

## **Case Study on the Seismic Performance of Reinforced Concrete Intermediate Moment Frames using ACI Design Provisions**

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### **ABSTRACT**

In reinforced concrete design, seismic provisions outline three types of moment resisting frames for use in high, moderate, and low seismic regions. However, there has been no methodology for assessing the performance of these structures other than through observations from actual seismic events or physical testing. The Applied Technology Counsel (ATC) has developed the ATC-63 Methodology (now FEMA P695) for assessing the seismic performance of structures through mathematical simulation, which may be used to assess the performance of reinforced concrete frames. This study investigates the performance of reinforced-concrete intermediate moment frames (IMF) used in moderate seismic zones. In this study, a four-story and six-story IMF were designed; once using current American Concrete Institute (ACI) seismic provisions and again with the addition of a strong-column weak-beam ratio. The earthquake simulation and dynamic analysis software OpenSees was then used to model the frames and analyze their structural performance. The ATC-63 Methodology as outlined in *FEMA P695: Quantification of Building Seismic Performance Factors* was used to assess the performance, focusing namely on the IMF's median collapse capacity, in terms of spectral acceleration. Three major findings applicable to IMFs of the type and height modeled were produced from the study. First, the results indicate that the current IMF provisions of ACI-318 2008 do provide acceptable seismic performance. Second, the addition of a strong-column weak-beam ratio did not offer significant improvement over current provisions. Third, the study indicates a potential concern for shear failure occurring that the models would not detect. Further investigation is needed to address this concern. These results add insight into the design provisions for intermediate moment frames and contribute to the technical base for future criteria and study.

## SEISMIC DESIGN FOR REINFORCED CONCRETE STRUCTURES

Today, seismic provisions used by practicing engineers are in the American Society of Civil Engineers (ASCE) publication entitled *ASCE 7-05 Minimum Design Loads for Buildings and Other Structures* (ASCE 7-05). Both the 2006 and 2009 versions of the International Building Code (IBC 2006 and IBC 2009) refer to the seismic provisions of ASCE 7-05. These provisions include the required design earthquake forces and some general detailing requirements. The design earthquake forces are dependent on the type of structural system of a building, but ASCE 7-05 does not include a complete set of detailing provisions for the various structural systems. Specific strength and detailing requirements pertaining to the seismic design of reinforced concrete structures are included in the American Concrete Institute (ACI) publication *ACI-318 Building Code Requirements for Structural Concrete and Commentary*. ACI-318 provides “minimum requirements for design and construction of structural concrete elements” (ACI-318 2008, 9) including seismic provisions for the strength and detailing requirements of reinforced concrete structures. This study focuses on reinforced concrete (RC) intermediate moment resisting frames.

If a moment frame is subjected to a major earthquake, such as a design level earthquake, localized regions of reinforcing steel yielding are expected to form. These regions within a structural element where plastification of the cross section or ductile behavior occurs during loading are referred to as plastic hinges. As the reinforcing steel yields in tension and compression at these hinges, hysteretic damping occurs. It is this damping that absorbs seismic energy, provided that the component is sufficiently ductile to withstand large deformations.

The beneficial effect of ductility and inelastic behavior is accounted for in the calculation of seismic design loads through the response modification factor,  $R$ , which is in the denominator of the equation for determining seismic load in ASCE 7-05 (ASCE, 2005). A higher  $R$  value reduces the level of seismic design load, since it is assumed that the structure has adequate ductility to undergo nonlinear behavior and dissipate energy through hysteretic damping. The goal of seismic detailing and strength requirements found in the ACI-318 code is to provide enough ductility to create plastic hinges within the structural elements of a frame, preferably beginning in beams rather than columns. Other provisions relate to proper design of transverse reinforcement to resist shear, so elements do not experience premature shear failure either before plastic hinges develop or within plastic hinges due to degradation of shear strength. Transverse reinforcement also improves confinement and ductility. Detailing requirements are contingent on the category of moment resisting frame being designed by the engineer.

