

Studies on the Design and Behavior of Strongback Braced Frames

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Abstract

When subjected to strong earthquake ground motions, conventional steel braced frames are vulnerable to soft-story mechanisms, whereby the weakest story accumulates more damage relative to the rest of the structure. This reduces the overall strength of the structure and increases the cost of repairs.

One method for mitigating this behavior is the use of a stiff vertical “spine” with a more ductile, energy-dissipating system. The spine typically spans the height of the structure and is designed to remain elastic, distributing earthquake demands across the height of the structure and bridging weak stories. One proposed frame is the “strongback” braced frame (SBF), which merges a steel buckling-restrained braced frame and an elastic truss, using the buckling-restrained braces for energy dissipation and the truss for force distribution.

Three studies are presented here on the behavior and design of strongback braced frames.

The first considers the effect of the strength and stiffness of the spine in SBFs, and how methods to design the spine vary. Designing the spine is challenging, as higher mode effects and partial nonlinear mechanisms have been shown to be significant. The results show that two proposed SBF-specific design methods provide value over a control procedure. Both SBF-specific methods produce designs with reserve strength in the strongback, but one is much more conservative.

The second considers the effect of the vertical distribution of the energy dissipators in the frame, and introduces a design method for the energy dissipators that supports the design of frames with non-standard dissipator configurations. The design method shows promising results, but the existing methods for design of the strongback members have inconsistent results when used with the non-standard dissipator configurations.

The third considers the mitigation of higher-mode effects by using multiple pivoting segments in the spine, and how passive control devices between the segments impact the behavior of the frame. The results show that multiple segments reduce demands on the strongback braces, but increase other demands such as maximum story drift, and the use of passive control devices allows finding a medium configuration between one- and two-segment spines, with benefits over both.

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CHAPTER 1

Introduction

Earthquakes have the potential to cause massive disruption. Historically, earthquakes have caused tremendous loss of life. To reduce the hazard to human life, conventional seismic design utilizes the energy dissipation provided by yielding of the structure's seismic force-resisting system (SFRS) to meet performance criteria. This can lead to large permanent deformations that render a building unusable after an earthquake. As the commentary to the seismic provisions of ASCE 7-22 (ASCE [2022](#)) states:

Structures designed in accordance with the standard are likely to have a low probability of collapse but may suffer serious structural damage if subjected to the risk-targeted maximum considered earthquake (MCE_R) or stronger ground motion.

Additionally, many classes of SFRS are susceptible to soft-story mechanisms, where a single story or group of stories either has initially lower stiffness or softens due to yielding or buckling. This lower stiffness compounds upon itself, potentially leading to runaway damage accumulation. Horizontal and vertical irregularities in a structure's SFRS can serve as initiators for soft-story mechanisms, but are not required for these mechanisms to form. Soft-story mechanisms can lead to instability and collapse, and reduce the ductility and survivability of the structure. Steel braced frames are an example of systems that are susceptible to the formation of soft stories even in systems without irregularities, especially if the frame columns are not capable of supporting the story after brace buckling (MacRae et al. [2004](#); Pollino et al. [2017](#)).

Rocking and pivoting spine systems, for example the strongback-braced frame (SBF), are a class of SFRS that seek to reduce overall damage, prevent soft-story mechanisms, and reduce the need to demolish buildings after severe earthquakes. SBFs evolved from buckling-restrained braced frames and zipper frames, and were proposed in their current form by Lai and Mahin ([2014](#)), and studied in detail by Simpson ([2018](#)). Among the key takeaways from these studies was the observation that demands on the spine due to higher-mode effects are significant. Designing for these demands is challenging, and open questions on the behavior and design of SBFs remain. For example, it is not known how to efficiently design SBFs in order to best exploit

the advantages that the strongback provides. A better understanding is needed of the behavior of strongback systems and how to design them. Otherwise, these systems will not get incorporated into standards and will remain underutilized.

1.1 State of the Art

Steel rocking and pivoting spine systems bring to bear ideas developed from long-standing research into shear walls and concrete rocking walls. These systems use a stiff, elastic spine to distribute damage across the height of the structure, preventing it from accumulating in a single story. The spine, being intended to remain elastic, is not designed to be capable of significant energy dissipation on its own, and so must be paired with some other mechanism for doing so. These mechanisms vary greatly, but most commonly use some form of “fuse”, such as buckling-restrained braces (BRBs), “butterfly” fuses (Eatherton et al. 2014), or force-limiting connections (Tsampras et al. 2016; Fahnestock et al. 2022).

One particular pivoting spine system is the strongback braced frame (SBF), which combines energy-dissipating components—typically buckling-restrained braces—and a stiff elastic spine within standard braced frame geometry (Lai and Mahin 2014). This arrangement potentially makes SBFs particularly suited for use in retrofitting older steel braced frame structures (Lai 2020; Simpson and Mahin 2018b). A schematic of an SBF is shown in Fig. 1.1.

SBFs provide many of the benefits of other spine systems within conventional braced frame geometry and without requiring novel connections; even the pin at the base of the spine can be realized without special detailing by orienting the strongback column in the weak-axis direction within the plane of the SBF (Simpson and Mahin 2018b).

1.2 Gaps

SBFs are promising systems for improving steel braced frames, but there remain several gaps in knowledge that must be addressed. In this dissertation, three of these gaps are pursued: (1) the performance effects of strongback deformations and inelasticity, and the effect of the stiffness and strength of the strongback itself; (2) how the vertical distribution of the energy-dissipating resistance affects system performance, and how

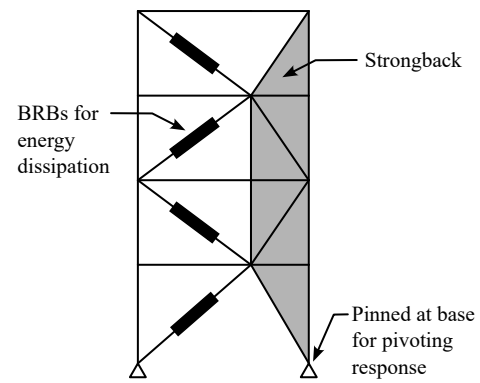


Figure 1.1: Strongback braced frame. Shaded region indicates the members comprising the strongback

to design energy-dissipating members for various vertical distributions; and (3) the effect of a segmented spine on SBF performance, and if the gap between segments can be modified with a tuned joint to improve performance. These gaps, along with their motivating research questions, are described in the following subsections.

1.2.1 Performance Effects of Strongback Deformations and Inelasticity

How does the overall performance of the structural system vary with the strength and stiffness of the strongback?

The amount of stiffness and strength necessary to keep the strongback elastic and meet performance criteria are uncertain. Additionally, allowing limited yielding in the strongback could reduce demands associated with higher modes, while still retaining the benefits to overall performance obtained from including a strongback in the first place.

SBFs and other spine systems reduce maximum deformations by distributing demands across the height of the structure. As the stiffness of the spine increases, the deformations approach a uniform distribution. The exact stiffness required to achieve this effect is unknown. Simpson (2018) performed some preliminary analysis and determined for their 4-story structure a particular ratio of strongback brace size to BRB size but those results may not hold for other structure configurations.

The strongback has typically been considered as having a design requirement to remain completely elastic. This may not be necessary, however. Limited yielding could be allowed while still achieving acceptable seismic response. Significant loss of stiffness due to partial yielding or buckling of braces should be prevented, but not all elements of the spine are as critical to its operation. Allowing yielding of strongback components would also require establishing ductility requirements, such as those in the AISC *Seismic Provisions* (AISC 2022).

1.2.2 Distribution of Energy-Dissipating Resistance

How does the vertical distribution of the energy-dissipating resistance affect overall system performance?

The ability of the strongback to distribute forces and deformations across the height of the structure allows consideration of configurations where energy-dissipating resistance is not provided at every story. This opens up an enormous design space to examine.

One of the purported benefits of strongback systems is the flexibility they provide to designers in the placement of the energy dissipating resistance within the structure (Simpson and Mahin 2018a). This has been utilized in practice to enable the use of uniformly-sized BRBs at each level, instead of the sizes needing to be scaled to the vertical distribution of demand (Panian et al. 2015). Simpson (2018) found no significant difference in response between a 4-story SBF with proportioned BRBs and one with uniform BRBs.

Extending this further, however, the effectiveness of the strongback at bridging gaps in the vertical energy-dissipation resistance can be examined. Braced frames in general provide stiffness and strength more efficiently than, for example, moment frames, but may be avoided in practice as a solution due to architectural or other project constraints. Strongbacks have been shown experimentally to bridge failures and continue to engage the reserve strength of the system (Simpson and Mahin 2018b); this capability can be utilized to re-arrange the primary lateral resistance of the system to accommodate project constraints.

1.2.3 Tuned Joints for Segmented Strongbacks

Can segmented strongback systems be modified with tuned joints such that they have more robust seismic performance?

Splitting the spine into multiple segments reduces the impact of the higher-mode effects that so frequently control the design of spine systems (Simpson 2020; Wiebe and Christopoulos 2009). The joints between these spines could be modified to include energy-dissipating elements or passive control devices.

The design of strongbacks and other spines is often controlled by higher-mode responses (Simpson 2020). A proposed method for reducing higher-mode demands in spines is to separate the spine into multiple sections (Wiebe and Christopoulos 2009). This approach has not been studied for SBFs. The connections between the sections—i.e., the joints—also represent an opportunity to tune the structure’s response.

1.3 Scope

The objective of this dissertation is to answer the research questions identified above through three studies.

The first study evaluates two proposed design procedures for the design of the strongback: the simplified modal pushover analysis (SMPA) and generalized modified modal superposition (GMMS), with a “control” procedure based on current standardized capacity design procedures. Nonlinear response history analyses were performed to evaluate the differences in behavior resulting from the different design methods. To determine the effect of the strength and stiffness of the strongback, the yield strength and elastic modulus of the strongback members were varied and the analyses repeated. The results show that the GMMS and SMPA design procedures are generally well-calibrated and provide benefit over current standardized procedures in several ways: collapse performance is improved, and yielding in the strongback and residual drifts are reduced. The GMMS procedure results in larger members, but provides similar outcomes to the more-complicated-to-implement SMPA.

The second study introduces a design method for sizing general arrangements of BRBs in SBFs based on overturning moments. The design method is evaluated for its ability to produce consistent reliability for different arrangements of BRBs, and for appropriate values of a scale factor γ , required to determine the design overturning moment. The study also evaluates the feasibility of SBFs with missing BRBs. Three BRB configurations were evaluated: (1) the typical full complement of BRBs, one at each level (X); (2) a configuration where the first-story BRB is removed (R); and (3) a configuration where the only BRB is a vertically-oriented “single energy dissipator” at the first story (S). A parametric study of configurations was performed using a simplified model with a rigid strongback to do a broader investigation. Then a more concentrated study was performed using a nonlinear frame-level model to investigate the behavior of 4-story SBFs with various BRB configurations. The results show that for a rigid strongback, removal of the first-story brace is workable, but a single energy dissipator configuration would require unrealistically large BRBs. Additionally, the SMPA method used for designing the spine does not generate strongbacks of adequate strength and stiffness when used with the R or S configurations.

The third study investigates the impact of segmenting the strongback, as well as the impact of passive control devices placed between the segments of the strongback. A seven-story continuous strongback was designed using the slightly conservative GMMS procedure. The frame was then evaluated using nonlinear response history analysis, with the fourth-story tie brace either in place or replaced with nothing, a hysteretic damper, or a rotational inertia viscous damper (a form of linear inerter). The dampers were evaluated for a

range of their respective constitutive parameters. The strongback itself was modeled with elastic elements. The results show that a two-segment strongback reduces peak story accelerations and axial demands in the strongback members, but increases peak story drift and story drift concentration. Adding a hysteretic damper with appropriate stiffness and strength can provide reduced accelerations and axial demands compared to the single-segment frame, while also reducing peak drifts and story drift concentration. Adding a linear inerter with appropriate damping and inertance can reduce peak accelerations compared to both the single-segment and two-segment frame, while also providing reductions to maximum drift and column moment demands compared to the two-segment frame alone.

The structure of the dissertation is as follows. Chapter 2 presents an overall literature review. Chapters 3, 4, and 5 present the three studies, each based on a submitted or anticipated manuscript for a journal article. The final chapter presents overall conclusions as well as recommendations for future work.

CHAPTER 2

Literature Review

2.1 Steel Braced Frames

Steel concentrically braced frames (CBF) are widely used as a lateral force resisting system due to their increased stiffness, relative economy, and ease of construction compared to moment frames (AISC 2018). However, they are especially vulnerable to story mechanisms during seismic events, and are generally less ductile than other steel lateral force-resisting systems. Stricter detailing requirements, such as those for special concentrically braced frames (SCBF), can improve ductility and reduce required brace sizes.

Historically, braced frames have primarily been constructed using single diagonal or chevron configurations. These configurations provide limited vertical continuity, and the chevron configuration especially can generate large unbalanced forces that must be resisted by the beam the braces are attached to (Khatib et al. 1988). Alternative configurations, such as the two-story X and zipper braced frames Fig. 2.1, provide improved vertical continuity and reductions to unbalanced forces, but are still vulnerable to story mechanisms, as they lack redundant paths for force redistribution.

When properly detailed, conventional steel braces can provide large amounts of energy-dissipating capacity. However, this is limited primarily to the yielding of the braces in tension, and thus CBFs must be designed so that braces act both in tension and compression (Fig. 2.2). To improve the compression behavior of braces, buckling-restrained braces (BRB) were developed. These braces most commonly comprise a steel core with a concrete casing, separated by an “unbonding” layer (Fig. 2.3). In compression, the steel core still buckles, but in a very high mode; combined with friction between the core and the casing, BRBs are typically stronger in compression than in tension, and the result is a stable hysteresis without the pinching behavior of conventional braces Fig. 2.4. Replacing the conventional braces in a CBF with BRBs results in the buckling-restrained braced frame (BRBF). BRBF have excellent ductility, and the balanced forces in tension and compression reduce the unbalanced forces on beams, but they are still susceptible to story mechanisms once the braces in a

single story have yielded.

2.2 Rocking and Pivoting Spine Systems

To alleviate the shortcomings of steel braced frames, especially the vulnerability to story mechanisms, a variety of “spine” frames have been studied. Spine frames are hybrid systems that combine a stiff “spine”, typically running the height of the building and designed to remain elastic during an earthquake, with ductile energy-dissipating components. The energy-dissipating components are intended to attract and absorb the earthquake demands, protecting the rest of the structure. The spine distributes seismic forces across the height of the building to the dissipators, bridging weak stories and preventing damage concentration. The dissipators are often, though not necessarily, made to be easily replaceable, as a sort of “fuse” that can be swapped out after a major earthquake (Burton et al. 2016; Chen et al. 2017; Eatherton et al. 2014).

Self-centering, such as through post-tensioned cables (Blebo and Roke 2015; Eatherton et al. 2014; Wiebe and Christopoulos 2015b), an enveloping moment frame (Chen et al. 2017), or combination of systems (Chen et al. 2022) may be provided to reduce residual drifts.

Spine frames can be largely grouped into two categories, based on the behavior of the spine: rocking frames allow uplift of the base, while pivoting frames do not (Fig. 2.5). Some pivoting frames in the literature are referred to as rocking frames (Blebo and Roke 2015).

Rocking behavior was first studied in observations of water towers and other tall, slender, free-standing structures after earthquakes in the 1950s and 60s. These observations showed that rocking behavior, often accompanied by large plastic deformations of foundation attachments such as anchor bolts, contributed to these structures having surprising survivability relative to more “robustly” designed structures (Housner 1956; Housner 1963). These observations led to design innovations in concrete shear walls (Priestley et al. 1978; Priestley et al. 1999) and studies of uplift in steel braced frames (Huckelbridge 1977). More recently, rocking behavior has been incorporated into the design of steel frames (Midorikawa et al. 2006; Roke et al. 2006). Deierlein et al. (2011) introduced the controlled rocking steel frame, which uses a replaceable “butterfly” fuse for energy dissipation; further work on the controlled rocking steel frame has included experimental testing (Ma et al. 2011) and development of design methods (Eatherton et al. 2014; Martin et al. 2019; Martin and Deierlein 2021).

