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Advances to the FEMA-P58 Methodology for Resilient BRBF Design and Implications for BRBF Building Performance

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Abstract

The FEMA P-58 Method for assessing the seismic performance of buildings is a rigorous, building-specific and site-specific method that can be used for estimating seismic repair costs and closure times for building structures. In the FEMA P-58 Method, seismic losses and repair times are predicted by quantifying seismic hazard at the site, quantifying the response of the building to seismic demands, and then predicting damage to the components in the building (both structural and non-structural). The sensitivity of the FEMA P-58 Method to nuances of the building and site make it important that any FEMA P-58 assessments be conducted with the best information available. Significant research has been conducted in the past two years to better understand and model buckling-restrained braced frame (BRBF) structural responses, as well as to assess how the buckling-restrained braces (BRB) themselves perform under seismic loading. This research specifically includes creation of advanced fragility functions for BRBF components, which reflect the precise geometry of the brace (slope, distance between work points, etc.). Additionally, this research created a method to estimate structural responses of BRBF buildings, including both peak drifts, residual drifts (big focus), and peak floor accelerations. This structural response method was created based on nonlinear response history (NLRH) modeling of 19 BRBF buildings (with 6 variants on each building), with heights between 1 and 18 stories, with the buildings being subjected to a suite of 44 earthquake records. In this paper, the effects of these advancements are explored and contrasted with earlier understanding of BRBF building seismic performance and it is shown that much conservatism was included in past FEMA P-58 assessments of BRBF buildings (specifically in terms of the effects of residual drifts).

Keywords: seismic risk, residual drifts, buckling-restrained, braced frames, FEMA P-58, SP3.

1. Introduction and Motivation

Over the past several years, the FEMA P-58 risk assessment method has become more commonly utilized for resilient design and advanced seismic risk assessment in structural engineering practice in the United States and internationally. This industry trend has led to significant additional research and development for specific structural system types (especially high-performing structural systems), with the goal of enabling more accurate FEMA P-58 seismic risk assessments for the specific structural systems.

This paper reports on such a study for buckling-restrained braced-frame (BRBF) buildings. The research presented in this paper was motivated by (a) the desire to enable more accurate BRBF risk assessments using FEMA P-58 and (b) the publication of a study that used approximate FEMA P-58 analysis methods to compare performance of various structural systems [1]. Based on the approximate analysis methods used in the prior study [1], the conclusion of that study was that BRBF buildings could show markedly poor performance for high ground shaking, and this made it clear that more accurate and less approximate (conservative) FEMA P-58 analysis methods were needed for BRBF buildings. To improve the modeling ability for BRBF buildings, this multi-year study looked at refined prediction of residual drifts, refined prediction of other building responses (peak transient drift and floor accelerations), and refined fragility models for BRBF components.

2. Prior Study on BRBF Repair Costs and Repair Times

The FEMA P-58 Volume 5 document [1] presented a comparative study of the seismic performance of multiple types of structural systems, with a focus on prediction of repair costs, repair time, and probability of building demolition due to unrepairable residual drifts (and other metrics were reported, but this paper focuses on these three most important metrics). The Volume 5 study compared Steel Special Moment Frames, Reinforced Concrete Moment Frames, Steel Building-Restrained Braced Frame, Steel Special Concentrically Braced Frames, and Special Reinforced Concrete Shear Walls. In addition to structural system variations, the Volume 5 study also looked at changes to occupancy including office occupancies and various types of medical occupancies (including hospitals).

Due to the broad scope of the Volume 5 study, comparing the seismic performance across the many building types and variations necessarily required that the FEMA P-58 seismic risk analysis done for each case be simplified and approximate, and this highlighted the need to refine the analysis methods for BRBF buildings and was part of the motivation for this study.