Current seismic provisions specify three categories of RC moment frames. Special moment frames (SMFs) are required in high seismic regions such as much of the West Coast (Seismic Design Category (SDC) D and above). SMF structures are designed with a response modification factor  $R$  equal to 8 because they have substantial ductility due to capacity design provisions, strong column-weak beam requirements and seismically detailed reinforcement. Ordinary moment frames (OMFs) are built in low seismic zones (SDC B and below), and are subject to less stringent design and detailing requirements. OMFs are less ductile than the SMF and are designed with a lower  $R$  factor equal to 3. Intermediate moment frames (IMFs)

are used in moderate seismic zones such as parts of the southern United States and the Northeast and have ductility requirements that fall between those of SMFs and OMFs. While criteria for SMFs have been much of the research focus, criteria for IMFs were developed based on past observations and the judgment of the ACI-318 committee. IMFs are designed with  $R$  equal to 5.

## **FEMA P695 METHODOLOGY**

Before the advent of computer simulation, assessing the performance of a structural system was limited to conducting dynamic analysis calculations manually during design and to observing the response of buildings during actual seismic events. Now, with current modeling and simulation software, along with a database of recorded ground motion histories, researchers and engineers are able to simulate seismic events and test building performance. The ATC-63 (FEMA P695) Project, funded by FEMA, set out to develop a methodology for assessing the performance of structural engineering systems, and ultimately the underlying criteria and code provisions for those systems, through the use of simulated seismic events.

The main objectives of the project were to establish an analysis procedure through which the buildings are modeled and analyzed, develop the benchmark standards to which results are compared and assessed, determine how uncertainty is included, and define how specific analysis results are scaled to compare with generalized benchmarks (Deierlein et al, 2007). Assessment criteria are based on ensuring a low probability of earthquake-induced collapse under a large earthquake.

The methodology for assessing seismic performance is composed of five major stages or steps.

- (1) A building system is identified, such as a RC moment frame, and structural behavior characteristics are determined for the system's structural components (i.e. concrete beams, columns, joints, etc.).
- (2) Design provisions are then identified for the building system. For a new system, the proponents would be required to develop new design provisions. For an existing system, such as a RC moment frame, these provisions are already established in ASCE 7-05, and ACI-318.
- (3) An archetype structure (or set of archetype structures) is then designed to capture the defining features and characteristics of the building system that would impact collapse performance (Deierlein et al., 2007). In this study, the archetype structure was devised to follow typical engineering practice for IMF design.
- (4) A model of the archetype structure is created and nonlinear dynamic analyses are conducted to simulate the structure's response during a seismic event. Parameters of interest in the analyses are the ground motion intensity level necessary to cause collapse of the structure and the failure modes of the structure.
- (5) The performance of the structure is then compared with FEMA P695 acceptance criteria and an assessment is made on whether the response is within acceptable limits. Acceptance criteria are based on achieving a low probability of earthquake-induced collapse under the maximum considered

earthquake (MCE)-level ground motion. The system and its underlying design criteria are then either approved or the structure is redefined.

A 2006 study by Liel, Haselton, and Deierlein (2006) applied the draft FEMA P695 Methodology in the modeling of RC moment-resisting frames, including a 2003 special, intermediate and ordinary moment frames. At the MCE spectral acceleration, which differed for each building depending on the design SDC, the SMF, IMF and OMF had collapse probabilities of 17%, 20%, and 12% respectively. The study found that 40% of collapse modes exhibited by the 2003 SMF were caused by a single-story mechanism at the third story and 69% of all failures were caused by single-story mechanisms.

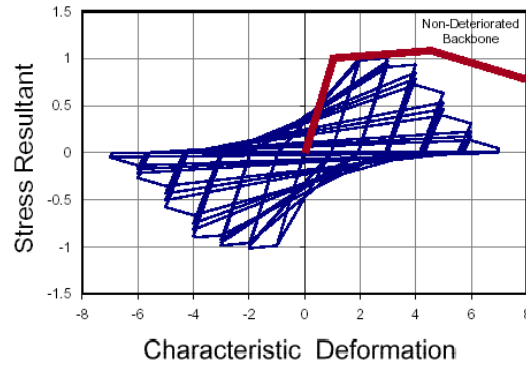
A 2007 study by Deierlein, Liel, Haselton, and Kircher redesigned the SMF using 2005 ACI provisions and compared it against the FEMA P695 benchmarks in the draft methodology (Deierlein et al, 2007). All but two high-rise SMF models passed the assessment indicating a large difference in collapse probability between frames of tall buildings designed with the ASCE 2002 minimum base shear requirement and the lower ASCE 2005 base shear value. The ASCE 7 committee ultimately issued an addendum to reinstitute the 2002 version of the provision based on these results (Deierlein et al, 2007).

