COLLAPSE IN SLIDING ISOLATED MRFS CONSIDERING DIFFERENT DESIGN METHODOLOGIES

T.C. Becker¹ and Y. Bao²

ABSTRACT

Seismic isolation protects both the structural and non-structural component at the expense of large horizontal displacement at the isolation level. Under some circumstances the excessive displacement may exceed the displacement capacity of sliding isolation bearing or cause the impact against the retaining wall, both of which may damage or even precipitate collapse of the superstructure. In this study a comprehensive numerical model, capable of capturing bearing uplift failure and nonlinear superstructure behavior, is used to investigate the effects of superstructure yielding on the collapse performance. Previous studies have not explicitly included bearing failure. Three base-isolated systems are designed; one designed to the ASCE 7 code, one with increased superstructure strength, and one with increased bearing displacement capacity. Fourteen pairs of near-fault pulse-like ground motions are used for the collapse risk assessment. Following the FEMA P695 methodology, the collapse margin ratio, system-level collapse mechanism and probability of collapse are quantified. Analysis results show that increasing bearing displacement capacity is more effective at minimizing collapse probability; however, this results in minimal margin between bearing rim impact and collapse. Additionally, the modes of collapse for each design are compared.

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Seismic isolation protects both the structural and non-structural component at the expense of large horizontal displacement at the isolation level. Under some circumstances the excessive displacement may exceed the displacement capacity of sliding isolation bearing or cause the impact against the retaining wall, both of which may damage or even precipitate collapse of the superstructure. In this study a comprehensive numerical model, capable of capturing bearing uplift failure and nonlinear superstructure behavior, is used to investigate the effects of superstructure yielding on the collapse performance. Previous studies have not explicitly included bearing failure. Three base-isolated systems are designed; one designed to the ASCE 7 code, one with increased superstructure strength, and one with increased bearing displacement capacity. Fourteen pairs of near-fault pulse-like ground motions are used for the collapse risk assessment. Following the FEMA P695 methodology, the collapse margin ratio, system-level collapse mechanism and probability of collapse are quantified. Analysis results show that increasing bearing displacement capacity is more effective at minimizing collapse probability; however, this results in minimal margin between bearing rim impact and collapse. Additionally, the modes of collapse for each design are compared.

Introduction

While base-isolation provides superior structural and non-structural performance under design level earthquakes, its performance under extreme events is less clear. Fixed-base buildings are designed with an idea of capacity design to maximize system ductility. However, for isolated buildings there is not a clear consensus on what should happen first: bearing failure, superstructure yielding, or moat impact.

Several researchers have looked at the influence of superstructure yielding using simplified two degree of freedom model [1,2] and have concluded that allowing yielding in the isolated superstructure will cause significantly larger ductility demands compared to yielding in fixed-base

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structures. However, these simplified studies cannot account for the complex behavior of a full frame. The general effects of pounding of isolated buildings with adjacent buildings have been examined in previous studies [3,4]. Both studies conclude that impact force will increase floor accelerations and excite undesirable higher mode vibration; however, these studies use simple lumped-mass models and assume elastic behavior of superstructure even after impact. The effects of pounding against the moat wall have been investigated both experimentally and numerically [5,6,7]. These studies include the inelastic behavior of superstructure and they found that moat-wall impact can induce yielding of superstructure and increase the collapse probability of base-isolated structures. Bao et al. looked at the failure of double friction pendulum bearings both numerically [8] and experimentally [9], and Becker et al. [10] explored the failure a triple friction pendulum bearing isolated frame. These studies show that for sliding isolation bearings, bearing uplift may significantly contribute to system-level failure. However, no study has determined whether designing so that superstructure yielding or bearing failure which design methodology results in the smallest collapse probability.

This paper presents the collapse assessment of three design methodologies for moment resisting frames isolated with sliding bearings: (1) a code compliant baseline design, (2) a design in which the superstructure is designed to remain elastic under moderate impact force, and (3) a design with the baseline superstructure but increased isolation displacement capacity. The isolated structures are designed with ASCE 7-16 [11] and evaluated following the FEMA P695 [12] framework. As isolated buildings are particularly sensitive to long-period motions, fourteen pairs of pulse-like near-fault ground motions provided in FEMA P695 are used to evaluate collapse margin ratio (CMR), collapse modes, and probability of failure under maximum considered level earthquakes. This study used a comprehensive model which can simulate both the failure of the sliding isolation bearings as well as the degrading behavior of superstructures.

