QUANTITATIVE ASSESSMENT OF CODE PROVISIONS FOR VERTICAL BUILDING IRREGULARITIES IN FRAME BUILDINGS

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ABSTRACT

Design codes address vertical irregularities in buildings by imposing additional design requirements and limitations on buildings deemed as irregular with the intention of maintaining acceptable seismic performance. However, relatively few studies have quantitatively evaluated the treatment of vertical irregularities in design codes. This study assesses the collapse performance of a variety of vertically irregular buildings as well as their treatment in building codes and standards (primarily ASCE/SEI 7) using the FEMA P695 Method.

The results demonstrate that the ASCE/SEI 7-16 seismic design provisions generally provide adequate collapse resistance for vertically irregular buildings. However, caution is recommended when “optimizing” a building design by reducing member sizes to the point that code requirements are barely met; this may dangerously reduce collapse resistance. It is also observed that prohibition of the Equivalent Lateral Force (ELF) method for some vertical irregularities, such as Mass Irregularity, is unnecessary, because ELF design forces tend to be more conservative than modal response spectrum analysis (even when both methods are scaled to the same base shear).

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Quantitative Assessment of Code Provisions for Vertical Building Irregularities in Frame Buildings

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ABSTRACT

Design codes address vertical irregularities in buildings by imposing additional design requirements and limitations on buildings deemed as irregular with the intention of maintaining acceptable seismic performance. However, relatively few studies have quantitatively evaluated the treatment of vertical irregularities in design codes. This study assesses the collapse performance of a variety of vertically irregular buildings as well as their treatment in building codes and standards (primarily ASCE/SEI 7) using the FEMA P695 Method.

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Objectives

The purpose of this study is to: (1) determine the effectiveness of modern code requirements, specifically ASCE/SEI 7-16 [1], for ensuring adequate earthquake collapse safety for buildings with vertical irregularities, and (2) to recommend updates to ASCE/SEI 7 for the treatment of vertical irregularities, if they are needed.

The following vertical irregularities are examined: mass irregularity; soft and weak story irregularities; also explored are thresholds for strong-column/weak-beam requirements, and effects of gravity induced lateral loads (e.g. from non-vertical columns) on collapse resistance.

References

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Method

The performance of archetype buildings with vertical irregularities is compared to baseline archetypes that are not irregular, using collapse resistance as the metric. Collapse resistance is computed with the FEMA P695 process [2]. The median spectral acceleration for which side-sway collapse occurs is computed by performing incremental dynamic analysis (IDA, [3]) with the FEMA P695 far-field ground motion set. The median collapse intensity is then divided by the Maximum Considered Earthquake (MCE) intensity to compute a collapse margin ratio (CMR) and adjusted collapse margin ratio (ACMR). FEMA P695 uses ACMR to compute probability of collapse, given MCE; this study focuses on changes to the CMR caused by vertical irregularities. If a vertical irregularity does not significantly reduce collapse resistance, then we conclude that the design procedures used to account for that irregularity are adequate.

Baseline archetypes

A set of baseline concrete and steel perimeter moment frame designs are developed, ranging from three to twenty stories. The archetypes are designed according to ASCE/SEI 7-16 for seismic design category (SDC) D_max and B_max and referenced design standards for steel [4 and 5] and concrete [6]. The equivalent lateral force procedure (ELF) and modal response spectrum analysis (MRS) from Chapter 12 are both used for design. All frames are designed as Risk Category II structures meeting a 2% design drift limit.

Structural modeling of the archetype buildings

General modeling considerations

Nonlinear models of the moment frame archetype buildings are constructed in OpenSEES [7] using the lumped plasticity method. The lumped plasticity models consist of linear elastic beam and column elements, joined by nonlinear elements where plastic deformations are expected to concentrate, as depicted in Figure 1. Nonlinear behavior is simulated with plastic hinges at the ends of beams and columns and with nonlinear joint panel zone elements.

Gravity loads are applied to the frames and also to a leaning column. Per FEMA P695, the load combination representing expected gravity loads, 1.05D + 0.25L, is used to compute the vertical load effects. Gravity loads directly carried by the frames are distributed on the beams. The remaining gravity load, which is carried by the building’s gravity system, is applied to the leaning P-Delta column. Building mass tributary to the walls is lumped at the wall nodes. For analysis, an expected mass of 1.05 times the dead weight is considered.