The purpose of this study was to expand on previous work and focus on the provisions for RC IMFs. Frames were designed using IMF seismic provisions outlined in the 2008 edition of ACI-318. The performance of the IMF models was then analyzed using the FEMA P695 Methodology.

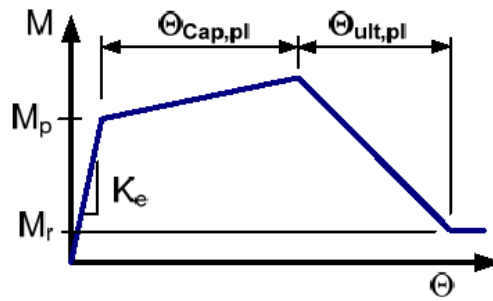
## **OPENSEES**

To simulate the seismic performance of a building system using the FEMA P695 Methodology, a software framework was required to conduct a nonlinear analysis described later. The Open System for Earthquake Engineering Simulation (OpenSees) was used for simulation in the IMF study. Developed at the University of California, Berkeley, OpenSees is an “object-oriented framework for finite element analysis” (Mazzoni et al. 2006) and the software allows researchers to build onto OpenSees’s framework to adapt it to their studies on structural seismic response. The model used in the IMF study is a frame composed of beam-column elements and joint elements. The beam-column elements contain lumped plasticity parameters where plastic hinges and ductile behavior is envisioned to occur at the members’ ends. The elements consider stiffness and strength deterioration overtime by using the hysteretic modeling parameters developed in a 2005 study by Ibarra, Medina, and Krawinkler. Model behavior is illustrated for cyclic and monotonic response in Figures 1 and 2. Bar splices were assumed to not occur at the ends of column members.

The parameters used for OpenSees beam-column elements were the moment capacity at yield  $M_y$ , the rotational capacity at yield  $\theta_y$ , the pre-capping slope  $K_s$  which equals  $M_c/M_{y\theta}$ , the plastic rotation capacity  $\theta_{cap}$ , the ultimate rotational capacity  $\theta_{ult}$ , and the post-capping slope  $K_c$  (Liel et al, 2006). These parameters capture the progressive deterioration of the ductile components and ultimately determine when collapse occurs in the structural model. Parameters used to model IMF beams and columns were based on predictive equations from Haselton et al. (2008).



**Figure 1: Hysteresis plot representing moment-rotation behavior of plastic hinges (Liel et al, 2006)**



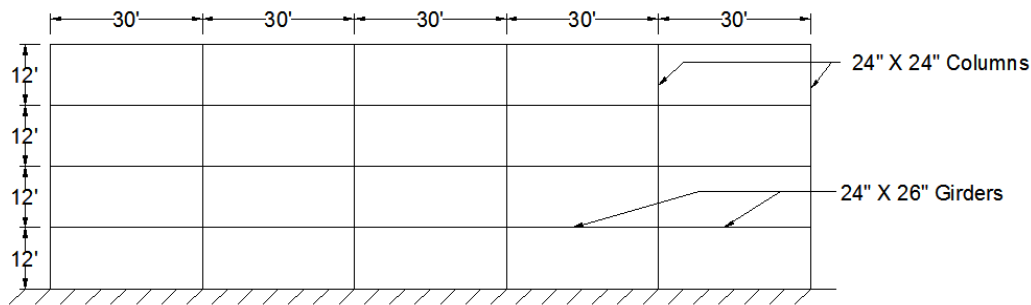
**Figure 2: Monotonic Backbone Curve (Liel et al, 2006)**

The finite size beam-column joint elements also capture progressive deterioration through the use of inelastic springs and elastic springs. Five, concentrated, inelastic springs model “joint panel distortion and bond slip at each face of the joint” (Liel et al, 2006). Additionally, elastic semi-rigid foundation springs are used to represent soil and foundation flexibility at supports.

