example, the upper-bound value of $\Omega_o = 3.0$ is warranted, due to the average Ω value of 2.8 for maximum seismic space frame Performance Groups and the large values of 3.5 and 4.0 observed for some individual archetype buildings.

SUMMARY, CONCLUSION, 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 ATC-63 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. Specifically, this section showed that 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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Then entire ATC-63 project team contributed greatly to this publication [see detail in Kircher and Heintz 2008], and Brian Dean and Jason Chou (graduate research interns) completed many of the designs and much of the structural analysis on which this publication is based.

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