**Design of the Isolated MRF**

Three different categories of systems are designed and evaluated in this study. In the first system, the superstructure and isolation system are designed in accordance with the most recent version of the ASCE isolation code [11]. In this design methodology, the building is designed to begin yielding under MCE level forces. However, the bearing’s displacement capacity is only nominally larger than MCE displacement. Thus, the code does not clearly define the preferred mechanism for which nonlinear behavior will control: superstructure yielding or potential bearing failure. Thus, the system-level failure mode may be heavily influenced by the superstructure and isolation design parameters. To study the system-level failure mode and investigate which design strategy results in the smallest collapse probability, two additional systems are investigated: one with a strengthened superstructure and one with an elongated bearing displacement capacity.

The isolated buildings are located in Los Angeles, California with site class is Class C (i.e. very dense soil and soft rock). The 5% damped MCE design spectrum has spectral acceleration values $S_s = 2.35$ g and $S_I = 0.98$ g. The superstructures are three-story three-bay steel moment-resisting and concentrically-braced frames, as depicted in Fig. 1. The sliding isolation bearing is selected as double friction pendulum bearing (Fig. 1) mounted below each column. Double friction pendulum bearings used in the US typically have an articulated (hinged) inner slider. The bearing chosen for this study is not articulated, but was chosen for two reasons: (1) its closer similarity to
the behavior the triple pendulum bearings the with rigid inner puck and (2) its relative simplicity for modeling. The target displacement under the MCE level is selected to be 0.65 m. With a sliding friction coefficient of 0.05, the radius of curvature of double friction pendulum bearing is then selected as \( R = 2.75 \) m, which corresponds to an effective period of 3.96 s with 19% equivalent damping.

System I has a code compliant design. Equivalent lateral force procedure is used to design the superstructure. The design base shear transmitted from the bearing at the MCE displacement is 0.169 g. An isolated steel intermediate moment resisting frame with \( R_i = 1.69 \) is used, resulting in a design shear of 0.087 g. The design of the frame is drift-controlled with a drift limit of 1.5%. The final beam and column designs are summarized in Table 1. The bearing is designed to account for accident torsion, thus, the ultimate displacement capacity for the isolation bearing is set as 0.79 m.

System II is designed using a much higher base shear coefficient so that the frame will develop yielding only after impact. For the superstructure design, the response modification factor \( R_i \) is selected as 1 and the base shear is assumed to be 120% of the shear force of the isolation bearing at its maximum displacement. The 20% increase is used to account for the impact. This results in design shear of 0.193 g. The final beam and column, and brace sections for system II are listed in Table 2. As expected, all cross sections are larger than system I increasing the stiffness and strength of the superstructures.

System III is designed so that the superstructure will develop considerable yielding before bearing impact happens. This could be done by increasing the response modification factor \( R_i \), however, the design code explicitly constrains \( R_i \). To comply with the design code, the displacement capacity of the isolation bearings is increased by 50% (i.e. displacement capacity is increased to 1.19 m) while keeping the superstructure design identical to the system I. The 50% increase in displacement capacity is roughly the median plus one standard deviation of displacement demand from the suite of ground motions.

![Figure 1. MRF (left) and double friction pendulum bearing (right) used in the study](image-url)
Table 1. Beam and column sections

<table>
<thead>
<tr>
<th>Floor</th>
<th>Systems I and III</th>
<th>System II</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Column section</td>
<td>Beam section</td>
</tr>
<tr>
<td>First floor</td>
<td>W14x132</td>
<td>W24x62</td>
</tr>
<tr>
<td>Second floor</td>
<td>W14x132</td>
<td>W24x62</td>
</tr>
<tr>
<td>Third floor</td>
<td>W14x90</td>
<td>W18x60</td>
</tr>
</tbody>
</table>

Modeling

The bearing model based on the work of [14] is used in this study. As this model assumes each component of sliding bearing as a rigid body, it is referred to here as the rigid body model. Uplift failure of the isolation bearing is characterized by any one of the four vertexes moves outside the restraining rim of top and bottom plates. The rigid body model cannot predict failure due to the restraining rim yielding, but a previous finite element study [8] found that critical ground motion parameters such as spectral accelerations are not significantly affected by the bearing failure modes. However, this does allow the model to develop potentially unlimited impact force, and shake table testing [9] has shown that the impact forces are limited due to the yielding of restraining rim. This is a drawback of the rigid body model in the assessment of collapse probability, but this will result in a smaller, rather than artificially larger CMR because the real impact forces should be less compared to rigid body model prediction. As the implementation of the rigid body model is in state-space form, the superstructure model is also implemented in state-space form. In order to evaluate the collapse risk of the isolated structures, the superstructure model includes stiffness and strength degrading behavior and second order geometric effects.