## CASE STUDY IMF DESIGN

In order to analyze the performance of an IMF during simulated seismic events, a representative archetype frame needed to be designed that would be indicative of a building that might actually be designed in a moderate seismic zone. Table 12.2-1 of ASCE 7-05 specifies that use of an IMF is permitted for seismic design categories (SDC) B and C but not SDC D, E and F. Therefore, to illustrate the upper bound use of an IMF on the boundary of SDC C and D, design spectral response accelerations were chosen as  $S_{DS}=0.49g$  and  $S_{D1}=0.19g$  according to the FEMA P695 methodology. The region around Nashville, Tennessee is a moderate seismic zone with  $S_{DS}$  and  $S_{D1}$  design values similar to those used in this study.

The archetype IMF was composed of a beam-column configuration. Other possible forms of IMFs include flat-slab and flat-plate moment frame designs but were not considered in this study. Figure 3 illustrates the longitudinal elevation of the case study frame with five 30 foot (9.1 m) wide bays and a consistent floor height of 12 feet (3.7 m).



**Figure 3: Four-Story Design Building Elevation**

Developed through consultation with Mr. Thomas C. Schaeffer, P.E., a structural engineer with the Structural Design Group based in Nashville, Tennessee, the four-story IMF is indicative of a design of a typical hospital patient wing. Girder dimensions were 24" by 26" (61cm X 66cm) and column dimensions were 24" by 24" (61cm X 61cm). Concrete was assumed to have a self weight of 150 pcf (2403kg/m<sup>3</sup>) and compression strength of 5,000 psi (34.47MPa). A 5.0 inch (13 cm) concrete slab was also assumed. Designed as a two-dimensional frame, the frame was considered to act only along its longitudinal direction. Support conditions at the column bases were designed to consider the existence of a basement with a column base rotational spring stiffness of 12,800,000 kip-in/radian (1.45(10<sup>8</sup>) kN-cm/rad). Even though a hospital is considered to be an essential facility with an importance factor of 1.5, an importance factor of 1 for non-essential facilities was assumed in order for the study's results to be applicable to a larger building set. As specified by the 2006 IBC, a 40psf (1.8kPa) live load and an 80psf (3.8kPa) live load were used for patient rooms and corridors respectively. Based on typical practice, the maximum and minimum steel reinforcement ratios were 2.5% and 1% respectively for the design of columns, while a maximum reinforcement ratio of 1.25% was used for the girder design. The building period T for the four story frame was 0.79s.

Microsoft Excel design spreadsheets and manual calculations were used to design the member sizes and reinforcement for the four-story IMF. Flexural and shear reinforcement was established for the floor slabs, girders and columns based on IMF requirements in Chapter 21 of the 2008 ACI-318. The flexural reinforcement was made to be symmetric both for individual members and the entire frame. Most column reinforcement was governed by the minimum reinforcement ratio requirement of 1%. Girders had top steel reinforcement ratios of up to 1.25%. The confinement of beam-column joints was checked and shear reinforcement was designed accordingly. Excel Visual Basic tools developed by Haselton and Deierlein (2007) were then used to create the OpenSees model input files from the finalized IMF design.