Table 5-6 of the Volume 5 document [1] provides BRBF seismic performance estimates, but they are averaged over a large range of building configuration and designs, so comparing analyses of individual buildings to results shown in those tables would not be meaningful. To have a more focused point of comparison to the Volume 5 BRBF seismic performance estimates Tables 1 and 2 below show their results for a “representative” mid-rise BRBF building design for a high-seismic U.S. site (Seismic Design Category D); these results are more similar to an actual building, but note that they are still averaged in some way for “mid-rise” buildings, so the later comparative example that is done for a case-study 5-story building gives numbers that are similar to these tables, but not identical. These tables show the mean and median values for repair costs, repair times, and the probability of residual drifts being large enough to result in the building being demolished. These values come from the spreadsheet that accompanies the Volume 5 report documentation. Note that the repair times shown in these tables do not include impeding times (which include times for permitting, financing, engineering design, and contractor mobilization, but rather are only the times for actual repair, so these times underestimate the actual building closure time that would be experienced. In this paper, only the actual repair times will be used to keep the comparison consistent between this study and the prior referenced study on BRBF performance.

Tables 1 and 2 shows that the Volume 5 document predicts poor performance of BRBF buildings, especially at high levels of ground shaking. This high estimate of performance comes mostly from the residual drift modeling assumptions used in the Volume 5 project, and this will be discussed in detail later in this paper.

Table 1 – Volume 5 Mean performance metrics for a representative mid-rise BRBF building in SDC D

Performance Measure	**Performance at Each Intensity Level**				
	20% MCE	**40% MCE**	**67% MCE**	**80% MCE**	**100% MCE**
Repair Cost (%)	0.3%	6%	22%	39%	53%
Repair Time (days)	2	25	125	250	360
Probability of Unrepairable Residual	0%	2%	15%	30%	45%

Table 2 – Volume 5 Median performance metrics for a representative mid-rise BRBF building in SDC D

Performance Measure	**Performance at Each Intensity Level**				
	20% MCE	**40% MCE**	**67% MCE**	**80% MCE**	**100% MCE**
Repair Cost (%)	0.1%	2%	9%	15%	48%
Repair Time (days)	0	6	25	40	--*
Probability of Unrepairable Residual	(same as above table)				

*Median value reported in ATC-58-5 tool appears to be in error so if not reported here.

3. Summary of BRBF Seismic Risk Modeling Improvements Considered in this Study

This study focuses on FEMA P-58 modeling improvements to remove conservatisms present in the Volume 5 study. This section discusses the research done over the past couple years to make these modeling improvements for BRBF buildings.

3.1 Improved Fragility Models for the Buckling-Restrained Brace Components

The first research focus was to look at data from recent BRB components since the FEMA P-58 default fragility function was conservative compared to actual CoreBrace BRB test data. In addition, the FEMA P-58 default fragility function were independent of bracing length and configuration, which was a large approximation and geometry details are shown to lead to up to a ~2x change in the mean fragility value.

Data were utilized from 17 cyclic loading tests of CoreBrace BRB components and Figure 1a provides the fragility function that was created for the strain at brace fracture. Since this fragility function is based on strain, creating an interstory-drift-based fragility function depends on the brace geometry and end connection details. Figure 1b shows a family of CoreBrace BRB fragility functions for various geometries and end connection conditions. These fragility functions have been implemented in the Seismic Performance Prediction Program (SP3) software [2]. Figure 1b shows that the updated fragility functions have 2-3 times the drift capacity as compared with the FEMA P-58 default fragility function. Additionally, vertical lines are shown at 1% and 2% inter-story drift (which are typical design drift levels for BRBF buildings) which shows that these is near zero probability of fracture in the BRBFs at these expected levels of drift (while the prior, more conservative curve, showed up to 50% probability of fracture in this typical range of drifts).