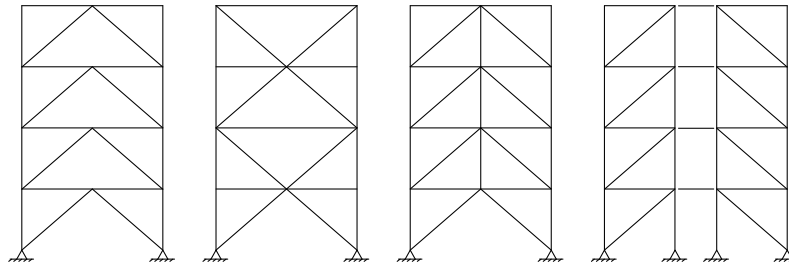


Figure 2.1: Typical concentrically braced frame configurations. Left to right: chevron/inverted V; two-story X; zipper braced frame; single diagonal (multiple bays)

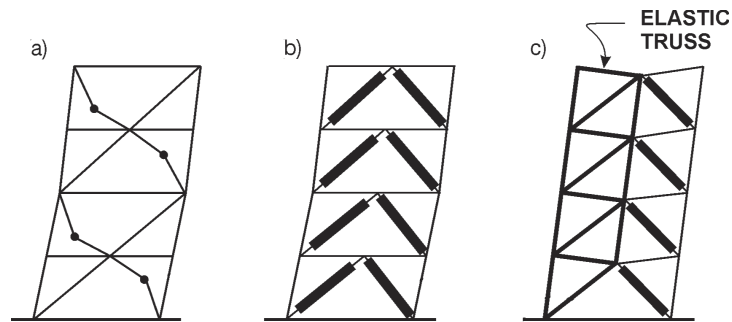


Figure 2.2: Deflected shape of various concentrically-braced frame systems. Left-to-right: conventional CBF, BRBF, strongback braced frame (Tremblay and Poncet 2007)

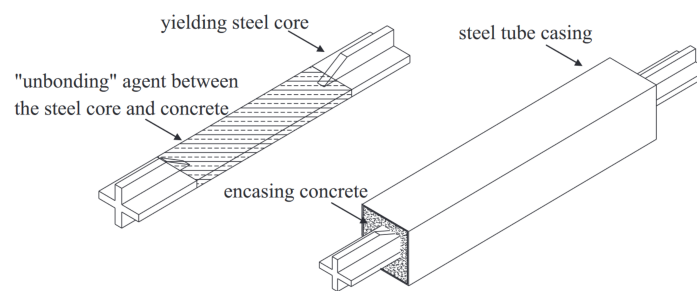


Figure 2.3: Components of a buckling-restrained brace (Talebi et al. 2014)

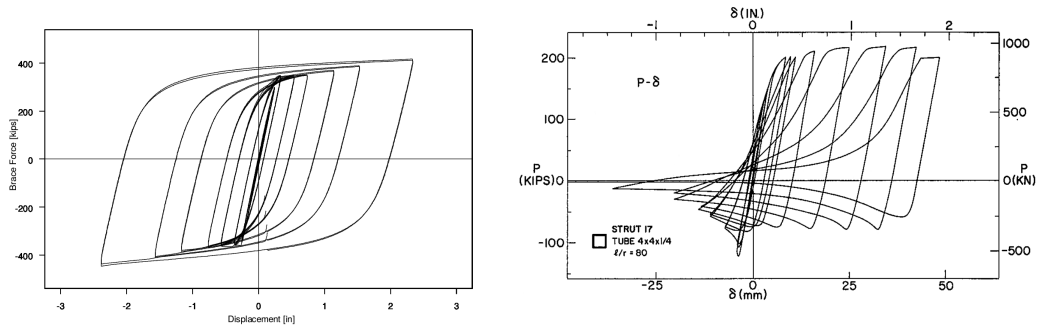


Figure 2.4: Hysteretic behavior of buckling-restrained brace (left) and conventional steel brace (right) (Clark et al. 2000; Popov and Black 1981)

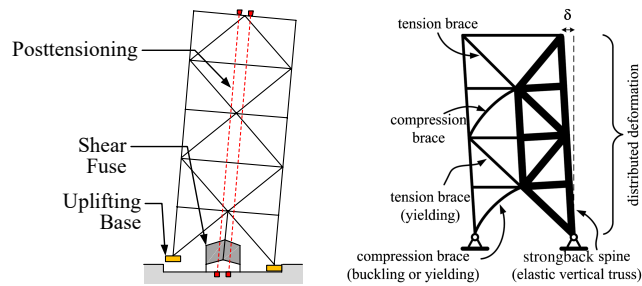


Figure 2.5: Rocking and pivoting behavior. Left: controlled rocking spine (Eatherton et al. 2014). Right: strongback braced frame with conventional steel braces (Lai and Mahin 2014)

Pivoting spine systems differ from rocking systems in having no uplift at the base. Examples include elastic truss frames (Merzouq and Tremblay 2006; Tremblay 2003; Tremblay and Poncet 2007), hinged rocking walls (Alavi and Krawinkler 2004), hinged trusses (Blebo and Roke 2018; Palermo et al. 2021; Takeuchi et al. 2015), and strongback braced frames (Lai and Mahin 2014; Simpson 2018).

Higher mode demands in spine frames can be so significant, especially with increasing height, that it can be advantageous to break the spine up into multiple rocking or pivoting segments (Wiebe and Christopoulos 2009). Using multiple segments reduces the shear and moment demands on the spine by allowing the spine to comply with the higher modes, while still providing regions of vertical continuity. The segments themselves can be connected by energy-dissipating components, taking advantage of the newly multiplied sites of rocking and pivoting actions (Eatherton et al. 2014; Wiebe and Christopoulos 2009).

2.3 Strongback Braced Frames

The strongback-braced frame (SBF) examined in this dissertation is a particular kind of pivoting spine system. A steel truss, pinned at the base, composes the spine (the eponymous “strongback”) and buckling-restrained braces (BRBs) form the ductile energy-dissipating system. Other energy-dissipating mechanisms, whether specially-detailed conventional braces (Lai and Mahin 2014) or components such as force-limiting connections (Tsampras et al. 2016) could be used in place of the BRBs, but BRBs are the primary component studied in this dissertation. The system has appeared under a number of different names. Tremblay (2003) refers to it as a “dual CBF”; Mar (2010), citing Tremblay, refers to it as a “trussed mast frame”. Lai and Mahin (2014) coined the term “strongback braced frame”, and cited it as an extension of zipper-braced frames (Khatib et al. 1988), tied eccentrically-braced frames, and elastic truss systems (Merzouq and Tremblay 2006; Tremblay 2003; Tremblay and Poncet 2007). The truss geometry of the strongback is particularly suited to the retrofitting of older braced frames (Simpson and Mahin 2018b).

Lai and Mahin (2014) examined the potential of the strongback to mitigate soft story behavior, using numerical analyses to compare concentrically-braced frames without strongbacks to those with strongbacks. Several frames were designed using the provisions of ASCE 7-05, with design of the strongback itself performed using overstrength load combinations for the primary braces and capacity-limited loads for the tie braces. Nonlinear pushover and dynamic analyses were performed to verify the design. They found that this

design methodology was sufficient to significantly improve the distribution of forces along the height of the structure, and that the strongback could, potentially, be a more economical system than a conventional SCBF.

Mar (2010) and Panian et al. (2015) present a case study of the design and construction of a four-story building utilizing SBF, with a particular eye towards the redundancy provided by the strongback itself. The objective of this focus was to avoid triggering the redundancy provisions of ASCE 7 that require a 30% increase in the applied seismic load. Design was accomplished using response spectrum analysis (RSA) and overstrength load combinations, using the same size of BRB at each level. Their analysis showed that the redundancy requirements were satisfied by the strongback, and reduced the total number and area of BRBs required for the structure.

A major study of SBF design and behavior was performed by Simpson (2018). The relevant portions are summarized as follows:

- They showed that an offset configuration, where the spine is only 1/4 to 1/3 the width of the bay instead of half, reduces inelastic demands on both the BRBs and the beams.
- Simpson's numerical model, especially the calibrated BRB model, forms the basis for the nonlinear "frame-level" models used in this dissertation.
- Simpson found that SBFs were particularly sensitive to variation between individual ground motions, with ground motions having high pseudo-accelerations corresponding to the SBF's higher modes and ground motions with long durations being more demanding.
- For the four-story SBF studied, the vertical distribution of BRBs was not found to be a significant factor on the system's response, allowing use of a single BRB size. To determine this single BRB size, Simpson suggests sizing the BRBs at each level to resist 80% of the base shear.
- The demands on the strongback are not limited by yielding of the BRBs. Higher mode forces are resisted by elastic bending action of the strongback, and because it remains elastic those forces are not capacity-limited.
- Simplified modal pushover analysis was introduced, and developed further in later work (Simpson and Rivera Torres 2021).

Palermo et al. (2018) investigated the response and distribution of forces in spine frames where the strongback is not integrated into the primary frame. The strongback was idealized as a rigid column, pinned at the base. They found that for “shear type” buildings—e.g., braced frames, or moment frames with very stiff beams—the story shear demands on the primary frame were uniformly distributed, while the strongback was subject to large moment demands at the mid-height of the structure. They also found that the axial demand in the links that couple the strongback to the primary frame was highest at the top floor.

Lin et al. (2019) evaluated different stiffness configurations of strongbacks and their impact on the response of a simplified model. They found that strongbacks were more effective at reducing story drift concentrations in low- to mid-rise buildings than high-rise buildings.

Faramarzi and Taghikhany (2020) and Faramarzi and Taghikhany (2021) studied the use of performance-based design procedures for the design of SBFs. They found that SBFs using conventional buckling braces had superior ductility to conventional CBFs designed to the same performance criteria.

Soleymani and Saffari (2024) studied SBFs where the energy-dissipating component is a shear link similar to those found in eccentrically-braced frames, and compared the behavior to SBFs with conventional buckling braces for the energy-dissipating component. They found that the SBFs using shear links had superior seismic performance to those with conventional braces.

Limited data is available on the cost of repair and expected downtime due to earthquake-induced damage for SBFs and spine frames. One study (Rhoades 2021), using the FEMA P-58 methodology (FEMA 2018), compared the post-earthquake expected damage and cost of repair of buckling-restrained braced frames (BRBF) to SBFs. They found that the cost of non-structural damage was generally higher in SBFs, but BRBFs were significantly more likely to need to be replaced entirely.

2.4 Seismic Design

Seismic design has traditionally focused solely on the “life safety” limit state, where the building is designed to accommodate significant permanent deformation of the seismic force-resisting system. This increases the energy dissipation provided by the structure while allowing occupants to escape the structure safely (AISC 2018). For critical infrastructure (such as hospitals and fire stations) and structures whose collapse would pose a hazard to the surrounding community (such as industrial facilities handling hazardous chemicals),

more stringent limit states are considered.

Most seismic design for buildings in the United States follows one of the methods defined within *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, commonly referred to as ASCE 7 (ASCE 2022). The latest version, ASCE 7-22, provides four analysis methods, roughly in order of increasing complexity: the equivalent lateral force (ELF) procedure, modal response spectrum analysis (MRSA), linear response history analysis (LRHA), and nonlinear response history analysis (NRHA). Most relevant to this work are the ELF and MRSA procedures, as they form the basis for design methods modified for the challenges posed by designing strongback braced frames. NRHA can be used as-is for design of SBFs, as it directly models the nonlinear responses, but using it for design is impractical for many design firms due to its requirements of expertise, time, and peer review.

2.4.1 Equivalent Lateral Force Procedure

The most commonly used seismic design method in ASCE 7-22 is the equivalent lateral force (ELF) procedure. The ELF procedure provides a means to capture the inelastic dynamic behavior of a structure through an elastic static analysis. It can be easily checked with hand calculations and does not require the selection of ground motions. Earthquake intensity is determined using site-specific accelerations and soil classification. Inelastic effects are accounted for using three seismic performance factors that are dependent on the specific SFRS being used: the response modification coefficient R , deflection amplification factor C_d , and overstrength factor Ω_0 . When originally introduced, these factors were determined “based largely on judgment and qualitative comparisons with the known response capabilities of relatively few seismic-force-resisting systems” (FEMA 2009). Even with the introduction of quantitative methods for determining these factors such as FEMA P695 (FEMA 2009), some degree of engineering judgment remains in the adoption of specific factors into consensus documents—for example, FEMA P695 recommends that C_d be taken as equal to R , but this recommendation is almost universally not followed in ASCE 7-22.

The response modification coefficient, R , reduces design loads. This reflects the limiting of the force demand on the structure due to yielding of the SFRS. More ductile systems have higher R factors, leading to increased energy dissipation and reduced loads, but also have higher deflections. These deflections are captured by the deflection amplification factor, C_d , which directly multiplies the deflections calculated from

the elastic analysis. The overstrength factor, Ω_0 , is used to design protected elements by amplifying the demands calculated using R .

ELF predominantly captures the effects of first-mode vibration. Prior to ASCE 7-22, other design procedures were required to be used for taller buildings, and buildings in regions with more significant seismic hazards. This restriction was lifted in ASCE 7-22, as ELF was seen as providing more consistent results than modal response spectrum analysis, but there remain situations that ELF does not sufficiently account for the dynamic demands from the earthquake (Martin et al. [2019](#); Steele and Wiebe [2016](#)).

2.4.2 Modal Response Spectrum Analysis

The ASCE 7-22 modal response spectrum analysis (MRSA) procedure uses linear dynamic analysis to determine the response of the structure in each relevant mode of vibration. MRSA accounts for nonlinear behavior using the same seismic performance factors as ELF, and applies them in each mode of vibration equally. Demands are typically combined using the square root of the sum of the squares (SRSS) method or the complete quadratic combination (CQC) method (for 2D analysis, SRSS and CQC are equivalent). This approach provides better estimates of structural demand when higher mode behavior is an important factor, but assumes that energy dissipation is associated uniformly with all modes of vibration, an assumption that does not hold true for SBFs due to the elastic strongback response (Simpson [2020](#); Steele and Wiebe [2016](#)).

2.4.3 Design of Protected Elements

To preserve the stability of the system while dissipating energy, certain components of a seismic force-resisting system are designated as protected elements. These elements must remain “essentially elastic” while the energy-dissipating components are activated. The simplest method to achieve this is by using the overstrength factor Ω_0 to amplify the applied loads. Alternatively, the protected elements can be designed specifically to resist the capacity-limited load effect resulting from the energy-dissipating components. In this approach, called “capacity design”, information about the expected strengths (as opposed to nominal or specified minimum) of materials and components is combined with knowledge of how the system will deform to generate appropriate load cases.

For steel braced frames, traditional capacity design only considers the fully-formed plastic mechanism of

the frame under a first-mode pattern of deformations. This approach is generally insufficient for spine systems and SBFs, as the demands on the spine are significantly affected by higher mode behavior and are not limited by the formation of the plastic mechanism (Simpson 2020).

2.4.4 Spine Systems

A significant obstacle to the uptake of spine systems is their combination of higher mode demands and force redistribution. The design methods available to engineers, standardized in building codes such as ASCE 7-22 (ASCE 2022) and seismic design standards such as AISC 341-22 (AISC 2022), do not specifically address these behaviors, leading to underprediction of demands on the spine (Simpson 2020). The exception to this is the use of nonlinear response history analysis (NRHA) to design the building, which is frequently cost-prohibitive in terms of time and expertise required to properly implement. Several alternative design procedures which are capable of modeling spine system behavior without resorting to NRHA have been proposed. These range from analytical estimations of the story shear and overturning moment (Wiebe and Christopoulos 2015c), to modal analysis procedures (Bosco et al. 2018; Martin and Deierlein 2021; Roke et al. 2009; Steele and Wiebe 2016), to nonlinear static analysis of the frame (Simpson and Rivera Torres 2021).