Steel moment frame modeling

Nonlinear beam and column hinges are modeled with the Modified Ibarra-Medina-Krawinkler deterioration model with bilinear hysteretic response [8] in OpenSEES. Initial backbone properties are calculated with the equations recommended by [9] and cyclic deterioration parameters are computed from [10]. For calculating the nonlinear properties of the beams and columns, the actual yield strength of the steel is assumed to be 10% higher than the nominal yield strength (per [9]).
Although composite action is not considered for the design of the frame beams, it is considered in the nonlinear analysis, because steel moment frames are commonly connected to a diaphragm. The controlling mechanism for composite action of the slabs is yielding of the shear studs, which are assumed be \( \frac{3}{4} \)" diameter, 75 ksi steel, placed at 12 inches on center. Panel zones are modeled with the Joint2D element in OpenSEES with nonlinear behavior idealized as trilinear without cyclic deterioration, based on Chapter 8 of [9] and [11].

Concrete moment frame modeling

Nonlinear beam and column hinges are idealized with trilinear backbones having peak-oriented hysteretic response and cyclic deterioration [8]. The backbone and hysteretic properties of the nonlinear beam-column hinges are computed from empirical relationships developed by [12] based on the design properties of the beams and columns (i.e. concrete strength, element dimensions, axial load ratio, and reinforcement detailing). RC OMF joints are calibrated according to [13-14] and are modeled with the Ibarra material model [8] with pinched cyclic response. Joint strength and rotation capacity depend on joint size, confinement, concrete strength, and axial load ratio. RC SMF joints are not expected to fail, so they are modeled with elastic elements.

Results

The performance of archetype buildings is gauged by dividing their ACMR by the ACMR of the baseline to get collapse resistance relative to the baseline—relative collapse resistance less than 1.0 corresponds to a reduction in collapse resistance. In some cases, the performance of buildings designed with the MRS method are contrasted with ELF designed buildings. Baseline and irregular
archetypes designed by the ELF procedure achieve equivalent or higher collapse resistance than the MRS designed buildings (even with ELF and MRS scaled to the same base shear). This observation indicates that some limitations on ELF, particularly the prohibition of ELF for mass irregular buildings, may be unnecessary.

**Weight (Mass) Irregularity**

Mass irregularity is examined by increasing the mass of a given story to produce a mass ratio ranging from 1.5 to 3 relative to the adjacent upper story. Increases in mass ratio trigger a re-evaluation of drift and strength requirements and the buildings are re-designed as needed.

Figures 2 and 3 show that concrete and steel moment frames tend to remain within 20% of the baseline collapse performance even at the highest mass ratio. In most models, the collapse performance decreases slightly with increasing mass ratio. These observations are seen regardless of the design method used for the model (MRS or ELF). In terms of collapse resistance, MRS provides no advantage over the ELF. This observation is important, because the only design requirement that is triggered by mass irregularity in ASCE/SEI 7-16 is the prohibition of the ELF design procedure. The location of the mass irregularity along the height of the building moderately affects collapse performance; archetypes with the mass irregularity in lower stories (where the building is already more naturally susceptible to damage) tend to perform less satisfactorily than those with mass irregularities in their upper stories.

![Figure 2. Collapse resistance relative to the baseline archetype vs. mass ratio for RC SMF archetypes designed with the ELF and MRS methods.](image)

**Soft and weak story irregularities**

Soft and weak story irregularities are investigated using concrete SMF and OMF frames for target strength ratios between 0.6 and 1.2 relative to the story above. Soft and weak story irregularities are achieved “organically” by simply increasing the story height. Weak story irregularity is also achieved with an alternative approach—increasing the strength of the story above.
The “organic” method (increased story height) of creating the irregularity does not produce clear trends in the RC SMF or OMF collapse performance—sample RC SMF results are shown in Figure 4. The collapse performance does not tend to drop below approximately 90% of the baseline model performance. In part, this is because increasing the story height does not always result in a decrease in story strength, particularly for OMF variants whose columns are governed by lateral force demands rather than strong-column-weak-beam ratio requirements. Similar observations are made when collapse resistance is plotted against relative stiffness, rather than relative strength.

The second method for achieving a weak story—increasing the strength of the story directly above the “weak” story—tends to result in collapse resistance somewhat higher for extremely low weak story ratios than for moderate weak story ratios. [15] showed that a weak story irregularity caused by strengthening all of the stories above (rather than only one story directly above) does not reduce collapse performance of RC moment frames either. The reason is a “weak” story irregularity in code-conforming buildings does not really mean that the story is
weak, but rather the adjacent story is strong—so collapse resistance is not compromised.

![Diagram](image)

Figure 5. Collapse resistance relative to the baseline archetype vs. story strength ratio for special RCMF archetypes designed with the ELF method. Story strength ratios are achieved by increasing strengths above the critical story.

**Strong-column-weak-beam**

The effects of strong-column/weak-beam are examined by analyzing suites of moment frame buildings designed for a range of strong-column-weak-beam ratios (SCWBR). In addition, archetypes with stepped SCWBR are examined; for these, the minimum SCWBR is higher at the lower stories and “steps” down in the upper stories of the building. For the stepped designs, minimum SCWBR at the first, second, and third quarters of the buildings are 1.5, 1.0, and 0.8 respectively or 2.0, 1.5, and 1.0, respectively, with no SCWBR requirement in the top quarter.