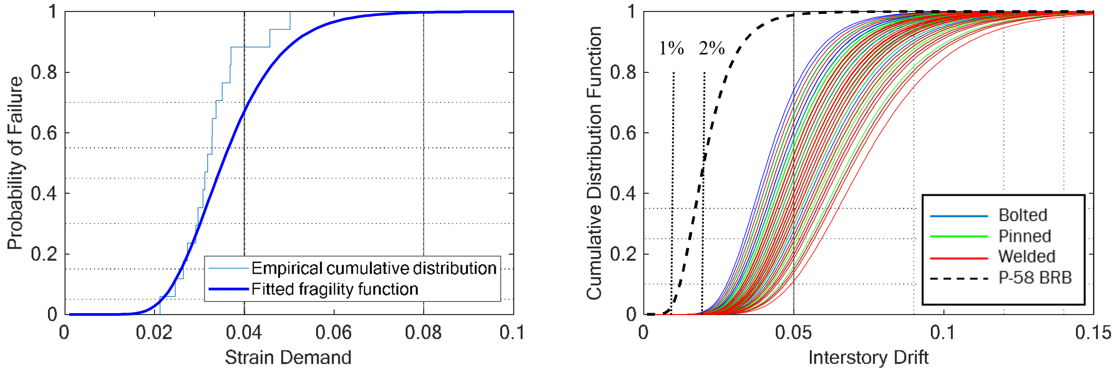

Figure 1 – Fragility functions for CoreBrace BRB components (a) based on strain, and (b) based on inter-story drift for various brace configurations

3.2 Improved Residual Drift Modeling Considering Stabilizing Effects of Gravity System and Possible Moment-Resisting Back-up Frame

The second major research topic was to improve the modeling of residual drifts for BRBF buildings. To accomplish this, 15 building designs were completed, and six analysis variants were used for each building design.

3.2.1 Building Designs

The designs used for the residual drift assessment were taken from the archetypes used in the NIST GCR 10-917-8 report, *Evaluation of the FEMA P-695 Methodology for Quantification of Building Seismic Performance Factors*, design by Rafael Sabelli [3]. These designs are summarized in Table 3 and were the high seismic (SDC D_{max}) single-diagonal brace and two-story X brace configurations. Table 4 summarizes the six variants that were run for each model, to investigate the effect of the backup frame presence, the base connection, and the presence of additional stiffness due to shear tab behavior in the gravity system bays. This includes a total of 90 building designs and modeling variants.

Table 3 – Summary of structures used for the development of the structural response method

Single Diagonal Bracing		**Two-Story X Bracing**	
Stories	**Bay Width**	**Stories**	**Bay Width**
1	25	2	20
2	15	2	30
3	25	3	30
4	15	6	30
6	15	12	20
9	15	12	30
12	15	16	30
18	15		

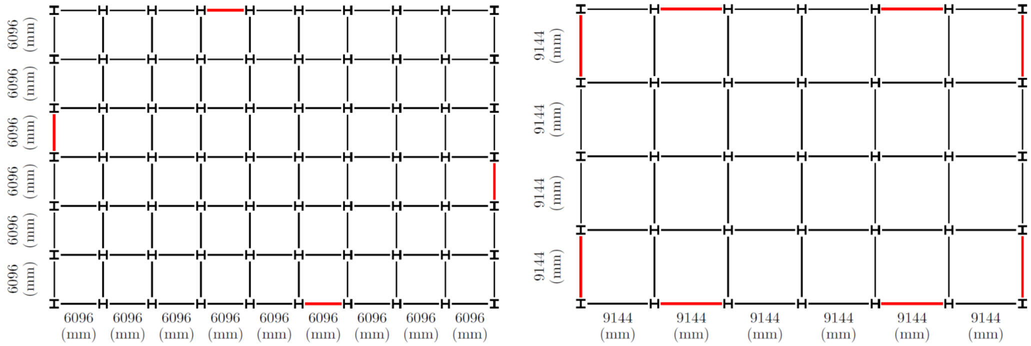

Figure 2 – Representative plan views of structures for two-story (left) and 12-story (right), taken from [5]. The 3-9 story archetypes have the same layout as the 12-story shown, but with only two braced frames in each direction rather than four.