No consensus design method for spine systems in general or strongback braced frames in specific exists, and seismic performance factors have not been established. Previous studies have used $R = 6$ (Martin and Deierlein 2021), $R = 7$ (Panian et al. 2015), and $R = 8$ (Simpson 2018).

Proposed methods of design for SBFs generally separate sizing the energy-dissipating elements from sizing the protected elements. For designing the BRBs in the system, it has generally been recommended to use ELF to determine force demands. For SBFs, Simpson (2018) found that sizing BRBs to ELF demands did not result in a significant difference from using a single BRB size at each level, and suggests sizing the BRBs to 80% of the ELF base shear.

CHAPTER 3

Evaluation of Methods of Design for Strongback Braced Frames

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3.1 Introduction

Modern seismic design provisions focus on preventing collapse and ensuring the survival of the occupants of a structure during an earthquake. These minimum goals, however, do not ensure that structures are able to be used after a significant seismic event. As a result, the structure may not be useable for habitation or other vital services, and can require significant repairs or even complete demolition. More resilient structures, ones which have reduced risk of collapse and experience less structural damage, can reduce the downtime after a seismic event and reduce long-term costs associated with repair and refit (Bruneau and Reinhorn 2006).

Important structures, such as the hospitals and fire stations assigned to ASCE 7-22 Risk Category IV, can, in effect, be designed to be more resilient by increasing the design-level demands but otherwise following a design procedure based predominantly on collapse prevention (ASCE 2022). A more explicit evaluation of resilience can be made through performance-based design, such as the procedures available in ASCE 41-17 or Chapter 16 of ASCE 7-22 (ASCE 2017; ASCE 2022). Some seismic force resisting systems, that are specifically intended to reduce damage during an earthquake, can result in buildings that are intrinsically more resilient.

Spine systems are a class of seismic force resisting-systems intended to reduce overall structural damage during an earthquake by preventing the accrual of damage in a single story or set of stories. In these systems, an elastic spine runs the entire height of the structure, providing vertical continuity between all levels and energy-dissipating components. Examples of spine systems include steel rocking frames (Eatherton et al. 2014; Roke 2010), concrete rocking walls (Kurama et al. 1999; Priestley et al. 1978; Priestley et al. 1999) and strongback braced frames (Lai and Mahin 2014; Simpson 2018; Tremblay 2003).

Strongback braced frames (SBF) are a variation on concentrically braced frames and buckling-restrained braced frames, using one half of the frame as the energy-dissipating component and the other half as the spine (here termed the strongback). An example SBF is shown in Fig. 3.1.

The most common energy-dissipating component proposed for SBFs are buckling-restrained braces (BRB), though other components such as conventional braces and viscous dampers have also been studied (Abolghasemi et al. 2024; Lai and Mahin 2014; Palermo et al. 2021). In this work, BRBs are used to provide both the primary lateral resistance and the primary energy dissipation capacity of the frame.

The spine in SBFs is configured as a steel truss, with tie braces to provide vertical continuity between levels in the spine. Under severe ground motions, the spine is intended to remain elastic and pivot about its base while the BRBs yield, imposing a first-mode deformation pattern (Lai and Mahin 2014). As the full yielding mechanism develops, demands are transferred between stories through the spine, allowing the BRBs at all levels to participate concurrently, and preventing runaway damage concentrations caused by the loss of stiffness due to yielding. With a strong and stiff enough spine, SBFs allow full development of the BRB hysteresis and significant energy dissipation at every level, while preventing accumulation of damage in a single story.

The remaining beams and columns in the frame serve primarily to support gravity loads and complete the load path of the seismic force-resisting system. The beams also serve as a secondary source of energy dissipation, through plastic hinging at the face of the strongback (Simpson 2018). The opposing column is not expected to see significant inelastic demands. The connections throughout the frame are standard braced frame connections. Within the strongback itself, the connections are expected to experience less ductility demand than those in a conventional braced frame, due to the strongback remaining elastic.

Designing a strongback braced frame to achieve its intended behavior can be challenging. Current linear seismic design methods (ASCE 2022) and traditional capacity design (AISC 2016a) do not sufficiently account for the demands on the strongback. As the spine is intended to remain elastic and distribute demand after the buckling-restrained braces yield, (1) demands on the spine are not limited by yielding and (2) a significant proportion of the demand is associated with higher mode response (Gioiella et al. 2018; Simpson 2020). The higher mode response can be mitigated by modifications to the frame, such as by using multiple rocking or pivoting segments in the spine (Broujerdian and Mohammadi Dehcheshmeh 2022; Wiebe and Christopoulos 2009). The beams are also subject to significant rotational demand at the face of the strongback. It has been suggested that the beam rotations at the face of the strongback be designed as displacement-controlled actions, thus incorporating them directly into the energy dissipation system by allowing them to yield, with their size

limited either by explicitly evaluated displacement limits or implicitly through force reduction (Simpson and Mahin 2018b).

While it is possible to evaluate the demands on the strongback using nonlinear dynamic analysis, this requires significant time, resources, and expertise, which may not be available to most design firms. Thus, it is strongly desirable to have a simpler method for determining demands on the spine. Several simplified methods have been proposed for the design of SBFs in specific and spine frames in general, varying from analytical estimations of the story shear and overturning moment (Wiebe and Christopoulos 2015a) to more complicated modal analyses (Bosco et al. 2018; Martin and Deierlein 2021; Roke et al. 2009; Steele and Wiebe 2016), to nonlinear analysis of the frame (Simpson and Rivera Torres 2021). Investigated here are the simplified modal pushover analysis (SMPA) procedure (Simpson and Rivera Torres 2021) and the generalized modified modal superposition (GMMS) procedure (Martin and Deierlein 2021).

The SMPA procedure (Simpson and Rivera Torres 2021), developed as a simplification of modal pushover analysis (Chopra and Goel 2002), explicitly considers the plastic mechanism of the SBF. A displacement-controlled analysis is used in the first mode, and load-controlled analyses up to a per-mode base shear are used in the higher modes. There are several options for modeling of the plastic behavior, ranging from highly detailed asymmetric hysteretic response to neglecting nonlinear behavior entirely. Simpson and Rivera Torres (2021) recommend using elastic-perfectly-plastic materials for the BRBs in the first mode and elastic materials in the higher modes, with post-analysis checks to ensure that behavior remains within the design strength range.

The GMMS procedure (Martin and Deierlein 2021) is simpler than the SMPA procedure, as no inelastic analysis is performed. GMMS was developed as an extension of the modified modal superposition procedure (Martin et al. 2019; Priestley and Amaris 2003) to steel rocking and pivoting spine frames, and is similar to other modal response-spectrum based procedures (Roke et al. 2009; Steele and Wiebe 2016). Unlike SMPA, GMMS does not provide information about the design of displacement-controlled components in the frame (i.e., the members and connections designed to experience inelasticity), and is instead solely used for the design of the spine and other force-controlled components (i.e., the members and connections expected to remain essentially elastic). It was developed and tested primarily on rocking frames and is less well-defined for strongback braced frames. The procedure uses an equivalent single-degree-of-freedom system to estimate the

system stiffness at the maximum considered earthquake (MCE), and replaces the energy-dissipating elements with equivalent stiffness springs. These springs are sized to provide the same system overturning resistance at MCE, with stiffness kept proportional to the stiffness of the elements they are replacing.

Of these two approaches, it is unknown which provides the best required strengths for design. It also remains unclear precisely how stiff and strong the strongback must be to enforce the desired behavior. Neither the SMPA nor the GMMS procedure includes an explicit check to ensure the strongback enforces near-uniform drifts. Furthermore, while the spine is generally specified to remain elastic to avoid damage to it during a seismic event, its members are designed using the AISC Specification (AISC 2016b) and AISC Seismic Provisions (AISC 2016a), which provide no guarantee of truly elastic response.

The objectives of this work are to (1) determine how the performance of the frames produced by the design methods compare to one another; and (2) determine how the performance varies when the strength and stiffness of the strongback are varied. These objectives were achieved by designing frames according to both SMPA and GMMS, and comparing them to frames designed by standard linear response spectrum analysis according to ASCE 7-22 (ASCE 2022) with capacity design by AISC 341-16 (AISC 2016a). The frames were analyzed using nonlinear pushover, response history, and incremental dynamic analyses, with both nominal strength and stiffness as well as a range of modified strength and stiffness values for strongback members.

3.2 Methods

3.2.1 Building Configurations

Three building configurations of varying heights (2, 4, and 8 stories), all using the same floor plan shown in Fig. 3.2, were investigated in this work. Elevation views are shown in Fig. 3.3. The seismic force resisting system consists of four total SBFs, two in each direction. The SBFs have a strongback occupying 1/3 the width of the bay; this configuration was identified by Simpson (2018) as reducing excessive inelastic demand on the beams and BRBs, without making the strongback too narrow. Each configuration was designed using the three design procedures, for a total of nine frame designs. The four SBFs and their tributary mass are the same regardless of orientation, so only one was designed and analyzed.

Gravity loads on the building were determined based on an assumed office occupancy, and are listed in

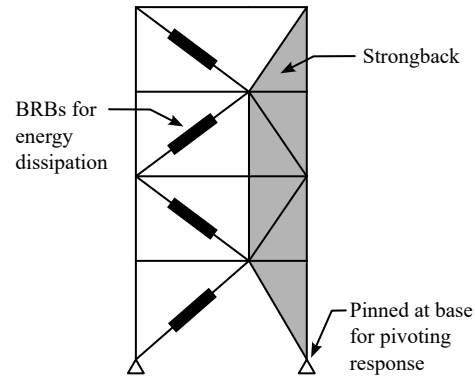


Figure 3.1: Strongback braced frame. Shaded region indicates the members comprising the strongback

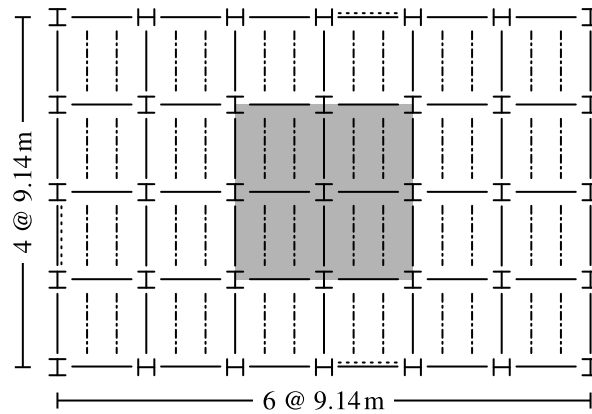


Figure 3.2: Floor plan. Dashed lines indicate location of SBFs. Shaded area indicates location of penthouse. Dash-dotted lines indicate location of secondary beams.

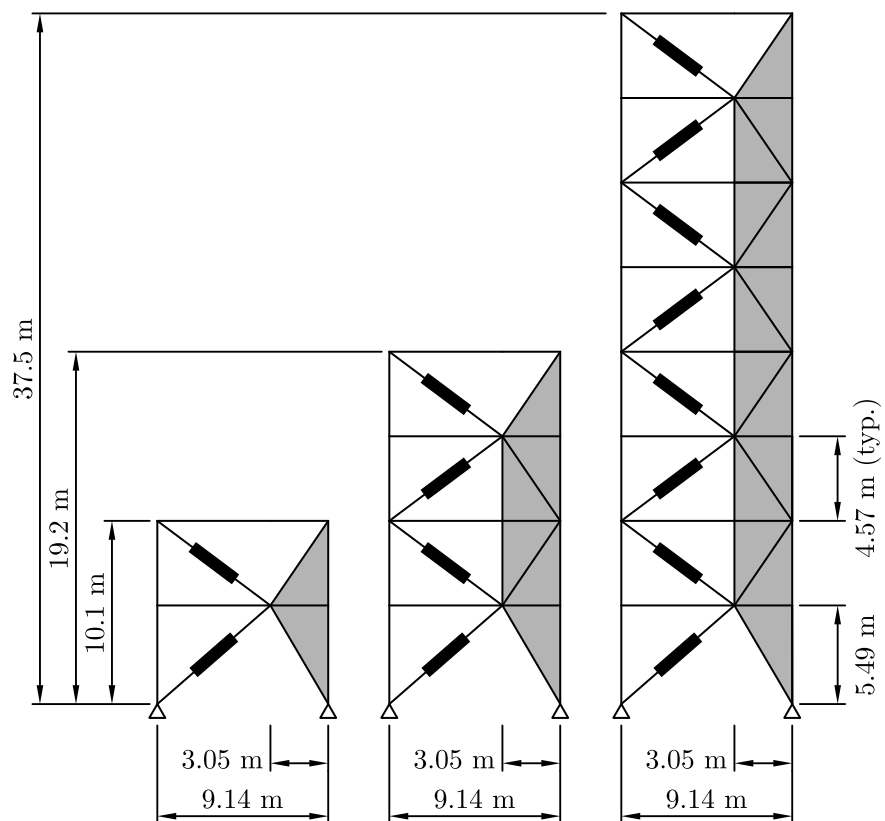


Figure 3.3: Elevation view of SBFs

Table 3.1. Member self-weight is included in the dead load. Loads from the penthouse are included in the roof loads. Seismic weight, also shown in Table 3.1, is based on the dead load and a 10 psf partition load. No snow or wind loads were included in the design. Seismic hazard parameters were taken from the FEMA P695 seismic design category $D_{\max}$ (FEMA 2009); the design-level response spectrum for seismic design category $D_{\max}$ is shown in Fig. 3.4. The buildings were assigned Risk Category II, for an importance factor $I_e = 1.0$ (ASCE 2022). Eccentricity due to accidental torsion was ignored.

3.2.2 Frame Design

The strongback braced frames for each of the three building configurations were designed following the procedures described in this section. These procedures were developed based primarily on current standards for design, such as ASCE 7-22 and AISC 360-16, and recommended approaches for design of spine frames, specifically, simplified modal pushover analysis (SMPA) and generalized modified modal superposition (GMMS). In some instances, additional decisions had to be made to cover cases not covered by current standards or recommended procedures, such as how to handle modeling the deformation-controlled beams in the equivalent linear model used by the GMMS procedure. These cases are described as they arise in the following sections.

The general process started with sizing the energy-dissipating components (i.e., the BRBs and the beams). Then, for trial designs, member strength evaluation was performed for non-capacity and capacity load combinations. Member sizes were selected through an iterative process to achieve least weight within the constraints. The selection of member sizes was manual, and a formal optimization was not performed. Seismic drift limits and stability coefficient checks from ASCE 7-22 (ASCE 2022) were also performed.

The frames consist of ASTM A992 wide-flange members with specified minimum yield strength $F_y = 50$ ksi and expected yield strength $R_y F_y = 55$ ksi for beams, columns, and conventional braces. The buckling-restrained braces have a specified yield strength of 42 ksi; stiffness factor $Q = 1.4$, which accounts for the additional stiffness due to the end regions; and hardening parameters $\beta = 1.2$ and $\omega = 1.46$, which are used to determine the capacity-limited force from the BRB. The BRB hardening parameters are the same as used for the calibrated BRB model described by Simpson (2018). For design purposes, column splices were assumed to occur every two stories, but were not directly modeled in the analyses.

Table 3.1: Gravity loads per floor. Roof loads include the penthouse load. For 2-story buildings, use “1st floor” and “Roof”.