Collapse Assessment

Fourteen pairs of near-fault pulse-like ground motions from FEMA P695 (28 ground motions in total) are used as input to evaluate the collapse probability. Each pair of ground motions is first normalized by their peak ground velocities using the normalization factor provided in FEMA P695 Appendix A; then all 28 ground motions are scaled as a group to match the target MCE design spectrum at the effective period of the bearings. These motions are used to conduct incremental dynamic analysis (IDA) of the MRF designs.

Usually IDA curves plot intensity measures (e.g. peak ground acceleration (PGA), pseudo acceleration as the first period (SA(T1)) etc.) against seismic response parameters (e.g. maximum drift ratio, floor acceleration etc.) of interest. In this study two response parameters are used: maximum story drift ratio and maximum ductility demand at the component level. The ductility is the ratio of maximum curvature within a beam or column element to its yield curvature. Collapse of the superstructure due to excessive yielding is defined by the maximum story drift ratio, which has a threshold value of 5%. The maximum ductility is presented only to highlight the effect of yielding on the safety margin between impact and system-level failure. The concepts of impact margin ratio (IMR), CMR, and safety margin are used as well. The IMR and CMR are defined as the ratio of spectral acceleration at the isolation period to the spectral acceleration of the MCE level at which half of the suite of ground motions cause impact or collapse respectively. Note that
the term collapse here means system-level failure, either triggered by excessive superstructure yielding or bearing uplift failure. The safety margin is defined as CMR minus IMR.

**Individual ground motions**

Fig. 2 shows the IDA curves with the maximum drift ratio and maximum ductility for a representative ground motion. The percentage of MCE level is used in lieu of traditional intensity measure for consistency with IMR and CMR. The IDA curve starts from 50% of MCE level to evaluate behavior from impact to bearing uplift failure for the moment-resisting frames. Varying the design has a profound influence on the system-level behavior. For this ground motion, for the System I code compliant design, bearing impact first occurs at 90% MCE. At 110% MCE, the maximum drift ratios of the 1st and 2nd floor exceed 5%, therefore superstructure collapse is assumed at this level. Explicit bearing uplift will not happen until 130% MCE. For the System II stronger frame design, impact occurs at 90% MCE, and at 120% MCE the bearing exhibits uplift failure but the maximum drift ratio is still less than 5%. This indicates the system-level failure mechanism shifts from superstructure yielding to bearing uplift failure, due to the increase in strength and stiffness of the superstructure. For the System III larger bearing design, impact first occurs at 130% MCE due to the increase in the bearing displacement capacity. Here, impact immediately causes drift ratios larger than 5% in the 1st and 2nd floors. Explicit bearing uplift failure happens at 140% MCE.

For ductility, the abbreviations of IO, LS, and CP in the figure denote Immediate Occupancy, Life Safety, and Collapse Prevention, with threshold values defined in FEMA 356 [13]. The t for maximum ductility is consistent with maximum drift ratio: when impact occurs, the system II stronger frame design develops the lowest ductility while system III larger bearing design has the largest ductility. It is also notable that for the larger bearing design, prior to impact the maximum ductility grows to as large as 4, but for the code compliant design and stronger frame design, this values are only 2 and 1.5 respectively.
Figure 2. IDA curves for maximum drift and curvature ductility for a representative ground motion
Suite of motions

The median IDA curves for maximum drift ratio and ductility are shown in Fig. 3; these median IDA curves only show the behavior up to system-level collapse and do not categorize the sources of system-level failure. The median IDA curves from a suite of ground motions are consistent with the conclusions made from the individual motion. The IRM and CMR are 120% and 140% MCE for the code compliant design, 110% and 150% for the stronger frame design, and 170% and 180% for the larger bearing design. The larger bearing design has the largest IMR but the safety margin between impact and collapse is only 10%. The stronger frame design has the lowest IMR but the safety margin is 40%, which is the highest among three systems. Note that for the median IDA curves for maximum drift ratio, when the CMR is reached, the median value may not exceed 5%. This is because the bearing uplift failure contributes to system-level failure.

The median IDA curves for maximum ductility also highlight the difference in design: prior to reaching IMR, the larger bearing design develops maximum ductility as large as 3, but the code compliant design and stronger frame design have only 2 and 1.5 respectively. These results show, for a flexible isolated frame such as the MRF, the strength has an important role on its ductility demand upon impact: increasing strength has considerable beneficial effects on avoiding excessive superstructure yielding. Increasing the displacement capacity is the most straightforward way to protect superstructure, but the consequence is the superstructure cannot survive any impact.