Table 4 – Modeling variants considered for each building design

Variant	Backup Frame	Base Connection	Gravity System
1	None	Pinned	None
2	None	Pinned	Present
3	Present	Pinned	None
4	Present	Pinned	Present
5	Present	Fixed	None
6	Present	Fixed	Present

3.2.2 Overview of Modeling Approach

For each building, a two-dimensional nonlinear model was created using the OpenSees modeling platform [4].

Modeling of BRBF Components. The BRB strain hardening parameters, β and ω, were estimated using the design guides publically available from CoreBrace. The stiffness modification factor, KF, was also estimated using the provided design guides. The BRB components were modeled as truss elements in OpenSees with the backbone of the axial force-displacement response calibrated to the corresponding β and ω values.

Modeling of Gravity System. The gravity system was modeled in accordance with Chapter 4 of NIST GCR 17-917-46v2 [5]. Based on the loads specified for the building designs, and the tributary widths, the shear tab connections for the gravity beams were detailed for the various bay widths to reflect how the stiffness changes with different gravity bay spans.

Modeling of the Back-up Frame. The backup frame was modeled by fixing the beam to column connections, rather than modeling them as pins (which was done for the case of no backup frame). A concentrated hinge model was used in accordance with Chapter 4 of NIST GCR 17-917-46v2, Guidelines for Nonlinear Structural Analysis for Design of Buildings Part IIa – Steel Moment Frames [5] to account for both beam and column yielding.

Modeling of P-Delta. The P-Delta effects are accounted for using the common "linear with P-Delta" method in OpenSees [4].

3.2.3 Overview of Analysis Approach

Each nonlinear building model was analyzed using the OpenSees modeling platform [4] using the FEMA P695 ground motion set [6] (which is a standardized set of 44 far-field motions documented in Appendix A of FEMA P695). Incremental nonlinear dynamic response-history analysis was then completed from small ground motions (with elastic structural behavior) up to large ground motions (e.g. MCE highly nonlinear response). The structural responses that are tracked in the analysis are interstory drifts, peak floor accelerations, and residual drifts. Uncertainties and correlations in the response values are also tracked.

Figure 3 shows examples of the residual drift results from these sets of nonlinear dynamic structural analyses. Curves were then fit to these analysis results, using the simplified bilinear approach used in the default FEMA P-58 residual drift estimation method (which is used across the board for all structural systems). Figure 4 shows these updated residual drift curves and provides a comparison to the default FEMA P-58 curve. This shows that the default FEMA P-58 curve is highly conservative for BRBF buildings when the gravity system is accounted for, and especially conservative for BRBF buildings that have moment-connected back-up frames (and note that it was found that the back-up frame size simply can be designed in the typical manner for a BRBF building and does not need to be designed as an official duel system).

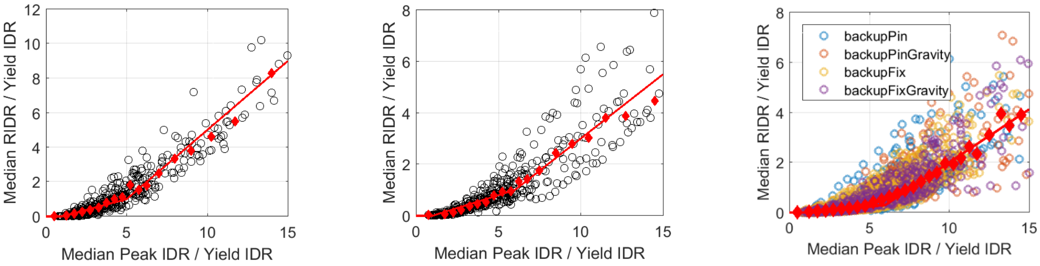

Figure 3 – Residual drifts for frames with (left) no backup frame and no gravity, (center) no backup frame with gravity, (right) all configurations with backup frame