Load Type	1 st Floor [kip]	Typ. Floor [kip]	Roof [kip]
Dead	1976	1953	1868
Live (reducible)	1728	1728	360
Live (unreducible)	324	324	—
Roof live	—	—	18.3
Seismic weight	2192	2169	2084

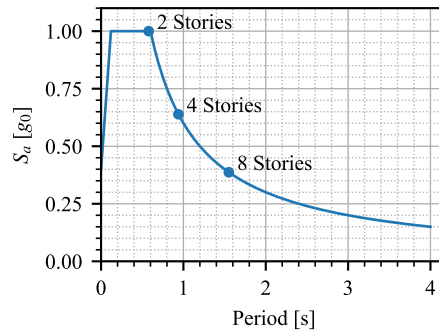


Figure 3.4: FEMA P695 $D_{\max}$ design spectrum. Markers indicate the location of the buildings on the design spectrum, using the approximate fundamental period T .

All analysis models used in this work, including those for design, were developed using OpenSees v3.4.0 (McKenna et al. 2010). Only the overview of each procedure and selected design variables are described here; full details are available online at <https://doi.org/10.17603/ds2-jrcm-2c58>.

Strength design

The initial phase of design consisted of sizing the energy-dissipating components of the frames—the core area A_{sc} of the BRBs and the member size of the beams—based on the equivalent lateral force (ELF) base shear.

The ELF base shear V was calculated using $R = 8$ (Simpson 2018). The approximate fundamental period T was calculated according to Eq. (3.1) following the guidelines from FEMA P695 (FEMA 2009):

$$T = \min (C_u C_t h_n^x, 0.25 \text{ s}) \quad (3.1)$$

where C_u is the coefficient for the upper limit on calculated period from ASCE 7-22, equal to 1.4 for seismic design category $D_{\max}$ (ASCE 2022); C_t and x are approximate period coefficients from ASCE 7-22; and h_n is the ground-to-level height of the roof in feet. For the strongback braced frames, values of $C_t = 0.03$ and $x = 0.75$ were selected due to the similarity to buckling-restrained braced frames.

To size the energy-dissipating components, the cross-sectional area of the BRB core was selected to be the same at each story. While standard braced frame design procedures call for BRBs to be sized proportional to the first-mode story shear profile, the elastic spine in SBFs offers the opportunity to use a single BRB size, simplifying construction and taking advantage of the ability of the strongback to redistribute internal forces (Simpson 2018). Assuming the strongback is stiff and strong enough to engage all levels of the structure fully, a uniform story drift profile is imposed, with corresponding uniform strain demands at all levels. Using the recommendation from Simpson (2018), the BRBs at each level were sized for a constant shear demand at each level equal to 80% of the ELF base shear. Since the first story is taller than the others, and thus the resultant axial force in the BRB is different, this method results in a different required steel core area $A_{sc,r}$ for different story heights. The average of the required area over the stories was used instead of the required area at each story to keep the BRB core area the same at each level. The actual core area sizes were selected to be the next 0.5 in² above the required area; these design values are listed in Table 3.2. Base shears shown in Table 3.2 are for the individual frames and not for the building as a whole.

Available member strengths for the force-controlled elements (columns and conventional braces) were determined using the effective length method with effective length factor, $K = 1.0$ (AISC 2016b). Axial, flexure, and axial-flexure interaction limit states were considered, according to AISC Specification Chapters E, F, and Section H1.1, respectively (AISC 2016b). Initial required strengths and sizes were determined from elastic second-order analysis under ELF loads, with final required strengths and sizes from the individual design procedures.

For non-capacity-limited load combinations, required strengths were determined using ASCE 7-22 Section 2.3.6 basic load combinations (6) and (7):

$$\begin{aligned} 1.2D + 0.5L + E_v + E_h \\ 0.9D - E_v + E_h \end{aligned} \tag{3.2}$$

where D is the dead load, L is the live load, E_v is the vertical earthquake load equal to $0.2S_{DS}D$, and E_h is the horizontal earthquake load, as determined by the specific analysis procedure. The gravity load portion of these load combinations was always directly included in the analysis. Load combination (6) usually calls for a 1.0 live load factor, but the use of 0.5 is permitted for distributed live loads less than or equal to 100 psf. Live load reduction was calculated in accordance with ASCE 7-22 (ASCE 2022) and applied to the model using the technique described by Ziemian and McGuire (1992). AISC 360-16 notional loads were included as 0.002 times the gravity load.

A two-dimensional second-order elastic model was used for determining ELF drifts and required strengths. The model consists of centerline beam-column elements with elastic fiber sections (to ensure that elastic properties remained the same between the ELF model and the models described later incorporating material nonlinearity). The BRBs were modeled with truss elements, with amplified elastic modulus QE_s .

The frames were also designed for capacity-limited demands and higher mode dynamic effects, using three different procedures to evaluate the differences between the procedures. The three procedures are (1) simplified modal pushover analysis, or SMPA (Simpson and Rivera Torres 2021); (2) generalized modified modal superposition, or GMMS (Martin and Deierlein 2021); and (3) linear response spectrum analysis according to ASCE 7-22 with capacity analysis by AISC 341-16, or RSAC (AISC 2016a; ASCE 2022).

Simplified modal pushover analysis

The SMPA procedure determines the forces on the spine through a combination of a displacement-controlled pushover analysis in mode 1 and load-controlled static analyses in higher modes.

Three models are required for the SMPA procedure: a pushover model with expected material strengths (used in mode 1), a pushover model with specified minimum material strengths (used in the higher modes), and an elastic design model (used for design criteria required to be determined by the equivalent lateral force procedure).

The pushover models used for SMPA, shown schematically in Fig. 3.5, are based on the numerical model developed by Simpson (2018), incorporating simplifications described by Simpson and Rivera Torres (2021) for the “PP” configuration, where force-controlled actions are modeled as elastic responses, and displacement-controlled actions are modeled as elastic perfectly-plastic. The model uses centerline force-based beam-column elements with distributed plasticity, using two elements per member and five Gauss-Lobatto integration points per element. The BRBs are modeled using corotational truss elements with area equal to A_{sc} and modified elastic modulus equal to QE_s . The beams are connected to the columns with rotational springs representing the shear tabs. The connections have a moment strength of $0.3M_p$ at “bare” shear tabs and $0.7M_p$ at shear tabs that are partially restrained by a brace gusset plate, where M_p is the plastic moment strength of the beam (AISC 2016b). The shear tabs are assumed to yield at a rotation of 0.005 radians. Rigid offsets representing connection regions were incorporated for all members based on member geometry and the location of brace gusset plates. As specific gusset plate sizes were not designed or detailed, offsets based on a percentage of the workpoint-to-workpoint length were used instead. At member ends where a gusset plate would restrain the rotation, a rigid offset equal to 15% of the member length was used. At member ends where no gusset plate is present, a rigid offset representing the physical offset from the workpoint due to the connecting member’s size was used (e.g., beams are offset from workpoints by half the column depth).

The SMPA procedure requires determination of target roof drifts, $\theta_{R,i}$, and target base shears, $V_{b,i}$, for each mode i up to a minimum of 95% modal mass participation. The first-mode displacement-controlled analysis only requires a target roof drift, while the force-controlled analyses in the higher modes use a target base shear as the primary target, with the target roof drift as a secondary limit indicating insufficient stiffness. The target values are determined from spectral displacements and eigenvalue analysis of the pushover model;

Table 3.2: Initial design values

Number of Stories	Approximate Period, T [s]	ELF Base Shear, V [kip]	Required BRB Size, $A_{sc,r}$ [in ²]	BRB Size, A_{sc} [in ²]
2	0.578	268	7.18	7.5
4	0.939	344	9.27	9.5
8	1.55	418	11.2	11.5

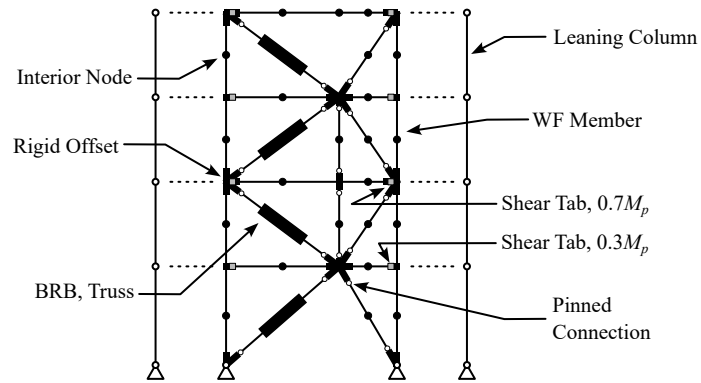


Figure 3.5: Schematic of pushover model for SMPA procedure

see (Simpson and Rivera Torres 2021) for details on calculation of $\theta_{R,i}$ and $V_{b,i}$. These values are listed in Table 3.3. Base shears shown in Table 3.3 are for individual frames and not for the building as a whole.

The modal combination for SMPA response to determine E_h is given by

$$E_h = \pm \left(|r_1| + \sqrt{r_2^2 + \cdots + r_N^2} \right) \quad (3.3)$$

where r_i is the response (axial, shear, moment, etc.) to SMPA demands from mode i ; and N is the number of modes being considered. Note that the signs of the individual responses are lost due to the modal combination.

Member sizes resulting from the SMPA procedure are shown in Table 3.4.

Generalized modified modal superposition

The GMMS procedure determines the forces on the spine using an equivalent linear model that represents the pivoting response of the spine at peak displacement under MCE-level shaking. Two models are required for the GMMS procedure: an equivalent linear model used to determine GMMS modal responses, and the elastic design model used for ELF design criteria.

The GMMS procedure uses an equivalent linear model to estimate the demands on the force-controlled elements by replacing the deformation-controlled elements with equivalent stiffness springs. An iterative procedure is used to estimate the roof drift ratio at MCE, θ_{MCE} , and an equivalent SDOF rotational stiffness, K_{eq} . The equivalent stiffness is equal to the secant stiffness M_{MCE}/θ_{MCE} , multiplied by the equivalent stiffness modification parameter λ ; λ is based on a linear regression of rocking frame data, and may not be appropriately calibrated for SBFs, as they have different response characteristics to rocking frames. M_{MCE} is the estimated overturning moment at MCE, which, for SBFs, Martin and Deierlein (2021) recommend be based on the capacity-limited forces in the BRBs. The stiffness of the individual equivalent stiffness springs is then determined by rational analysis, such that the global frame overturning stiffness is equal to K_{eq} . Response spectrum analysis, with a modified modal combination rule, is then used to apply forces to the equivalent model and determine required strengths.

The GMMS equivalent linear model for strongback braced frames as described by Martin and Deierlein (2021) does not include the secondary energy dissipation provided by beam bending at the face of the strongback. Initially for this work, the beams were to be modeled as force-controlled elements to stay

Table 3.3: Target roof drifts and base shears for SMPA procedure

Building	Mode	Target roof drift, $\theta_{R,i}$ [%]	Target base shear, $V_{b,i}$ [kip]
SMPA-2	1	1.20	—
	2	0.028 9	151
SMPA-4	1	1.04	—
	2	0.059 1	631
	3	0.007 57	90.8
	4	0.000 204	9.39
SMPA-8	1	0.987	—
	2	0.106	1 440
	3	0.012 5	348
	4	0.001 18	16.9

Table 3.4: Member sizes for frames determined using the SMPA procedure

Building	Story	BRB Size [in ²]	Beam	Left Column	Strongback Brace	Strongback Column	Tie Brace
SMPA-2	1	7.5	W12×96	W12×72	W12×96	W12×53	—
	2	7.5	W12×96	W12×72	W12×53	W12×53	—
SMPA-4	1	9.5	W12×96	W12×170	W12×210	W12×170	—
	2	9.5	W12×96	W12×170	W12×79	W12×170	W12×106
	3	9.5	W12×96	W12×96	W12×79	W12×79	W12×106
	4	9.5	W12×96	W12×96	W12×79	W12×79	—
SMPA-8	1	11.5	W12×120	W14×426	W14×426	W14×500	—
	2	11.5	W12×120	W14×426	W14×342	W14×500	W14×426
	3	11.5	W12×120	W14×342	W14×193	W14×500	W14×426
	4	11.5	W12×120	W14×342	W14×176	W14×500	W14×426
	5	11.5	W12×120	W14×311	W14×159	W14×342	W14×426
	6	11.5	W12×120	W14×311	W14×159	W14×342	W14×257
	7	11.5	W12×120	W14×159	W14×145	W14×159	W14×257
	8	11.5	W12×120	W14×159	W14×132	W14×159	—

consistent. However, initial design analysis showed that the beams could not be sized appropriately, as larger beams simply attracted more force. For this work, the procedure was modified to include equivalent stiffness rotational springs at the face of the strongback in order to model the beam yielding, as well as the standard equivalent stiffness springs for the BRBs. A schematic of the equivalent linear model is shown in Fig. 3.6.

The modal combination for GMMS responses to determine E_h is given by

$$E_h = \pm 1.3 \left(\frac{|r_1|}{R_1} + \sqrt{r_2^2 + \dots + r_N^2} \right) \quad (3.4)$$

where R_1 is the GMMS reduction factor for mode 1, which for unstacked frames is equal to the GMMS equivalent stiffness modification parameter λ ; r_i is the response to GMMS demands from mode i ; N is the number of modes being considered; and the 1.3 factor comes from the suggested load combination for GMMS (Martin and Deierlein 2021). No specific number of modes or level of modal mass participation is specified in the description of the GMMS procedure; for consistency with SMPA, the same number of modes were considered. Design variables for the GMMS procedure are shown in Table 3.5.

Member sizes resulting from the GMMS procedure are shown in Table 3.6.

Response spectrum analysis with capacity design

To provide a control case based on current seismic design practice, a response spectrum analysis with capacity design (RSAC) procedure was used. The RSAC procedure requires two models: the elastic design model, used for the response spectrum analysis and ELF design criteria, and a capacity design model, where the BRBs and beam plastic hinges (at the face of the strongback) are replaced with equivalent capacity-limited forces and moments. The capacity design model, shown schematically in Fig. 3.7, only incorporates the effects from mode 1. Capacity-limited BRB forces in compression, P_c , and in tension, P_t , were calculated using the equations from AISC 341-16 (AISC 2016a):

$$\begin{aligned} P_c &= \omega \beta R_y F_{y_{sc}} A_{sc} \\ P_t &= \omega R_y F_{y_{sc}} A_{sc} \end{aligned} \quad (3.5)$$

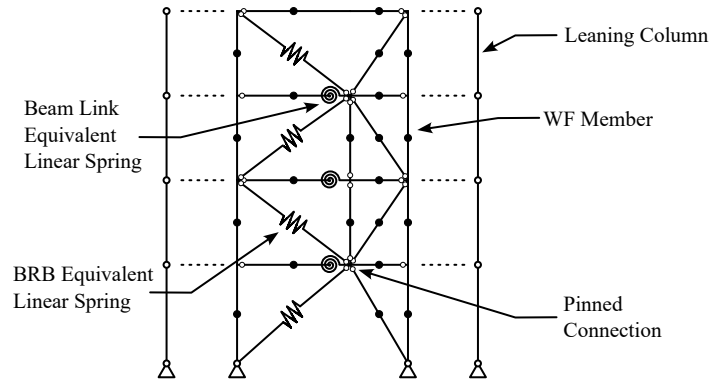


Figure 3.6: Schematic of equivalent linear model for GMMS procedure

Table 3.5: GMMS design variables

Building	Equivalent SDOF Stiffness, K_{eq} [10^6 kip in/rad]	Roof Drift Ratio at MCE, θ_{MCE} [%]	λ	N
GMMS-2	12.0	2.40	1.14	2
GMMS-4	34.0	1.95	1.08	4
GMMS-8	77.7	1.78	0.948	4

Table 3.6: Member sizes for frames determined using the GMMS procedure

Building	Story	BRB Size [in ²]	Beam	Left Column	Strongback Brace	Strongback Column	Tie Brace
GMMS-2	1	7.5	W12×96	W12×72	W12×170	W12×72	—
	2	7.5	W12×96	W12×72	W12×72	W12×72	—
GMMS-4	1	9.5	W12×96	W12×230	W12×305	W12×279	—
	2	9.5	W12×96	W12×230	W12×152	W12×279	W12×210
	3	9.5	W12×96	W12×79	W12×152	W12×136	W12×210
	4	9.5	W12×96	W12×79	W12×152	W12×136	—
GMMS-8	1	11.5	W12×120	W14×550	W14×426	W14×665	—
	2	11.5	W12×120	W14×550	W14×342	W14×665	W14×426
	3	11.5	W12×120	W14×398	W14×257	W14×550	W14×426
	4	11.5	W12×120	W14×398	W14×193	W14×550	W14×426
	5	11.5	W12×120	W14×311	W14×193	W14×398	W14×426
	6	11.5	W12×120	W14×311	W14×176	W14×398	W14×283
	7	11.5	W12×120	W14×159	W14×159	W14×159	W14×283
	8	11.5	W12×120	W14×159	W14×159	W14×159	—

where ω and β are the BRB hardening parameters, $R_y F_{y_{sc}}$ is the expected yield strength of the steel core = 42 ksi, and A_{sc} is the area of the steel core. Capacity-limited moments from the beams were conservatively assumed to be equal to the full expected plastic moment $M_{pe} = R_y F_y Z_x$, where $R_y = 1.1$ for ASTM A992 members (AISC 2016a).