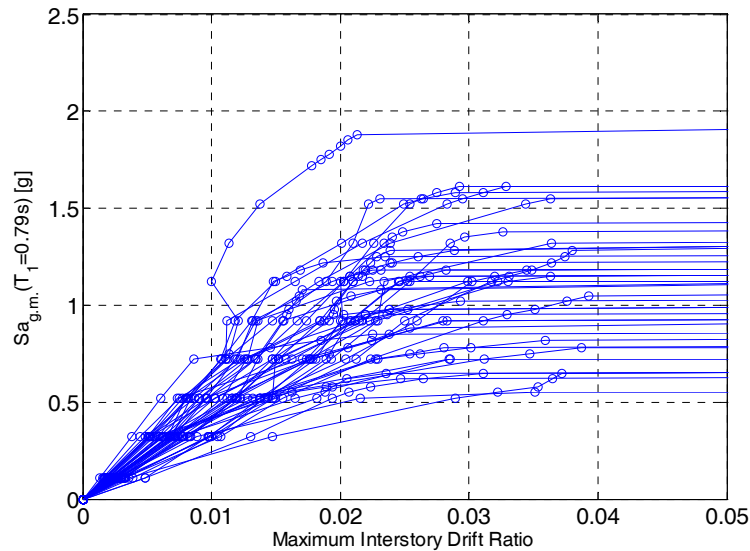
## NONLINEAR ANALYSIS

The model files developed by Excel were then input for use in the dynamic analysis software OpenSees. The earthquake simulation using the OpenSees software consisted of two main components: a static pushover analysis and a nonlinear dynamic analysis. The static pushover analysis was conducted by applying an increasing static lateral load to the frame until the inter-story drift became too large

and the model collapsed. With the aid of MATLAB, the OpenSees simulation plotted the frame's base shear versus the roof drift ratio. The maximum base shear, and the ultimate roof drift ratio,  $\delta_u$  taken at a value of  $0.8 V_{\max}$  were then determined for use in the dynamic analysis. Specifically, the ultimate drift is used to determine the amount of ductility in the IMF model or the period based ductility  $\mu_T$ . The value of  $\mu_T$  for the four-story IMF was 13.9. This value is defined as the ratio of the ultimate story roof drift displacement and the effect yield roof drift displacement in the FEMA P695 document. The period based ductility factor is need to compute the spectral shape factor discussed later as more ductile buildings are more sensitive to the impact of spectral shape.

For the dynamic analysis, five possible collapse scenarios are simulated within the OpenSees model. These scenarios involve various forms of side-sway collapse such as beam and column flexural hinging, flexural-shear failure, and joint-shear failure; all of which lead to a side-sway mechanism.

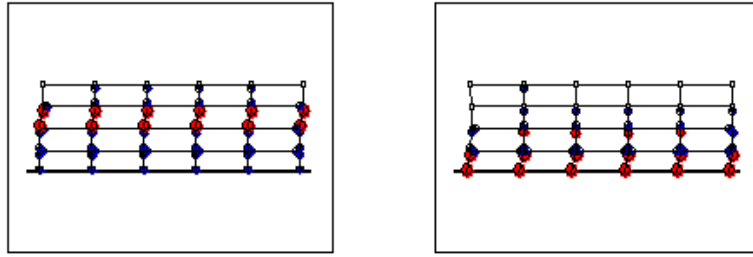
The ground motion acceleration spectra recorded from 44 large magnitude earthquakes were then used to conduct an incremental dynamic analysis (IDA) on the frame. This involved applying an incremental ground acceleration that follows the time history spectra of one of the 44 earthquakes from the record. The frame's maximum inter-story drift was then determined for that increment. Collapse occurs when inter-story drifts increase without bound, i.e. dynamic instability. At this point, the simulation is stopped and the IDA begins again for the next acceleration spectra record. Figure 4 below illustrates the IDA plot for the four-story IMF frame. Plotting ground acceleration  $S_a$  v. maximum inter-story drift, the graph shows the value of ground acceleration for each earthquake spectra at which collapse occurred (i.e. significant increase in drift). The plot illustrates that inter-story drifts begin to increase without bound (i.e. failure) after a drift ratio of 0.02 and no iteration is able to exceed a ratio of 0.04 before collapse occurs for the four-story IMF.



**Figure 4: Incremental Dynamic Analysis (IDA) Plot for Four-Story IMF**

The failure mode at the collapse acceleration in the analysis was also sketched for every ground motion spectra using MATLAB (Shown in Figure 5 for the four-story