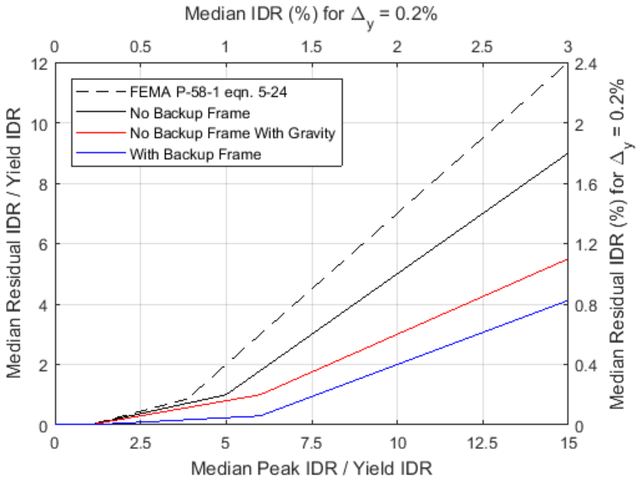

Figure 4 – Updated BRBF residual drift models and comparison to default FEMA P-58 model

3.3 Improved Structural Response Modeling for Peak Drifts and Floor Accelerations

In addition to improving the predictions of residual drifts, a major research effort also focused on improving the prediction of peak interstory drifts and peak floor accelerations, quantifying how they vary over the height of the building, and quantifying how they vary over ground motion intensity. This topic is too long for this paper, but these advanced methods are implemented in the SP3 Structural Response Prediction Engine module of the SP3 software [2].

3.4 Improved Design Assumptions for Building Models

In the FEMA P-58 Volume 5 report, the design process for the archetype designs kept strength and stiffness design uncoupled, which is obviously not the way that BRBF buildings are designed, leading to some conservatism in the risk assessment results. This was a required simplification in the Volume 5 study (due to the broad scope of that study), but the archetype model used in this study were fully designed with coupled strength and stiffness, to better reflect the design of these types of buildings.

4. Effects of Modeling Improvements on Predicted Building Performance

4.1 Overview of Case-Study Building and Comparative Analysis Steps

In order to see the effects of the improved modeling, and how the improved modeling compares with the prior Volume 5 [1] approximate performance assessments, a sample case is considered of a 5-story BRBF office building in San Francisco (at coordinates 37.8, -122.4) and assumed to be on Site Class D soil; this example is a fairly characteristic high-seismic site in California. The FEMA P-58 [7] analysis is completed using the SP3 software platform [2]. The Volume 5 approximate analysis assumptions are replicated and then the analysis is incrementally improved (per the research improvements summarized previously) to see the quantitative effects on the improvements, as follows. Table 5 then summarizes these steps in a concise manner.

- **Step 1:** Baseline analysis that replicates the approximate analysis assumptions used in the FEMA P-58 Volume 5 report [1] (results are shown to be similar to those shown in Section 2, though they do not match exactly due to averaging of results done in the Volume 5 report data.
- **Step 2:** Redesign the archetype building with coupled strength and stiffness in the design process (to reflect how BRBF buildings are designed).
- **Step 3:** Use the improved structural response methodology and improved residual drift modeling accounting for only the gravity frame of the building (assuming that the frames are not moment connected around the braced bays).
- **Step 4:** Use the improved BRB fragility functions that have been developed. (these show 2-3 times the deformation capacity at fracture, versus the FEMA P58 base values.)
- **Step 5:** Reanalyze the structure considering the effects of adding a moment-resisting back-up frame around the braced bays and reflect the positive impact of this frame in the residual drift modeling. Note that this modeling step reflects moment-connecting the beams and columns that are around the braces as part of the normal BRBF design and not redesigning the frame to be a technical dual system (e.g. to be able to hold 25% of the base shear).

Table 5 – Steps Completed for Comparative BRBF Analysis using FEMA P-58

Step	Coupled Strength/Stiffness	Structural Response Prediction Method	Residual Drift Model	BRB Fragilities
1	No	P-58 Simplified	P-58 Baseline	P-58 Default
2	Yes	P-58 Simplified	P-58 Baseline	P-58 Default
3	Yes	SP3 SRPE*	w/ GravitySystem	P-58 Default
4	Yes	SP3 SRPE	w/ GravitySystem	CoreBrace
5	Yes	SP3 SRPE	w/ BackupFrame	CoreBrace

The following three sections show the risk assessment comparisons for repair costs, repair time, and the probability of the building requiring demolition due to high residual drifts.