Member sizes resulting from the RSAC procedure are listed in Table 3.7.

Other design checks

Beam, column, and conventional brace members were also designed to satisfy ductility requirements from AISC 341-16. SBFs have not yet been codified in AISC 341-16, so the required ductility categories were selected based on the expected role of the members. Since the beams form part of the energy dissipation system, they were selected to be highly ductile members, while columns and braces were selected to be moderately ductile since they are intended to remain elastic.

The equivalent lateral force procedure with $C_d = 5$ was used to check the ASCE 7-22 limits on stability coefficient θ and story drift Δ . This is the same C_d as for buckling-restrained braced frames (ASCE 2022), since SBFs have not been assigned performance factors. The stability coefficient was calculated using Eq. (G19-2) from Charney et al. (2020), where two analyses are performed, one with P- Δ effects included and one without. Simple limits of $\theta_{max} = 0.1$ and $\Delta_a = 0.02h_s$ were used. The only case where stability checks controlled the design was RSAC-8, where story drift limits were exceeded in the intermediate stories under a purely strength-based design.

3.2.3 Nonlinear Analysis

To evaluate the resulting designs, nonlinear static pushover and nonlinear dynamic analyses were performed. Three performance variables for the strongback were investigated: (1) resistance to collapse, measured by the collapse margin ratio calculated from incremental dynamic analysis; (2) ability to distribute drift demands across stories under MCE-level shaking, measured using the maximum “marginal drift”; and (3) ability of the strongback to remain elastic under MCE-level shaking, measured using the total strain energy absorbed by the strongback over the duration of the ground motion. To investigate the required strength and stiffness of the strongback, and to evaluate how the different design methods compare to each other, these variables were

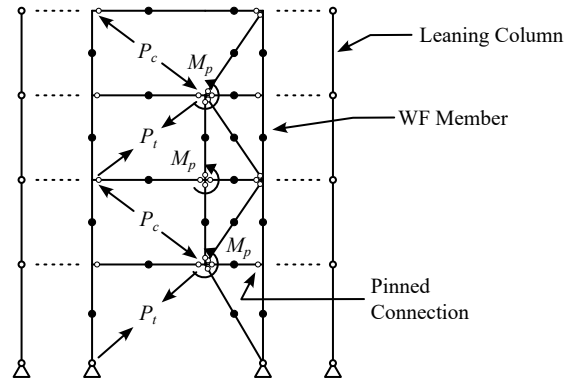


Figure 3.7: Schematic of capacity design model to accompany response spectrum analysis

Table 3.7: Member sizes for frames determined using the RSAC procedure

Building	Story	BRB Size [in ²]	Beam	Left Column	Strongback Brace	Strongback Column	Tie Brace
RSAC-2	1	7.5	W12×96	W12×72	W12×106	W12×45	—
	2	7.5	W12×96	W12×72	W12×50	W12×45	—
RSAC-4	1	9.5	W12×96	W12×190	W12×190	W12×106	—
	2	9.5	W12×96	W12×190	W12×53	W12×106	W12×58
	3	9.5	W12×96	W12×72	W12×72	W12×45	W12×53
	4	9.5	W12×96	W12×72	W12×53	W12×45	—
RSAC-8	1	11.5	W12×120	W14×398	W14×342	W14×283	—
	2	11.5	W12×120	W14×398	W14×159	W14×283	W14×233
	3	11.5	W12×120	W14×283	W14×159	W14×159	W14×233
	4	11.5	W12×120	W14×283	W14×159	W14×159	W14×159
	5	11.5	W12×120	W14×211	W14×159	W14×145	W14×159
	6	11.5	W12×120	W14×211	W14×159	W14×145	W14×82
	7	11.5	W12×120	W14×159	W14×159	W14×82	W14×82
	8	11.5	W12×120	W14×159	W14×159	W14×82	—

measured for a range of stiffness and strength modifications made to the strongback members. While these performance variables do not directly measure resilience, they are used in this study as surrogate metrics to quantify the effect of strongback parameters and design on the system's resilience to structural damage. Each of them is related to resilience: for example, a structure not collapsing or otherwise being less damaged can enable a quicker return to normal operation after a seismic event.

The model used for the nonlinear analyses is similar to the one used for the SMPA design procedure (Fig. 3.5), but incorporates additional sources of nonlinearity. Wide-flange members were represented using centerline beam-column elements using the force-based formulation, 5 integration points per element, and 2 elements per member, with fiber cross-sections utilizing the Steel02 uniaxial material model. Residual stresses were incorporated for wide-flange members using the Lehigh pattern (Galambos and Ketter 1959). Buckling restrained braces were represented using truss elements with a Steel4 non-symmetric hysteretic model and Fatigue wrapper, with parameters taken from the BRB material model calibration from Simpson (2018). All materials used the expected yield strength $R_y F_y$ for the “nominal” yield stress. Geometric nonlinearity was incorporated using corotational truss elements and a corotational transformation for the beam-column elements. Both the strongback base and the opposite column base were modeled as pinned. Tributary gravity loads equal to $1.05D + 0.25L$ were lumped at brace workpoints. Mass based on $1.0D$ was distributed in the same fashion as the gravity loads. P- Δ effects were included through leaning columns which carried the remaining gravity load not directly tributary to the frame. The leaning columns were connected to the rest of the frame by constraining their lateral displacements to the corresponding workpoint.

To investigate the effect of the strongback's strength and stiffness, the strongback members (here comprising the strongback column as well as the diagonal and tie braces, but excluding the beams) were modeled with varying yield strength F_y and elastic modulus E . F_y and E were selected as the parameters for the strongback's strength and stiffness due to their ease of modification and, in contrast to cross-section properties, their relative ability to decouple strength and stiffness. The parameters were modified by applying a constant factor to their nominal values. When varying strength and stiffness together, the same constant factor was used to modify both strength and stiffness. A range between 0.1 and 10 times as stiff or strong was identified as the range of interest through preliminary analysis. Further analysis showed that significant strength reduction led to poor, unrealistic seismic performance, so a range between 0.6 and 10 times was utilized when varying strength.

A total of 810 frames were evaluated: the 9 base designs were evaluated at nominal strength and stiffness, with 34 values of strength and stiffness varied together, 29 values of stiffness varied alone, and 26 values of strength varied alone.

The collapse performance of the overall structure was measured using incremental dynamic analyses (IDA), following the FEMA P695 methodology, to determine the adjusted collapse margin ratio, *ACMR*, for each configuration. An *ACMR* of 1.0 indicates a 50% probability of collapse at the MCE intensity, with increasing values of *ACMR* corresponding to decreasing probability of collapse. Following FEMA P695 Appendix F for collapse evaluation of individual buildings, a 10% probability of collapse at MCE was considered the minimum acceptable collapse performance, evaluated by comparing the *ACMR* to the minimum acceptable *ACMR* for a 10% probability of collapse, *ACMR*_{10%}. Collapse was not directly simulated, and instead defined using a story drift limit of 5%. This limit is based on experimental tests of column drift capacities, which showed capacities of 7–9% (Newell and Uang 2006), and conservative estimates of braced frame drift capacities (Uriz and Mahin 2008). A full FEMA P695 study requires identification of performance groups and a broader set of configurations than investigated here; this study is thus strictly a subset of a full FEMA P695 study of strongback braced frames.

The strongback's ability to distribute demands under MCE-level shaking was measured using the maximum “marginal drift” L . Here, marginal drift is defined as the difference between the story drift ratio and the average story drift ratio of the other stories. The maximum value of this marginal drift is defined by Eq. (3.6):

$$L = \max_{i,t} \left(\theta_i(t) - \frac{1}{N-1} \sum_{j=1, j \neq i}^N \theta_j(t) \right) \quad (3.6)$$

where $\theta_i(t)$ is the absolute value of the story drift ratio at story i and time t , and N is the number of stories. This measure identifies when drift concentrations occur, and avoids misleading results that can occur at small drift levels.

The strongback's ability to remain elastic under MCE-level shaking was measured using the total energy absorbed by the strongback members through deformation, calculated cumulatively over the duration of the ground motion. The total energy is representative of the amount of steel yielding that occurs in the strongback members, including partial yielding that occurs with inelastic buckling. The total energy was calculated from

element-level forces and deformations, which has the limitation of including elastic energy from locked-in stresses. The elastic contribution to the strain energy is small relative to the plastic and hardening contributions at the MCE-level shaking considered. A primary goal of the strongback is to remain elastic and limit damage under severe shaking to specified locations in the frame (i.e., the primary energy dissipation mechanism); if significant yielding and hardening occurs, then the strongback may need to be repaired as well. Furthermore, detailing requirements for SBFs have not been established—if significant yielding occurs in the strongback, requirements should be established to ensure a ductile global response.

Nonlinear static pushover analysis

Nonlinear static pushover analysis was used to identify the period-based ductility, μ_T , of the frames. This quantity is required to determine the spectral shape factor, SSF , and the total system uncertainty, β_{TOT} , which are used in the FEMA P695 collapse evaluation procedure (FEMA 2009).

The pushover analysis was performed using a first-mode distribution of lateral forces

$$f_i \propto m_i \phi_i \quad (3.7)$$

where m_i is the mass at node i and ϕ_i is the x-direction eigenvector for mode 1 at node i (determined by eigenvalue analysis of the model), until a drop in force of 20% was detected. The maximum base shear V_{max} and the ultimate displacement δ_u were recorded. V_{max} was used to calculate the effective yield displacement $\delta_{y,eff}$ per FEMA P695 Eq. (6-7) and (6-8). μ_T is then defined as the ratio of δ_u to $\delta_{y,eff}$.

Due to convergence issues, not all pushover analyses reached the specified 20% drop in force. These convergence issues were associated with BRB fracture. Thus, it was concluded that the failed analyses still accurately captured V_{max} and $\delta_{y,eff}$, but may underpredict δ_u and μ_T (e.g., if a single BRB fracture does not result in a 20% drop in strength). Values of μ_T are high enough, however, that this potential underprediction does not impact the results, as SSF is constant with respect to μ_T when $\mu_T > 8.0$ and β_{TOT} is constant with respect to μ_T when $\mu_T > 3.0$, and all the analyses that potentially underpredict μ_T have $\mu_T > 8.0$.

Nonlinear dynamic analysis

Response history analyses and incremental dynamic analyses were performed using the 22 ground motion pairs of the FEMA P695 far-field record set, which provides a set of site-independent strong ground motions suitable for collapse evaluation (FEMA 2009). The ground motions were scaled as shown in Eq. (3.8):

$$\ddot{x}_g = SF \cdot \frac{S_{MT}}{\hat{S}_{NRT}} \cdot NM \cdot \ddot{x}_{g,recorded} \quad (3.8)$$

where SF is a selectable scale factor, S_{MT} is the spectral intensity of the MCE at the fundamental period of the building, $\hat{S}_{NRT}$ is the median spectral intensity of the normalized record set at the fundamental period of the building, and NM is the normalization factor for the specific ground motion. NM is calculated for each ground motion as the median peak ground velocity (PGV) of the record set divided by the PGV of the ground motion. The fundamental period used for calculation of these values is the approximate fundamental period listed in Table 3.2. Values of $S_{MT}/\hat{S}_{NRT}$ and NM are tabulated in FEMA P695, and a value of $SF = 1.0$ provides the ground motion scaled to MCE. For incremental dynamic analysis, SF was increased until collapse was detected (using the non-simulated collapse limit of 5% story drift).

Using the results of the IDAs, the adjusted collapse margin ratio, $ACMR$, was determined. $ACMR$ is the product of the collapse margin ratio, CMR , and the spectral shape factor, SSF , as defined in FEMA P695 (FEMA 2009). The collapse margin ratio is the ratio of the median collapse intensity $\hat{S}_{CT}$ to the intensity of the maximum considered earthquake S_{MT} . Given the scaling used in this work (Eq. (3.8)), the collapse margin ratio is equal to the scaling factor, SF , that results in half the ground motions causing collapse.

The spectral shape factor, SSF , depends on the record set, the fundamental period of the building (Table 3.2), seismic design category, and the period-based ductility, μ_T , determined from static pushover analysis. Values of SSF are tabulated in FEMA P695 Table 7-1 (FEMA 2009).

Calculating the acceptable $ACMR$ for a 10% collapse probability, $ACMR_{10\%}$, requires an assessment of uncertainty in the evaluation. This requires rating the quality of (1) the nonlinear model, (2) the design requirements, and (3) the available test data. For this work, all “good” ratings were used. The nonlinear model captures important collapse-related behaviors (e.g., P- Δ , BRB fracture, yielding and residual stresses), but does not capture all potentially collapse-related behavior (e.g., fracture of conventional braces or connections). The

design requirements are based in sound principle and build from well-established requirements for structural steel systems, but aspects specific to strongback systems have not been significantly tested. Significant amounts of test data exist for the individual components, though full-scale testing of strongbacks is limited. While other system uncertainty ratings could be justifiable, these selections are similar to previous SBF studies and align with previous FEMA P695 studies (Korlapati et al. 2021; Simpson 2018). The final component of the uncertainty is the record-to-record variability, which is constant when $\mu_T \geq 3.0$. Given these ratings, and since $\mu_T \geq 3.0$ for all configurations, the total system collapse uncertainty $\beta_{TOT} = 0.525$. This gives an acceptable value of the adjusted collapse margin ratio for a 10% collapse probability, $ACMR_{10\%}$, of 1.96.

3.3 Results and Discussion

The presentation of results and discussion thereof is divided into topics of collapse resistance, distribution of demands, and strongback inelasticity. The plots in Figs. 3.8 to 3.10 show the median value of the frame responses when subjected to the 22 ground motion pairs of the FEMA P695 far-field record set.

3.3.1 Collapse Resistance

The plots in Fig. 3.8 show the adjusted collapse margin ratio, $ACMR$, for the frames as a function of the modulus of elasticity, E , and steel yield stress, F_y , used for the strongback members, as well as the acceptable value of the adjusted collapse margin ratio for a 10% collapse probability, $ACMR_{10\%}$, of 1.96, plotted as a horizontal dot-dashed line.