It is useful to categorize the sources of system-level failure for the isolated structures with different design methodologies. In this study, whichever failure mode (superstructure yielding or bearing failure) is observed earlier is defined as the source of the system-level failure. Using this definition, the decomposition of system-level failure is depicted in Fig. 4 for the first 14 ground motions to cause failure. For System I the code compliant design, no system-level failure is due solely to the isolation bearings, six ground motions exhibit a mixed failure mode where both isolation bearing uplift failure and superstructure yielding contribute, and eight ground motions have failure from excessive superstructure drifts. But for the stronger frame design, three ground motions have failure due solely to the isolation bearing uplift without excessive superstructure yielding. When using larger bearing design, superstructure yielding becomes the dominant failure mode, causing system-level failure in thirteen out of fourteen motions.

Although it is not the focus of this study, the case in which the isolated moment-resisting frames impact against a moat wall prior to researching the bearing restrainer is also considered. In the configuration with the moat wall, impact and failure of the sliding isolation bearing is eliminated. When the moat wall is considered, the CMR are similar when comparing impact against moat wall with isolation bearing. The only significant difference is the failure mode which is controlled exclusively through excessive superstructure yielding. Furthermore, the general findings in this study are in agreement with those of Masroor and Mosqueda [7] which investigated the collapse of isolated frames including the moat wall but without bearing failure. This study found that, predictably, increasing the displacement limit decreased collapse probability, as with the larger bearing design presented here.
Figure 3. Median IDA curves for maximum drift (top) and curvature ductility (bottom) for the suite of motions

Figure 4. Decomposition of failure modes for the three designs

Collapse probability

The collapse probabilities of isolated structures are evaluated using the FEMA P695 methodology. As the intensity of the suite of ground motions is larger than the seismic design category $D_{\text{max}}$, the spectral shape factor (SSF) must be calculated. Taking into account the location and the pushover curve, the SSF is determined to be 1.275 for Systems I and III and 1.277 for System II. The adjusted collapse margin ratio (i.e. ACMR) can then be calculated from $\text{ACMR} = \text{SSF} \times \text{CMR}$.

It is also necessary to account for uncertainties when assessing the collapse probability. Four different uncertainties are introduced in the FEMA P695 methodology: 1) record to record uncertainty $\beta_{\text{RTR}}$; 2) design requirement uncertainty $\beta_{\text{DR}}$; 3) test data uncertainty $\beta_{\text{TD}}$; and 4) modeling uncertainty $\beta_{\text{MDL}}$. In this study the uncertainties are assigned ratings: (B) good for design requirement, (B) good for test data, and (B) good for modeling. Thus, the uncertainties are
quantified as: $\beta_{DR} = 0.2$, $\beta_{TD} = 0.2$ and $\beta_{MDL} = 0.2$. The record to record uncertainty is $\beta_{RTR} = 0.314$ for the code compliant design and $\beta_{RTR} = 0.344$ for the stronger frame design.

The collapse probability curves are shown in Fig. 5. All three designs meet the required collapse probability, i.e. less than 10% collapse probability at the MCE level as specified in FEMA P695. The code compliant design has a collapse probability of 8% while the stronger frame design and the larger bearing design both have a collapse probability of 5% and 3% respectively. This demonstrates good seismic performance for isolated moment-resisting frames as well as the clear benefit of enhanced designs.

![Figure 5. Collapse probability curves of isolated MRFs](image)

**Conclusions**

In this paper the collapse risk of isolated moment resisting frames considering the extreme behavior of sliding isolation bearings is evaluated. Three different systems are designed in accordance with the latest ASCE 7-16 code: the code compliant design, the stronger frame design, and the larger bearing design. Fourteen pairs of near-fault pulse-like ground motions are selected as the suite of input motions. Varying the design has a profound influence on the mode of system-level failure. For the code compliant design and stronger frame design, the system-level failure mode is mixed; both bearing uplift failure and superstructure yielding contribute. In contrast, for the larger bearing design, system-level failure is dominated by superstructure yielding. While the larger bearing design has the largest CMR, it has the smallest safety margin between impact and collapse. The stronger frame design has the largest safety margin making it the most resistant to impact forces. All three designs meet the requirement of acceptable collapse probability at the MCE level.

While not discussed in this paper, the using concentrically braced frames yields very different results. For this stiffer superstructure, the code design structure did not meet the target 10% probability of collapse in the MCE and increasing the frame strength did not provide significant benefit. Also, as the impact force imposes substantial ductility demand on the superstructure regardless of its strength, the failure comes solely from superstructure yielding. Only increasing bearing displacement capacity can improve collapse risk in extreme events. This observation suggests that for isolated stiff superstructures impact should be avoided. These results are discussed in [15].
References


11. American Society of Civil Engineers. ASCE 7-16: Minimum design loads and associated criteria for buildings and other structures, Reston, VA, 2016.