4.2 Comparison of Repair Costs

Figure 5 shows the mean and median repair costs for each level of analysis. Note that mean values are commonly much higher than median values, due to the residual drift contributions because averaged into the mean results (and losses are 100% when demolition is required due to excessive residual drift). This is shown over a range of ground motion intensity levels up to the Maximum Considered Earthquake (MCE) shaking intensity. In U.S. design practice, the MCE shaking intensity and 0.67 of the MCE intensity is the shaking level that the building is actually designed for. For high-seismic U.S. sites, the MCE has a return period of nominally 2500 years and the design level (0.67 of the MCE) has a return period of nominally 500 years (though actual return periods vary widely by site).

Figure 5 and Table 6 show that the mean repair cost from the baseline analysis (similar to the FEMA P-58 Volume 5 analysis) is 15% at the design level and 55% at the MCE level. After implementing the modeling improvements summarized in this paper, it is show that these baseline values are highly conservative, with most of the conservatism coming from the approximate residual drift modeling done in the Volume 5 analysis. After modeling improvements, the repair cost is 5% at the design level and 10%-25% at the MCE level (10% when a back-up frame is used and 25% when there is no moment-connected back-up frame).

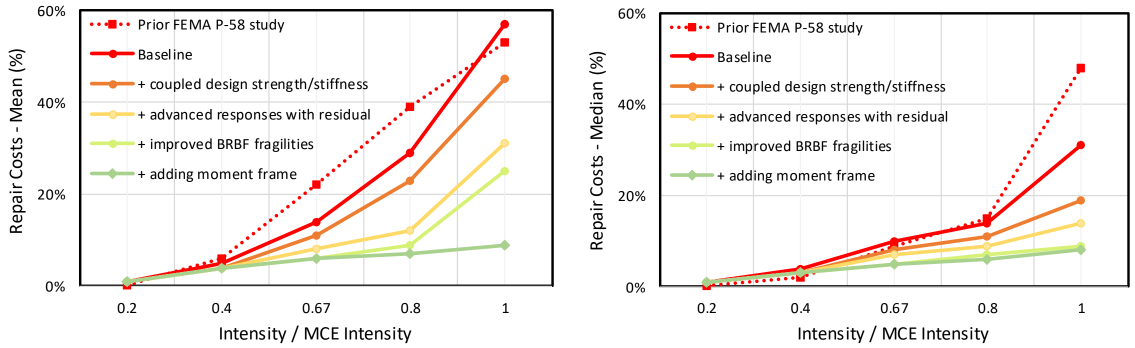

Figure 5 – Comparison of repair costs for various levels of analysis refinement
(mean values on left and median values on right)

Table 6 – Comparison of repair costs for various levels of analysis refinement
(mean values on top and median values on bottom)

Modeling Step for Mean Repair Cost	Performance at Each Intensity Level				
	20% MCE	40% MCE	67% MCE	80% MCE	100% MCE
Prior FEMA P-58 study	0%	6%	22%	39%	53%
Baseline	1%	5%	14%	29%	57%
+ coupled design strength/stiffness	1%	4%	11%	23%	45%
+ advanced responses with residual	1%	4%	8%	12%	31%
+ improved BRBF fragilities	1%	4%	6%	9%	25%
+ adding moment-connected frame	1%	4%	6%	7%	9%

Modeling Step for Median Repair Cost	Performance at Each Intensity Level				
	20% MCE	40% MCE	67% MCE	80% MCE	100% MCE
Prior FEMA P-58 study	0%	2%	9%	15%	48%
Baseline	1%	4%	10%	14%	31%
+ coupled design strength/stiffness	1%	3%	8%	11%	19%
+ advanced responses with residual	1%	3%	7%	9%	14%
+ improved BRBF fragilities	1%	3%	5%	7%	9%
+ adding moment-connected frame	1%	3%	5%	6%	8%