At nominal strength and stiffness, the design methods provide acceptable collapse performance (i.e., $ACMR \geq ACMR_{10\%}$) except for the 4-story frame designed using the RSAC method. The RSAC method is a simple method that captures higher-mode effects and yield mechanisms independently; it is used in this work as a point of comparison to the other methods that more rigorously capture these behaviors and their interaction, including partial yield mechanisms. That the RSAC method provides sufficiently collapse-resistant designs for the 2- and 8-story frames indicates the novel design methods (i.e., SMPA and GMMS) are not necessary to provide collapse resistance for all SBF configurations. Furthermore, the results indicate that the 4-story frame occupies a “mid-range” where the combination of higher-mode effects and partial yielding mechanisms—a combination not accounted for by RSAC—is substantial. This is especially noted by the

smaller column sizes produced by the RSAC method, for which the SMPA and GMMS methods predict significantly higher moment demands. Investigation of a broader range of frames is necessary to confirm the existence of this mid-range for SBFs in general.

The 8-story frames all have similarly high collapse resistance, with $ACMR$ exceeding $ACMR_{10\%}$ by 50% at nominal strength and stiffness for all three methods. The 8-story RSAC-designed frame is notably different from the other 8-story frames, despite providing equivalent collapse performance. RSAC-8 was controlled by drift limits, leading to larger member sizes than would have been obtained from strength design alone (the frames designed by the other methods were not controlled by drift limits). The RSAC frames also consistently have larger opposite columns than strongback columns; while this is also seen in SMPA-2 and SMPA-4, the difference is larger in the RSAC frames.

$ACMR$ increases when increasing strongback stiffness and strength together. For the 2- and 4-story buildings, a plateau is reached at 2–3 times the nominal strength and stiffness. For the 8-story buildings, no plateau is observed within the ratios investigated. Increasing strength or stiffness individually also provides benefit. The collapse resistance is more sensitive to changes in strength than stiffness, though very large increases in strength provide less benefit proportionally and eventually reach a plateau at 2–7 times the nominal strength. Changes to stiffness affect the results across the entire range.

The fact that collapse resistance varies, especially with strongback strength, indicates that the strongback with nominal E and F_y does not remain elastic at the median collapse intensity. This is expected in part because the design methods target elastic behavior at MCE, not collapse. If the strongback were stronger, such that it did remain elastic, the collapse resistance would increase by as much as 78% (in the case of RSAC-4). The $ACMR$ for the stronger strongback is significantly higher than the acceptable value, $ACMR_{10\%}$; this implies, for example, that fewer or smaller BRBs could be used with a stronger strongback while still achieving the desired collapse performance. However, using smaller BRBs decreases the demands on the strongback predicted by the design methods. Thus, the design methods investigated here for the strongback are satisfactory only for the BRBs sizes generated by the initial assumptions about the strongback, and not for the general case of arbitrary BRB sizes and distributions. Further work is needed to develop rigorous and broadly applicable BRB size selection criteria.

With nominal parameters, and generally across the parameter range, the 2- and 4- story GMMS frames

have higher collapse resistance than the other design methods; this is expected because the GMMS-designed frames have bigger members (see Table 3.8). For the 8-story configuration, the GMMS frame performs very similarly to the SMPA frame. With the exception of the 0.1 stiffness factor in the 4-story building, $ACMR$ is greater than $ACMR_{10\%}$ for all variations of the GMMS frames, indicating that GMMS produces larger members than necessary for SBFs; thus, details of the method, such as the 1.3 factor in Eq. (3.4), could be revisited for SBFs.

3.3.2 Distribution of Demands under MCE-level Shaking

One of the defining features of a strongback is its ability to more uniformly distribute demands along the height of the building. The strongback's ability to distribute demands under MCE-level shaking is shown in Fig. 3.9. The median value at MCE of the maximum marginal drift—see Eq. (3.6)—is plotted.

For all buildings and design methods, except for RSAC-4, the marginal drift remains the same when strength is increased from nominal, indicating that only minimal yielding is occurring at MCE in the strongback in the other frames at nominal strength and stiffness. Marginal drift will increase rapidly when strength is reduced sufficiently, indicating failure of the strongback to maintain vertical continuity and the formation of a soft story. This increase in marginal drift was not observed for the GMMS frames, but it is reasonable to assume that it would see the same rapid increase in maximum marginal drift if strength was decreased further. This further indicates that the GMMS procedure produces larger members than necessary.

Changing stiffness corresponds to changes in marginal drift across the entire range of factors investigated. Decreasing stiffness increases marginal drifts, though not as rapidly as decreasing strength. Increasing stiffness reduces marginal drifts, approaching zero marginal drift at 10 times nominal stiffness, indicating near-uniform drifts over the entire duration of the ground motion.

Changing stiffness and strength together simply shows a combination of the above behaviors, with reductions in strength and stiffness compounding with each other.

For the 2-story building, the RSAC and SMPA methods result in similar median behavior, while GMMS has slightly lower marginal drifts. For the 8-story building, all design methods provide similar behavior over the whole range of factors investigated, with GMMS and SMPA providing nearly identical median results while RSAC performs slightly worse than the novel design methods. The similarity of the GMMS and SMPA

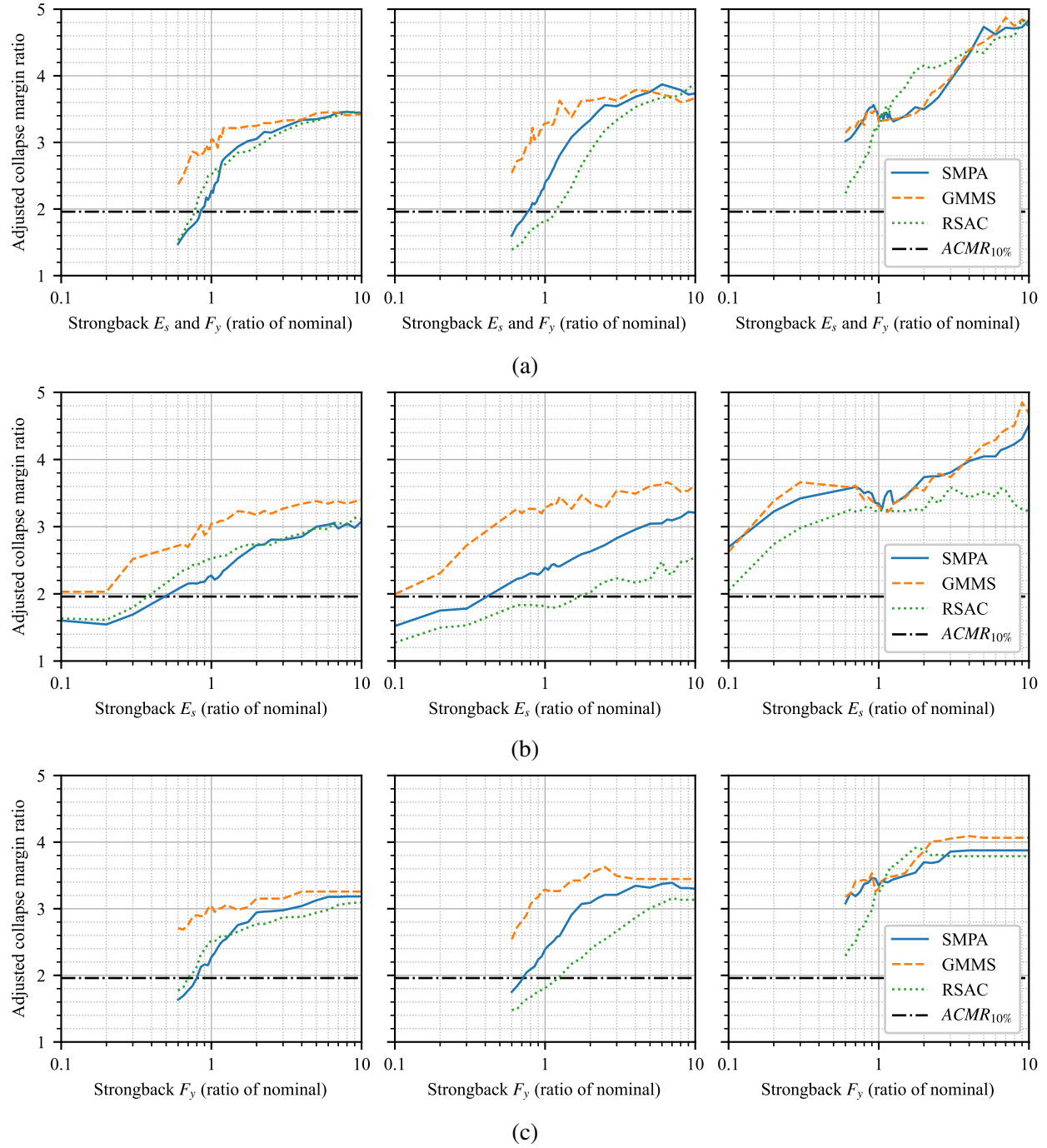


Figure 3.8: Adjusted collapse margin ratio. (a) Varying strength and stiffness; (b) varying stiffness, F_y remains nominal; (c) varying strength, E remains nominal. Left-to-right: 2, 4, and 8 stories

results can be attributed to the similarity of the strongback members, especially the strongback braces, which are the primary contributor to the shear-type response of the strongback.

3.3.3 Strongback Inelasticity under MCE-level Shaking

Strongbacks are expected to remain essentially elastic under ground shaking at the MCE level. The strongback's ability to remain elastic under MCE-level shaking is shown in Fig. 3.10 as the ratio of the strain energy absorbed by the strongback to the total strain energy absorbed by the frame. As noted previously, this measure includes some amount of "locked-in" elastic energy, but the dominant source is dissipation through yielding.

At nominal strength and stiffness, the strain energy absorbed by the strongback is less than 1% of the strain energy absorbed by the frame as a whole (except for RSAC-4, which is 5%). The strain energy ratio increases rapidly with reductions in strength, while increases in strength provide only marginal reductions. When increasing stiffness alone, the share of the strain energy in the strongback tends to increase. As the strain energy decreases when increasing both strength and stiffness, the increase with stiffness alone is not attributable simply to large amounts of stored elastic energy, and is indicative of the stiffer strongback attracting greater demands.

Comparing design procedures, the control design method (RSAC) shows more yielding in the strongback across all building heights and modifications. GMMS generally shows less yielding, though GMMS and SMPA show very similar behavior for the 8-story structure. This indicates that the novel design methods are more effective at producing designs that remain essentially elastic during MCE-level shaking.

3.3.4 Other Results

Additional results are shown here: (1) the median strain energy absorbed at MCE by all the components of the frame, grouped by element type; (2) the median strain energy absorbed at MCE by the components of the strongback, grouped by element type; and (3) median residual drift at MCE. Results (1) and (2) provide additional insight as to which parts of the frame are responsible for the energy dissipation of the frame, as well as which components are more sensitive to variation in the strongback's properties. Result (3), residual drift, is important to measure, as residual drifts are an additional indicator of permanent damage, and large residual drifts can result in a building being marked as unsafe for occupancy. For brevity, the

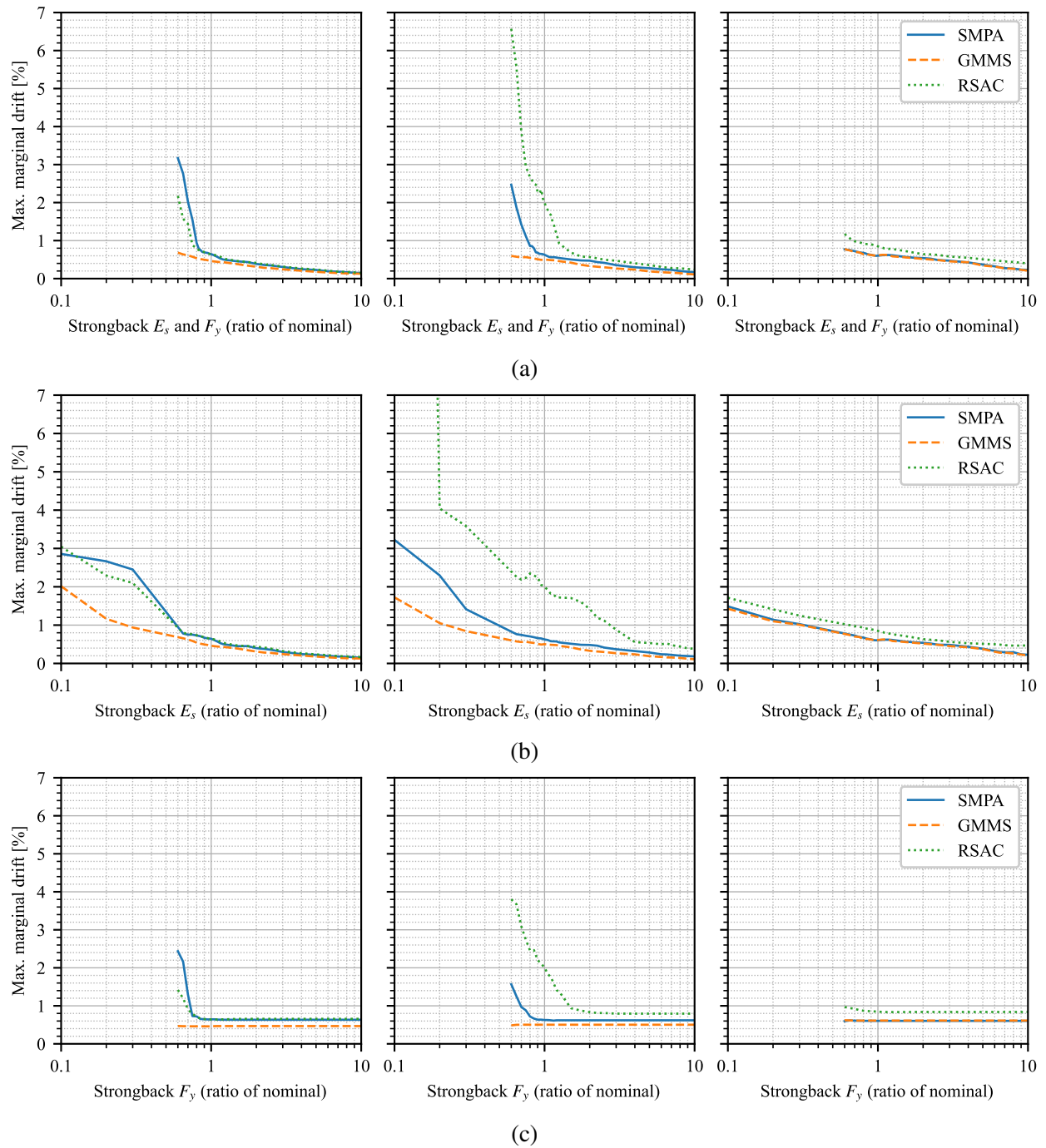


Figure 3.9: Story drift concentration at MCE, measured by maximum marginal drift. (a) Varying strength and stiffness; (b) varying stiffness, F_y remains nominal; (c) varying strength, E remains nominal. Left-to-right: 2, 4, and 8 stories