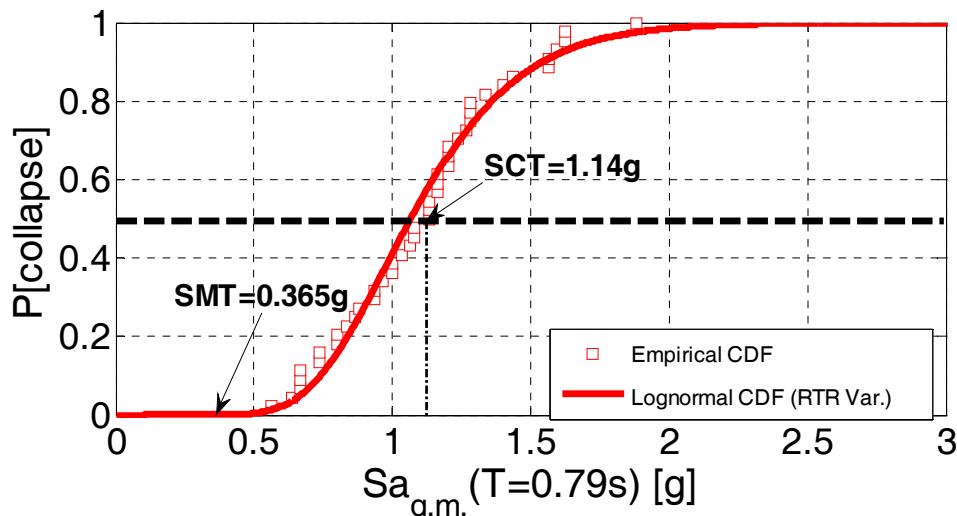
IMF). Failure modes may vary for different ground motions because of differences in ground motion frequency content and duration.



**Figure 5: Sample Failure Modes for a Four-Story IMF. In both cases illustrated, the building failed in a single-story mechanism.**

This allows for a pictorial representation of how the building will possibly fail during a specific seismic event. Based on these failure modes, 32.5% of the ground motions caused the formation of a single-story mechanism at the first story (Figure 5 right), 27.5% failed from a single-story mechanism at the second floor, and 15% from a single-story mechanism at the third floor (Figure 5 left). 7.5% of the ground motions induced a sideways mechanism not confined to one floor while 17.5% showed no clear side-sway collapse mechanism pictorially.

However, the main parameter of interest obtained from the IDA was the median collapse level acceleration,  $\hat{S}_{CT}$ . Figure 6 illustrates the cumulative distribution function plot for the four-story IMF analysis results. The median collapse level acceleration is the value at which 50% of the model ground motions of the IMF have collapsed. Restated, the  $\hat{S}_{CT}$  is the ground acceleration at which, for any seismic event, the specific building frame has a 50% chance of survival. For the four-story IMF frame, a  $\hat{S}_{CT}$  value of 1.14g was determined from the analysis.



**Figure 6: Cumulative Distribution Function (CDF) Plot for Four-Story IMF**

The value of  $\hat{S}_{CT}$  is specific to the individual building system being studied and therefore cannot be readily compared to the performance of other systems or to FEMA P695 benchmarks. Therefore, the median collapse level acceleration is



compared to the maximum considered earthquake (MCE) spectral ground acceleration  $S_{MT}$  for the period of the frame, which is used in design.

The ratio between  $\hat{S}_{CT}$  and  $S_{MT}$ , known as the Collapse Margin Ratio (CMR), compares a specific frame's actual simulation performance to the performance standard that was used for design. Since modern seismic provisions provide some factor of safety against collapse at the MCE, the value of CMR will be a value greater than one. For the four-story IMF model, the  $S_{MT}$  value at its period of 0.79 seconds was 0.365g and its CMR was then calculated as 3.11.

## PERFORMANCE ASSESSMENT

The performance assessment using the ATC-63 Methodology consisted of two main steps: calculating the adjusted collapse margin ratio (ACMR) and determining the acceptable benchmark to compare with the value obtained from simulations.

The CMR must be adjusted to the ACMR in order to account for the effect of rare ground motion acceleration spectra. This factor is needed because ground motions large enough to cause earthquake-induced collapse have a characteristic spectral shape, which may be less damaging (Haselton et al. 2009). This effect is adjusted by multiplying the CMR by a spectral shape factor (SSF) to obtain the ACMR. The SSF is determined from tabulated values within the FEMA P695 Methodology. It is based on the seismic design category, the period of the building  $T$  and the frame's period based ductility  $\mu_T$  determined from the pushover analysis. The SSF for the four-story IMF was determined as 1.198 and the ACMR value was calculated as 3.73.