4.3 Comparison of Repair Times

Figure 6 and Table 7 show the mean and median repair time for each level of analysis. These values are based on FEMA P-58 simplified repair times and do not include full repair scheduling nor impeding factors that delay the onset of repair. The baseline analysis shows 45 days of mean repair time for the design level shaking and 275 days of repair time for MCE shaking. After implementing the modeling improvements summarized in this paper, the repair times are 10-15 days for the design level shaking and 20-100 days from MCE shaking (with the lower values being for a building with a moment-connected back-up frame and the higher values being for a building without a moment-connected frame).

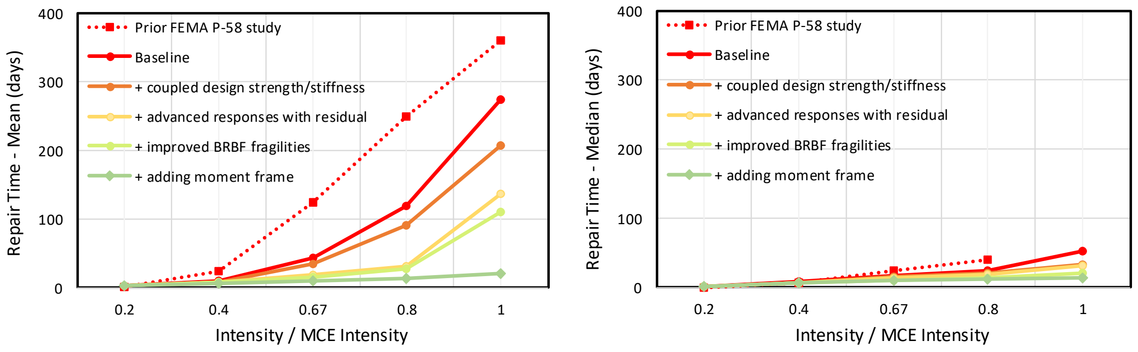

Figure 6 – Comparison of repair times for various levels of analysis refinement
(mean values on left and median values on right)

Table 7 – Comparison of repair times for various levels of analysis refinement
(mean values on top and median values on bottom)

Modeling Step for Mean Repair Time	Performance at Each Intensity Level				
	20% MCE	40% MCE	67% MCE	80% MCE	100% MCE
Prior FEMA P-58 study	2	25	125	250	360
Baseline	3	11	43	119	274
+ coupled design strength/stiffness	3	8	34	91	207
+ advanced responses with residual	3	8	19	32	136
+ improved BRBF fragilities	3	8	15	27	111
+ adding moment-connected frame	3	7	11	14	21

Modeling Step for Median Repair Time	Performance at Each Intensity Level				
	20% MCE	40% MCE	67% MCE	80% MCE	100% MCE
Prior FEMA P-58 study	0	6	25	40	--
Baseline	1	8	18	25	52
+ coupled design strength/stiffness	1	7	15	21	33
+ advanced responses with residual	1	7	14	19	32
+ improved BRBF fragilities	1	7	11	14	20
+ adding moment-connected frame	1	6	10	12	14

4.4 Comparison of Residual Drifts and Probability of Demolition

Figure 7 shows the median predicted residual drifts for each level of analysis, and it is noted that the baseline value or residual drifts are not reported in the prior Volume 5 study (only the demolition probabilities are reported, per Figure 8). The baseline analysis shows 0.3% median residual drift for the design level shaking and 0.9% for MCE shaking (and these are comparable to a 1% average residual drift causing expected demolition of the building). After implementing the modeling improvements summarized in this paper, the residual drifts are 0.05% to 0.15% for the design level shaking and 0.15% to 0.5% for MCE shaking (with the lower values being for a building with a moment-connected back-up frame and the higher values being for a building without a moment-connected frame).