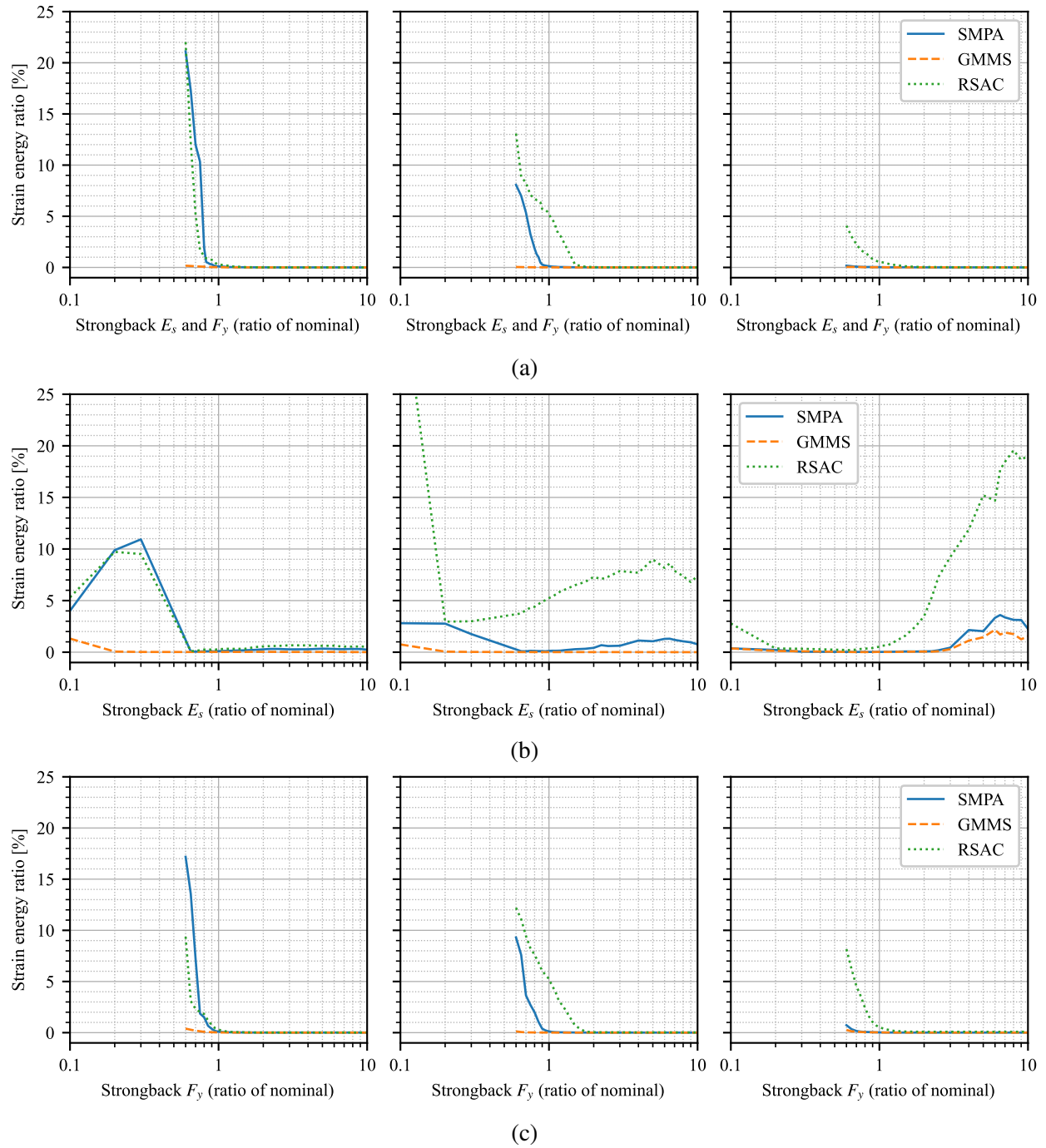


Figure 3.10: Strain energy stored in and dissipated by the strongback at MCE, as a ratio of the total frame strain energy. (a) Varying strength and stiffness; (b) varying stiffness, F_y remains nominal; (c) varying strength, E remains nominal. Left-to-right: 2, 4, and 8 stories

results shown in this section are for a subset of the frames; complete figure sets are available online at <https://doi.org/10.17603/ds2-jrcm-2c58>.

Fig. 3.11 shows the median strain energy absorbed by all the components of the 2-, 4-, and 8-story SMPA frames at MCE, grouped by element type. As expected, the BRBs are the primary source of energy dissipation, far exceeding the combined contribution of the other elements. The beams, despite being designated as a secondary energy dissipator, do not contribute significantly to the energy dissipation of the frames. For the 2- and 4-story frames at nominal strength and stiffness, the strongback, which is designed to remain essentially elastic, absorbs as much energy as the beams; noting that the strongback is significantly larger and thus has much more strain energy capacity. Similar results are seen for the frames designed by the other design methods.

Fig. 3.12 shows the median strain energy ratios of the different elements of the strongback itself at MCE. The diagonal braces are the most significantly affected by changes to strongback strength and stiffness. At low strength and stiffness, large buckling deformations lead to significant increases in the strain energy handled by the braces. This indicates that, for these frames, as the strongback becomes stiffer and stronger, the response transitions from a shear-dominated response to a flexure-dominated response. Notably, the ties do not contribute significantly to the dissipation in the strongback. Similar results are seen for the frames designed by the other design methods.

Fig. 3.13 shows median values of the maximum residual story drift at MCE for all the frames when varying strength and stiffness together. The maximum residual story drift was taken as the maximum value of the drift at each story at the end of the analysis, with time allowed at the end of the ground motion (equal to 10% of the ground motion duration) for the model to settle. The design methods all provide acceptable residual drift control at nominal strength and stiffness, with median residual drifts under 1%. Similar to story drift concentration, reductions in strength and stiffness quickly lead to large increases in residual drift. The residual drifts of the GMMS frames are largely insensitive to changes in strength and stiffness over the range investigated. Overall, the SBFs are very capable at controlling residual drift, indicating their usefulness in improving resilience to seismic events.

Table 3.8: Weight of strongback braced frames

Building	Member Weight [kip]	Member Weight : RSAC Weight
SMPA-2	12.8	1.01
SMPA-4	39.8	1.18
SMPA-8	179	1.47
GMMS-2	14.9	1.18
GMMS-4	52.2	1.55
GMMS-8	194	1.59
RSAC-2	12.7	1.00
RSAC-4	33.5	1.00
RSAC-8	122	1.00

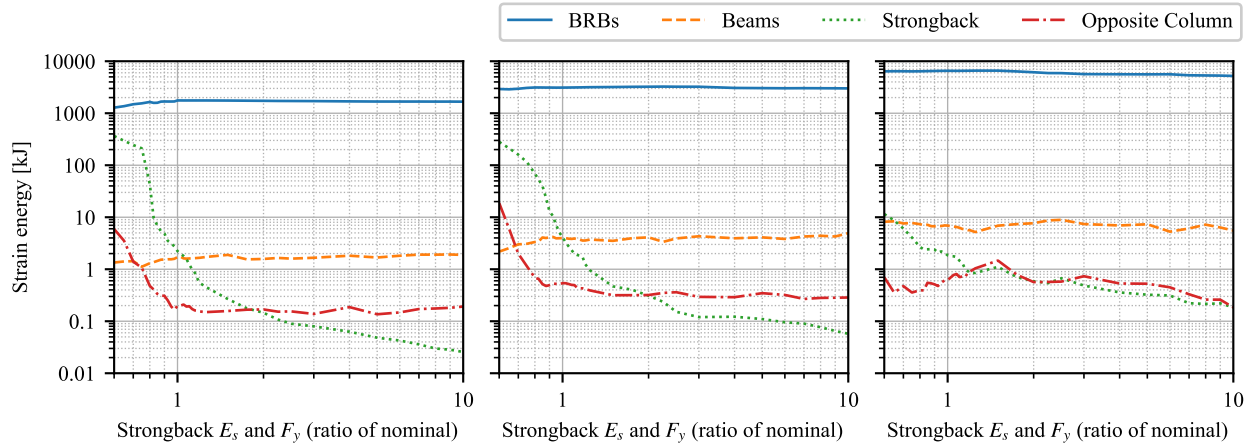


Figure 3.11: Median strain energy stored in and dissipated by the various components of the SMPA frames at MCE, varying strength and stiffness together. Left-to-right: 2, 4, and 8 stories

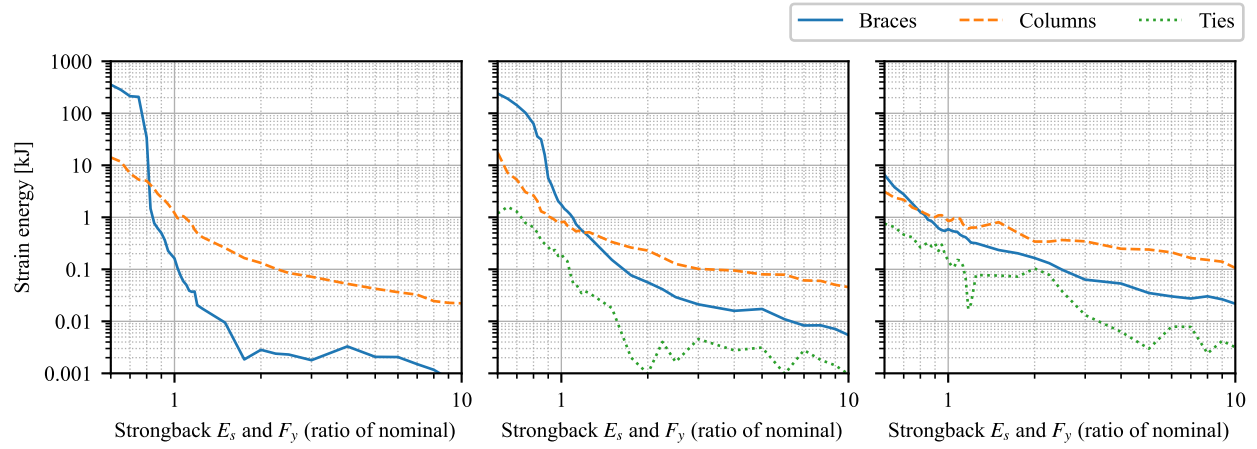


Figure 3.12: Median strain energy stored in and dissipated by the components of the strongback in the SMPA frames at MCE, varying strength and stiffness together. Left-to-right: 2, 4, and 8 stories

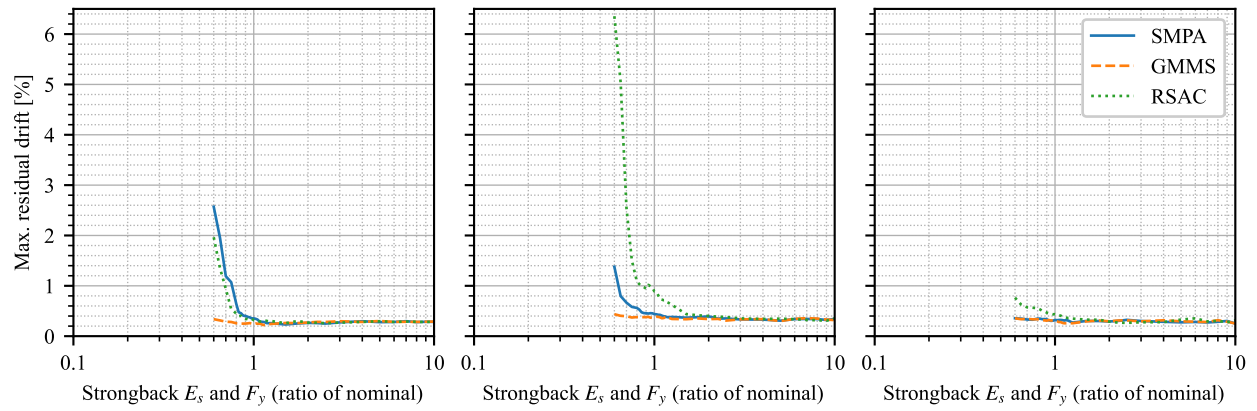


Figure 3.13: Median value of maximum residual drift at MCE, varying strength and stiffness together. Left-to-right: 2, 4, and 8 stories

3.4 Conclusions

In this work, three design methods for strongback braced frames were evaluated for three buildings of varying heights. The design methods were evaluated using nonlinear analysis across a range of modifications to the strength and stiffness of the resulting frames. The methods were evaluated for their ability to (1) provide collapse resistance, (2) limit story drift concentrations at MCE, and (3) limit yielding in the strongback spine itself at MCE. The results show that there is utility in the use of novel design methods for the design of strongback braced frames. The key observations from this work are:

- The simplified modal pushover analysis (SMPA) and generalized modified modal superposition (GMMS) procedures are generally well-calibrated and provide value above the simpler response spectrum analysis with capacity design (RSAC).
- Frames designed according to SMPA and GMMS have acceptable ACMR, and the strongback behaves as expected with minimal drift concentration and yielding.
- GMMS is more conservative, resulting in a heavier strongback, but also improved performance. The 1.3 factor on the GMMS loads—see Eq. (3.4)—could be revisited for strongbacks.
- For all measures observed in this study, the greatest impact was from not having enough strength in the strongback. Reductions in strongback strength led to rapid loss of collapse resistance, and large increases in drift concentration and strongback inelasticity at MCE-level shaking. With the exception of RSAC-4, the frames saw minimal benefit to MCE-level responses when strength was increased from nominal. All frames saw reductions in story drift concentrations from increased stiffness, but increased stiffness on its own also led to increased strongback inelasticity, especially for RSAC-4 and RSAC-8.
- The BRBs, as expected, provide the vast majority of the energy dissipation in the structure. The beams do not provide large amounts of energy dissipation. This challenges their description in this work and by others as secondary energy dissipators in strongback braced frames, and may indicate the beams could be designed for only moderately ductile requirements instead of highly ductile requirements.

Overall, this work furthers the development of the SBF system as a viable seismic force-resisting system for highly resilient buildings. Several lines of inquiry for future work remain, including:

- Further work is needed to develop rigorous and broadly applicable BRB size selection criteria. While the SMPA and GMMS design methods can provide good predictions of the strongback demands, they are reliant on the initial BRB design step in order to provide sufficient lateral resistance. Prediction of strongback demands and selection of BRB sizes are inherently coupled, but the current methods for selecting BRB size for SBFs rely on assumptions about the overall frame behavior that cannot be applied to generalized BRB distributions.
- Further work is needed to clarify the role of the beams in strongback braced frames and to develop criteria for sizing beams in strongback braced frames, including the appropriate design ductility level.

CHAPTER 4

Design of BRBs and Collapse Performance of Strongback Braced Frames with Varying BRB Configurations

4.1 Introduction

The strongback braced frame (SBF) is a lateral force-resisting system intended to reduce damage concentration and improve the resilience of steel braced frames (Lai and Mahin 2014; Simpson 2018). SBFs combine buckling-restrained braces (BRBs) and a stiff elastic spine—here termed the strongback. The BRBs provide the necessary energy dissipation for the frame, while the strongback collects demands from across the height of the structure and distributes them to the BRBs. Owing to the ability of the strongback to distribute demands, SBFs open up a largely unexplored design space for braced frames, including a broad range of possible BRB configurations.

SBFs have two main parts to their design: sizing the primary energy-dissipating members, and sizing the strongback members themselves. Methods for the latter have been proposed and studied for SBFs and other spine systems (Martin and Deierlein 2021; Simpson and Rivera Torres 2021), but comparatively little investigation has been done for determining the required size of the primary energy-dissipating members. Often, the designer is directed to use existing standardized methods, such as the equivalent lateral force (ELF) procedure from ASCE 7-22 (ASCE 2022) to determine the initial size of the lateral resistance. However, as the demands on the strongback are related to the size of the energy dissipators, a poor initial estimation of the necessary lateral resistance can lead to under-sized strongback members.

Alternative methods for sizing the lateral resistance, which take into account the geometry and yielding behavior of the system, have also been proposed. Simpson (2018) suggests designing the BRBs at each level to resist 80% of the ELF base shear, with the assumption that the strongback is stiff and strong enough to engage the BRBs fully at each level. With uniform or near-uniform story heights, this has the advantage of producing a single size for all the BRBs in the structure. However, this method was developed for SBFs that have BRBs at every story and lacks justification for other configurations.