The SCWBR variants are designed two ways: (1) adjust the minimum SCWBR that is allowed for design, but allow it to be greater where other factors control; (2) target an exact SCWBR, even if it makes the design unrealistic (e.g. impossible reinforcing configurations). The concrete frames show a distinct trend of increasing collapse performance with increasing SCWBR (Figures 6-7), as expected. This is a result of damage spreading more over the height of the building as the SCWBR increases. Similar observations have been made in prior studies as well, e.g. [15].

Performance of RC SMF buildings with stepped SCWBR requirements are examined in Figure 7. Targeting higher SCWBR in the lower portions of buildings and relaxing SCWBR requirements in the upper stories may be a more efficient way to proportion a building, because it increases collapse resistance by placing additional column strength were it is appears to be most effective—the lower portion of the building. However, the SCWBR should not be allowed to go below current code limits, even in the upper portion of the building, as evidenced by the results for stepped SCWBR variants with very low SCWBR in their upper stories; the reduced collapse resistance in those buildings is caused by column failures in the upper portions of the buildings.

Results for the steel frames, shown in Figure 8, reveal less consistent trends with increasing SCWBR. This inconsistent performance is due predominately to a mechanism shift from joint shear yielding at low SCWBR to linear panel zone behavior and beam hinging at higher SCWBR.
A side-study revealed that adding oversized doubler plates to completely eliminate panel zone yielding reduced the collapse resistance of the archetypes with low SCWB and, as a result, produced the expected trend of increasing performance with increasing SCWB. The benefit of moderate yielding in the joint panel zone has been noted in prior studies as well, e.g. [16].

![Figure 6. Strong-column-weak-beam, RC SMF.](image)

![Figure 7. Stepped SCWB, 12-story RC SMF.](image)

**Gravity-induced lateral demand (GILD)**

GILD irregularity is examined with 20 story steel frames. The irregularity is achieved by sloping the columns at one story, e.g. Figure 9. The severity of the irregularity is quantified by the GILD ratio—gravity induced shear demand ($Q_G$) divided by lateral capacity of the story ($Q_V$).

GILD ratios even up to 0.5 do not negatively affect collapse performance, as long as the gravity induced lateral demand is considered in design. To further test the effect of GILD ratios on building performance, selected baseline archetypes subjected to GILD without any change to the building design are also examined. Figure 10 illustrates that building performance may be significantly reduced by higher GILD ratios if the gravity-induced lateral demand is not accounted.
for in the design process, especially for OMF’s.

![Graph](image)

**Figure 8.** Strong column/weak beam, steel SMF.

**Limitations of the GILD studies**

GILD Irregularities are examined in this study through models subjected to only horizontal ground accelerations with irregularities placed at predetermined stories. However, vertical ground motions may have a significant effect on collapse performance when columns are not vertical. The effect of GILD ratios on buildings is also expected to be more significant in the case where the irregular story and the area of the building prone to damage coincide (e.g. the first or second story). Since these factors are not fully examined in this study, the results presented should not be interpreted as a complete picture of the effects of GILD on building performance but rather a useful data point.

![Graph](image)

**Figure 9.** Gravity-induced lateral demand (GILD).

**Figure 10.** GILD irregularity, 20 story steel SMF. GILD accounted for in design.
Conclusions and Recommendations

None of the vertical configuration irregularities examined in this study cause unacceptable decreases in collapse resistance for moment frame buildings, as long as the building still meets code requirements. Additionally, the equivalent lateral force procedure (ELF) consistently produces safer buildings than modal response spectrum analysis (MRS), regardless of the presence of a vertical irregularity, even when MRS base shear is scaled to 100% of ELF. Therefore, requiring MRS for vertically irregular buildings (e.g. mass irregularity) may be unnecessary.

Studying the effects of strong-column-weak-beam ratio (SCWBR) requirements on moment frames reveals that designing a building with higher SCWBR in the lower portion of the building is an efficient way to enhance collapse performance. However, a “stepped” design that reduces SCWBR up the height of a building should not allow SCWBR below current code requirements at any level. However, additional studies are required in order to make more detailed recommendations for stepped SCWBR design.

Gravity induced lateral demand resulting from non-vertical columns does not reduce collapse resistance if the additional lateral demand is considered in the design process. Neglecting to account for gravity induced lateral demand, however, can drastically reduce collapse performance. No evidence is observed that sources of gravity induced lateral demand should be specifically treated as an irregularity in ASCE/SEI 7, but additional studies are needed to solidify this observation.

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