The acceptable benchmark to which the ACMR is compared is based on the total uncertainty contained within the model. The FEMA P695 Methodology identifies four sources of uncertainty: test data uncertainty  $\beta_{TD}$ ; model uncertainty  $\beta_{MDL}$ ; record-to-record uncertainty  $\beta_{RTR}$ ; and design requirement uncertainty  $\beta_{DR}$  (FEMA P695, 2009). These parameters are quantified by FEMA P695 using a scale from superior ( $\beta=0.1$ ) to poor ( $\beta=0.5$ ). The total system collapse uncertainty can then either be calculated with these values or determined from tabulated values in FEMA P695. For the IMF modeling, the test data uncertainty was determined to be good ( $\beta_{TD}=0.2$ ), model quality was determined to be good ( $\beta_{MDL}=0.2$ ), and the design requirements were determined to be superior ( $\beta_{DR}=0.1$ ). A value of 0.500 was then obtained for the total system collapse uncertainty.

The FEMA P695 Methodology specifies that the average collapse probability should not be less than 20% for any one archetype model of a building system, for this case an IMF, or 10% for a building system class (FEMA P695, 2009). Therefore, the methodology tabulates values of the ACMR at collapse probabilities of 20% and 10% for values of total system uncertainty. The simulated value of ACMR must be higher than this benchmark (ACMR<sub>20%</sub> or 10%), indicating that the model collapse probability is lower than the benchmark. The IMF design considered here has an ACMR of 3.73 which is greater than the minimum acceptable value of the ACMR<sub>20%</sub>, 1.52. The ACMR is also greater than the ACMR<sub>10%</sub> value of 1.90. Therefore, based on the FEMA P695 Methodology, the four-story archetype IMF designed using ACI-318 criteria was deemed acceptable for seismic performance.

## PARAMETRIC STUDY

The parametric study sought to see how the seismic performance would be affected by changing two parameters: the height of the structure and the addition of a strong column-weak beam ratio.

First, a six-story IMF was designed using the ACI-318 2008 seismic provisions. The six-story frame design was based on the same plan, but column and beam dimensions were adjusted. Concrete columns were increased to 28 inches square (71cm) in order for the reinforcement ratio to remain below the maximum reinforcement ratio requirement of 2.5%. The building period  $T$  for the IMF was 1.14s and the value of  $\mu_T$  for the six story frame was 6.803. For this IMF, 45.5% of the ground motions caused a multi-story side-sway mechanism while 20.5% of the failures caused single-story mechanisms at either the 1<sup>st</sup>, 4<sup>th</sup>, or 5<sup>th</sup> stories. From the IDA, IMF collapse occurred between maximum inter-story drift ratios 0.03 and 0.09. Results of the FEMA P695 assessment are described below.

Next, the four-story IMF and the six-story IMF were redesigned to include a strong column-weak beam ratio of 1.2. This provision is already part of ACI-318 requirements for SMFs. Its intent is to force the failure of beams before column failure and prevent a single-story mechanism. The columns in the four-story frame were required to be increased to 28 inches square (71cm) in order to stay below the maximum reinforcement ratio of 2.5%. Meanwhile, the columns in the six-story frame were increased to 34 inches square (86cm) and the floor girders were increased to 26 inches (66cm) wide by 28 inches (71cm) deep. These measures were again done to stay under the reinforcement limit of 2.5% for columns and to also ensure that joints were confined.

The failure scenarios for the redesigned four-story IMF were governed by either a multistory side-sway mechanism or a single-story mechanism at the first story. Failure occurred between maximum inter-story drift ratios of 0.035 and 0.105. For the redesigned six-story IMF, a majority of failure scenarios involved a multistory side-sway mechanism and failure occurred between drift ratios of 0.035 and 0.0925.

The CMR and ACMR values for each of the four models along with the acceptable ACMR20% and ACMR10% values are shown in Table 1 below. The addition of a SCWB ratio caused an increase in  $\hat{S}_{CT}$  for both frames: 70.2% for the four-story IMF and by 19.4% for the six-story IMF. The increase in seismic performance caused by the SCWB ratio is also evident from looking at the ACMR values. Comparing ACMR to the FEMA P695 benchmarks, the results of Table 1 show that all four models obtained ACMR values above the acceptable benchmark of ACMR20% and ACMR10%, deeming them acceptable based on FEMA P695.