Figure 8 and Table 8 show the resulting probability of demolition (due to residual drifts above the allowable threshold) for each level of analysis. The baseline analysis shows 5% probability of demolition for the design level shaking and 50% for MCE shaking. After implementing the modeling improvements summarized in this paper, the probabilities of demolition are 0% to 1% for the design level shaking and 1% to 20% for MCE shaking (with the lower values being for a building with a moment-connected back-up frame and the higher values being for a building without a moment-connected frame).

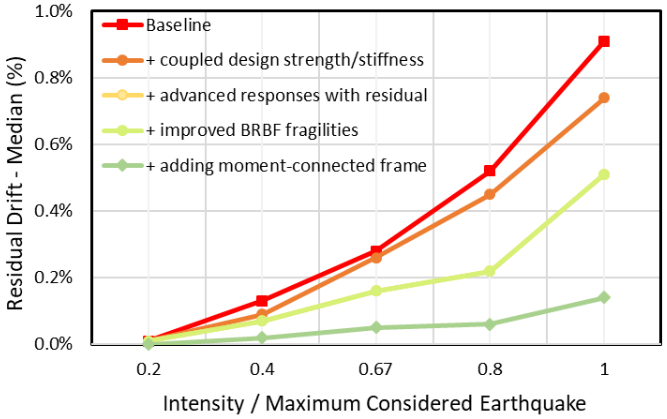

Figure 7 – Comparison of median residual drifts for various levels of analysis refinement

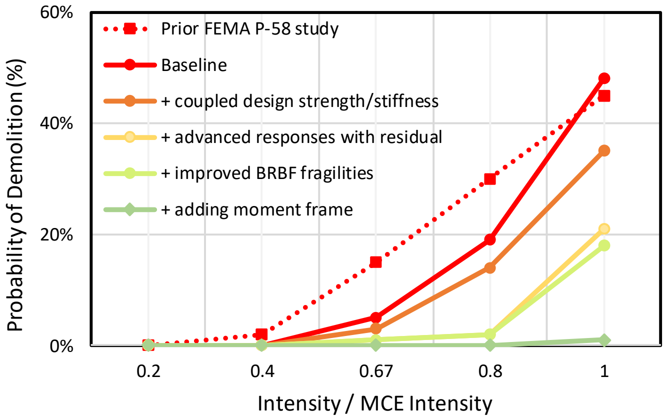

Figure 8 – Comparison of the probability of demolition from excessive residual drifts, for various levels of analysis refinement

Table 8 – Comparison of probabilities of demolition from residual drifts, for various levels of analysis refinement

Modeling Step for Probability of Demolition from Residual Drifts	**Performance at Each Intensity Level**				
	20% MCE	**40% MCE**	**67% MCE**	**80% MCE**	**100% MCE**
Prior FEMA P-58 study	0%	2%	15%	30%	45%
Baseline	0%	0%	5%	19%	48%
+ coupled design strength/stiffness	0%	0%	3%	14%	35%
+ advanced responses with residual	0%	0%	1%	2%	21%
+ improved BRBF fragilities	0%	0%	1%	2%	18%
+ adding moment-connected frame	0%	0%	0%	0%	1%

5. Summary and Conclusions

This paper presented the several research-based modeling improvements for risk analysis of BRBF buildings using the FEMA P-58 analysis method and shows how the risk assessment results change with the improved modeling. These included improvements to the analytical methods for residual drift modeling, brace fragility, and structural response modeling. The result of the more detailed analyses shows both good performance of BRBF buildings and that the prior approximate analyses published for BRBF buildings were highly conservative. The advanced methods show much lower repair costs, repair times, and probabilities of demolition from residual drifts. This substantially better predicted performance applies to new buildings with or without a moment-connected backup frame (and results are almost the same for design level shaking), but is maximized when a back-up frame is present in the building (with improved performance particularly for above-design MCE-level shaking).

6. Acknowledgements

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7 Copyrights

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