**Table 1: Summary of Performance Assessment using ATC-63 Methodology**

Stories	SCWB	$T(s)$	$S_{MT}$	$S_{CT}$	CMR	SSF	ACMR	ACMR	ACMR
								20%	10%
4	N	0.79	0.365	1.14	3.11	1.20	3.73	1.52	1.90
4	Y	0.79	0.365	1.94	5.30	1.20	6.35	1.52	1.90
6	N	1.14	0.250	1.44	5.74	1.24	7.11	1.52	1.90
6	Y	1.14	0.250	1.72	6.86	1.24	8.51	1.52	1.90

## CONCLUSIONS

The case study illustrated the inherent overstrength found in the IMF design provisions for flexural capacity and detailing along with shear reinforcement detailing at plastic hinge locations. All of the four models analyzed had ACMRs greater than the acceptable benchmark ACMR20% and also the ACMR10% benchmark.

Additionally, the study showed that the suggested addition of a 1.2 SCWB ratio is not needed as part of the IMF design criteria for buildings up to six stories tall. While the ratio did improve the performance, the benefit was not significant due to the acceptable ACMR values obtained by the IMFs without the SCWB addition. Therefore, the addition of the ratio would only serve to increase member dimensions and the amount of reinforcement within the frame. Further study is needed to see if buildings taller than 6 stories would benefit from such a ratio.

One interesting finding discovered during the static pushover analysis was the amount of maximum base shear being subjected to the four IMF models. Table 2 illustrates the maximum base shear being subjected to the IMF along with the base shear value used in design. The overstrength  $\Omega$  is the ratio of  $V_{max}$  and  $V$ . The table shows that the ultimate base shear is much larger than the shear value used for design. This difference allows two conclusions to be drawn from the data. First, the large amount of overstrength signified that the gravity design requirements along with the minimum design values were governing the IMF design and aiding in the seismic performance.

**Table 2: Comparison of Maximum Base Shear and Design Base Shear**

<i>Model</i>	<i>Stories</i>	<i>SCWB</i>	<i>Max. Shear, <math>V_{max}</math></i>	<i>Design Shear, <math>V</math></i>	<i>Overstrength, <math>\Omega</math></i>
1	4	No	758 kips (3371 kN)	126 kips (563 kN)	6.02
2	4	Yes	1018 kips (4528 kN)	126 kips (563 kN)	8.08
3	6	No	1052 kips (4679 kN)	132 kips (587 kN)	7.97
4	6	Yes	1471 kips (6543 kN)	132 kips (587 kN)	11.2

The second conclusion that can be drawn from the base shear data, and an area of potential concern, is that the IMF may collapse due to shear failure before it reaches the collapse level acceleration associated with a flexural mechanism as indicated in the simulation. In design, ACI-318 2008 detailing requirements for IMFs specify that transverse reinforcement be designed for shear forces calculated using the provisions of section 21.3.3. The intent of this transverse reinforcement is to provide sufficient shear capacity to enable the development of member flexural capacity prior to shear failure. The IMF model in the study therefore assumed that adequate shear capacity was provided to allow the formation of plastic hinges and did not consider member shear capacity directly. Beam-column elements used studied flexural mechanisms at member ends where it was envisioned that plastic hinges would form prior to shear failure. However, with values of  $V_{max}$  from Table 2 much higher than the value of  $V$

for which transverse reinforcement was designed, shear failure could be occurring in members. This concern is a limitation of the current study and model yet OpenSees can account for shear failure using shear springs. Therefore, future study is needed in order for shear failure along beam elements to be accounted for in the dynamic analysis of the IMF model.

The results of this study on IMF performance illustrate the usefulness of the FEMA P695 Methodology to test the performance of design criteria for structural systems. Current ACI-318 criteria for IMF design were found to be acceptable for the buildings studied in terms of seismic performance. The results also highlight the need for additional study on the shear capacity and seismic shear performance along the span of framing members. This study contributes to the knowledge on IMF seismic performance.

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