In rocking frames, design of the energy-dissipating elements is often accomplished through an overturning-based approach, where the system is designed to resist the overturning moment calculated from an equivalent lateral force distribution. Wiebe and Christopoulos (2015b) designed the energy-dissipating elements of a rocking frame as part of the determination of the minimum base rocking moment, which is the overturning moment that must be applied prior to the initiation of rocking. The minimum base rocking moment includes

contributions from the weight of the frame and post-tensioning strands as well, and the relative contribution of the energy-dissipating elements is selected by the designer using a unitless energy dissipation parameter. Takeuchi et al. (2015) investigated a rocking frame with buckling-restrained columns (BRC) as the energy-dissipating elements. The BRCs were sized to yield under the overturning moment associated with the level 2 seismic design hazard from the Japanese seismic code, which is similar to the design-basis earthquake from ASCE 7. Blebo and Roke (2018) investigated a rocking frame with BRCs and post-tensioning strands for self-centering. The BRCs were designed using a BRC strength factor and a frame strength factor. The BRC strength factor controls the size of the BRCs relative to the post-tensioning, while the frame strength factor amplifies the overturning moment the system is designed to resist.

There are also damper optimization methods that have been used for BRB design (Terazawa and Takeuchi 2019). These methods support configurations with missing BRBs, and have the potential to provide well-tuned designs, but require significantly more complex models and analysis than is desirable for design office work.

The method investigated in this work is similar to overturning-based methods used in previous studies (Blebo and Roke 2018; Takeuchi et al. 2015; Wiebe and Christopoulos 2015b), but is adapted particularly for SBF geometry and can be used for a broad range of BRB configurations. The structure is analyzed in its capacity-limited configuration, with design-level forces resisting ELF overturning forces. The ELF overturning forces must be amplified by a scale factor, γ , in order to achieve appropriate BRB sizes. The method can easily be adapted to include other sources of overturning resistance, such as the moment in the beams at the face of the strongback.

The objectives of this work are to: (1) confirm the use of the overturning method for SBFs, (2) calibrate the scale factor γ for various BRB configurations, and (3) evaluate the performance of SBF design methods in unusual BRB configurations. To achieve these objectives, a parametric study using a simplified model with a rigid strongback was used to evaluate a range of values of γ for a range of building heights. A more detailed “frame-level” model, with finite strength and stiffness strongback members, modeled with line elements, was then used to design and analyze SBFs for a four-story building using multiple BRB design methods, with the response analyzed using nonlinear static pushover and incremental dynamic analyses.

4.2 Frame Configurations

Three BRB configurations, shown in Figure 1, were investigated: (1) a standard “X” configuration, with BRBs at every level; (2) a configuration with BRBs at every level except the first, called the “R” configuration; and (3) a configuration with only a single BRB at the base, oriented vertically, called the “S” configuration. The X configuration is the default and most-studied configuration for SBFs (Lai and Mahin 2014; Martin and Deierlein 2021; Panian et al. 2015; Simpson 2018; Simpson and Rivera Torres 2021). The R configuration was identified as important since the first story in a structure often has a different occupancy and use case than the higher levels, and this difference frequently makes it desirable to have fewer structural elements at that level; the first story must also resist the greatest demand, so the elimination of the first-story BRB is expected to have the greatest effect. The S configuration evaluates the opposite extreme from the R configuration, and is similar to setups in rocking frames and controlled spine frames (Blebo and Roke 2018; Takeuchi et al. 2015).

Three design methods for the BRBs were investigated. For each configuration, the overturning method was evaluated with 5 values of the scale factor γ . The 80% base shear method (Simpson 2018) was evaluated for the X configuration. Sizing the BRBs for the forces from ELF analysis was also evaluated for each configuration.

For the frame-level analysis, the strongback members were designed primarily using simplified modal pushover analysis (SMPA), which was developed for SBFs (Simpson and Rivera Torres 2021). SMPA explicitly accounts for partial yield mechanisms and higher mode effects. A subset of frames were also designed using generalized modified modal superposition (GMMS), an alternative design method primarily developed for use with rocking frames (Martin and Deierlein 2021). For the standard X configuration, GMMS produces more conservative designs than SMPA (Talley et al. Forthcoming). The frames were analyzed using nonlinear dynamic analysis.

4.3 Parametric Study of Configurations

To evaluate the overturning method for a broad range of building configurations, strongback braced frames were designed for prototype buildings of varying heights. To isolate the effect of BRB size, frame elements other than the braces were modeled with high stiffness, essentially rigid, elastic elements. The frames were

then analyzed as simple 2D OpenSees models using dynamic response history analyses.

4.3.1 Methods

Buildings of 2, 4, 6, 8, 10, and 12 stories were evaluated for each of the three BRB configurations, and for each of the three BRB design methods. The buildings were selected to be symmetric-plan office buildings, designed for FEMA P695 seismic design category $D_{\max}$ (FEMA 2009). All buildings have a first story height of 5.5 m (18 ft) and a typical story height of 4.6 m (15 ft). The seismic weight of the buildings is 9740 kN (2190 kip) for the first story, 9650 kN (2170 kip) for a typical story, and 9250 kN (2080 kip) for the roof. For base shear calculations, the response modification factor R was taken as 8 (Simpson 2018). The typical floor plan is shown in Fig. 4.2.

Design parameters that depend on the number of stories, N , are given in Table 4.1. The fundamental period listed in Table 4.1 is the ASCE 7-22 approximate fundamental period $T_a = C_t h_n^x$ (ASCE 2022), modified as described in FEMA P695 to include the upper limit on calculated period C_u (FEMA 2009), where h_n is the ground-to-level height of the roof in meters, and C_t and x are empirical factors tabulated in ASCE 7-22 based on the frame type. Here, the values for buckling-restrained braced frames are used, with $C_t = 0.0731$ ($C_t = 0.03$ when h_n is in feet) and $x = 0.75$. For seismic design category $D_{\max}$, $C_u = 1.4$.

Design of Buckling-Restrained Braces

Three design methods were used to design the BRBs for the study: (1) proportioning BRBs based on forces from equivalent lateral force (ELF) analysis; (2) sizing BRBs using the 80% base shear rule (Simpson 2018); and (3) an overturning-based design procedure developed for this work.

The overturning design procedure used here was originally introduced by Talley et al. (2023). In that work, capacity-level forces were used in the overturning procedure; here, design-level forces are used. By using design-level forces, the procedure aligns better with the design intent to produce BRB sizes appropriate to resist the overturning force generated by the design earthquake, and allows the designer to produce BRB sizes prior to coordination with the BRB manufacturer.

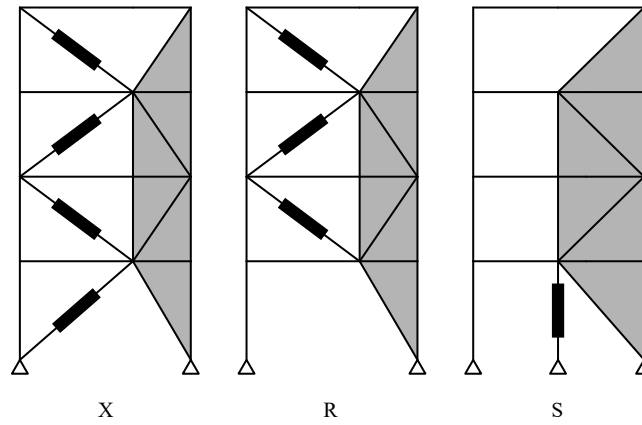


Figure 4.1: BRB configurations evaluated.

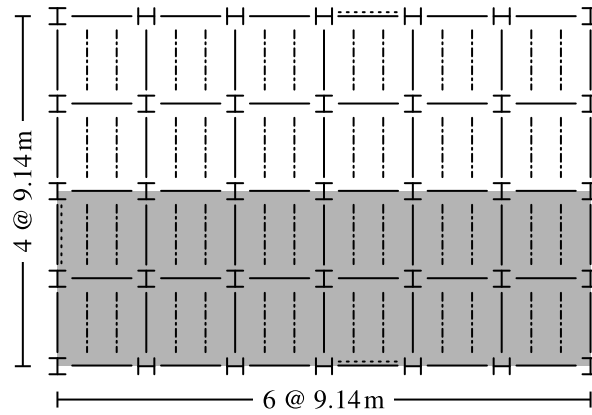


Figure 4.2: Typical floor plan. Strongback bays are indicated by dashed lines; dash-dotted lines indicate joist locations. Shaded area indicates tributary area for the bottom strongback braced frame.

Numerical Model

The numerical model of the structure was built in the OpenSees finite element framework (McKenna et al. 2010). A schematic of the model is shown in Fig. 4.3. Analyses were performed in two dimensions of a single strongback braced frame representing half the building. Elastic beam elements with high stiffness— $EA = 4.4 \times 10^{11}$ kN (10^{10} kips), $EI = 2.9 \times 10^{16}$ kN mm² (10^{12} kip in²)—were used for the strongback. Elastic truss elements with high stiffness— $EA = 4.4 \times 10^{11}$ kN (10^{10} kips)—were used for the beams, columns, and leaning columns. The BRBs were modeled with truss elements with an axial force-displacement relationship calibrated to experimental data and including the effects of low-cycle fatigue (Simpson 2018). Geometric nonlinearity was included at the element level using a corotational transformation for beam-column elements and corotational truss elements. Mass corresponding to half the building's seismic weight was lumped at each story on the nodes of the leaning column. Gravity load equal to half the building's seismic weight at each level was included as point loads on the nodes of the leaning column. Rayleigh damping of 2.5% at periods $2T_1$ and $0.2T_1$ was included, where T_1 is the first-mode period determined by eigenvalue analysis of the model.

The strongback in the simplified model is rigid. This was done to avoid variability that arises due to the strength and stiffness of the strongback itself: a stiffer strongback more effectively distributes demands, and insufficient strength leads to member failures and loss of overall stability (Talley et al. Forthcoming). By selecting a rigid strongback, failure of the strongback is not considered in the analysis. As discussed later in the frame-level analysis section, it may not be feasible for finite strength and stiffness strongbacks to support certain brace configurations.

Analysis Procedure

Incremental dynamic analysis was performed on all frame configurations using the 22 ground motion pairs from the FEMA P695 far-field set (FEMA 2009). Collapse was not directly simulated, and was considered to occur when the maximum story drift ratio, $\theta_{\max}$, exceeds 5%. The ground motions were scaled according to FEMA P695 Appendix A:

$$\ddot{x}_g = SF \cdot \frac{S_{MT}}{\hat{S}_{NRT}} \cdot NM \cdot \ddot{x}_{g,recorded} \quad (4.1)$$

Table 4.1: Prototype building parameters

N	Roof height, h_n (m)	Fundamental period, $C_u T_a$ (s)	Seismic weight, W (kN)	ELF base shear, V (kN)	MCE spectral demand, S_{MT} (g_0)
2	10.1	0.578	19000	2380	1.50
4	19.2	0.939	38300	3060	0.958
6	28.3	1.26	57600	3440	0.716
8	37.5	1.55	76900	3720	0.580
10	46.6	1.83	96200	4230	0.493
12	55.8	2.09	116000	5080	0.431

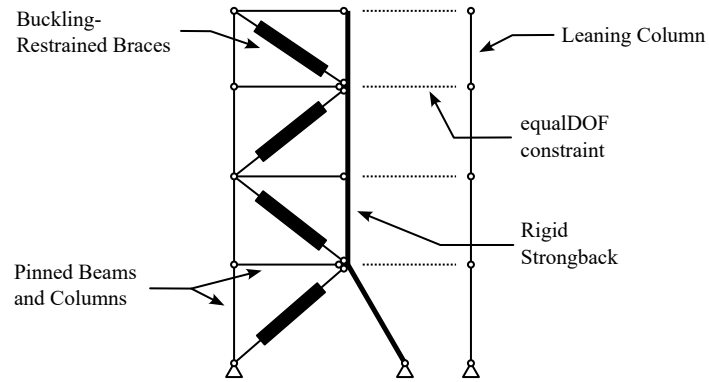


Figure 4.3: Schematic of the numerical model used for parametric study (4-story X configuration shown)

where SF is a selectable scale factor, S_{MT} is the spectral intensity of the MCE at the fundamental period of the building, $\hat{S}_{NRT}$ is the median spectral intensity of the normalized record set at the fundamental period of the building, and NM is the FEMA P695 normalization factor for the specific ground motion (FEMA 2009). SF was incremented with a step of 0.25 until the collapse condition was met. The intensity used for collapse was then calculated by linearly interpolating between the final non-collapse analysis and the first analysis that caused collapse.

Collapse Evaluation

Collapse probability was evaluated by computing the adjusted collapse margin ratio, $ACMR$, and the total system uncertainty, β_{TOT} . The $ACMR$ requires calculation of the spectral shape factor, SSF , which accounts for the beneficial effects of spectral shape. Both SSF and β_{TOT} are dependent on the period-based ductility, μ_T , which is normally calculated by static pushover analysis; however, for sufficiently ductile frames ($\mu_T > 8.0$) both SSF and β_{TOT} are constant with respect to μ_T . Values of SSF and β_{TOT} for the FEMA P695 far-field record set are tabulated in FEMA P695 (FEMA 2009). Previous analysis of SBFs has shown them to have μ_T well in excess of 8.0 (Talley et al. Forthcoming). Thus, it is assumed here that μ_T is greater than 8.0 for all configurations. β_{TOT} also requires qualitative ratings of the design criteria, modeling criteria, and test data; these are rated as “good” (B), except for the model which is rated as “fair” (C). The collapse probability at MCE is then calculated from the assumed lognormal cumulative distribution function used by FEMA P695:

$$p_{collapse,MCE} = \frac{1}{2} \text{erfc} \left(\frac{\ln ACMR}{\sqrt{2}\beta_{TOT}} \right) \quad (4.2)$$

where erfc is the complementary error function.

4.3.2 Results & Discussion

Buckling-Restrained Brace Designs

BRB sizes resulting from the design procedures are tabulated in Table 4.2, which lists the BRB sizes when designed using ELF, and Table 4.3, which lists the BRB sizes for the overturning and 80% base shear procedures. Sizes were rounded to three significant figures. For the overturning method, only BRB sizes

corresponding to $\gamma = 1.0$ are listed; since the method is linear, other BRB sizes may be obtained by multiplying the tabulated size by the desired scale factor. Note that the ELF procedure is the only procedure evaluated here that results in differently-sized BRBs at each story; the others produce a single shared BRB size.

The overturning design method using $\gamma = 1.0$ (O100) and ELF produce very similar BRB designs in terms of total area. For S frames, the designs produced are identical. The 80% base shear rule produces BRBs around 45% larger than O100 for $N = 2$, and around 55% larger for $N = 4$ through $N = 12$. Comparing the different brace configurations, X frames had the lowest total BRB area, with R frames having between 1% and 7% more total BRB area and S frames having between 21% and 29% more total BRB area. For both R and S frames, the increase from the corresponding X frame was greater for shorter buildings.

Collapse Probability

The collapse probability of each frame, quantified using incremental dynamic analysis of the simple model, is shown in Fig. 4.4. In this work, a 10% probability of collapse at MCE is considered the maximum allowable collapse probability for acceptable collapse performance; this limit is based on FEMA P695 Appendix B for collapse evaluation of individual buildings (FEMA 2009).

In general, collapse probability decreases with both scale factor and height. No design method produced BRBs which led to unacceptable collapse probability ($> 10\%$ at MCE). It should be noted that this result relies on the strongback remaining stiff and strong. In this model, the strongback is rigid at all times, but reduced strength or stiffness of the strongback would lead to increases in collapse probability (Talley et al. Forthcoming).

Collapse probabilities for frames designed using $\gamma = 1.0$ are approximately equivalent to those designed using ELF. This is expected, due to these two methods being designed to resist the same overturning demand, as well as the total BRB area being similar. Collapse probability when using the 80% base shear rule corresponds to the overturning method with γ approximately equal to 1.5, regardless of the height of the building.

Response when designed using either ELF or the overturning method is similar for all three brace configurations. This indicates that there is no natural weakness or strength of any of the three brace configurations, and that a sufficiently stiff and strong strongback can bridge the weak stories introduced by the missing braces.

Collapse probability was higher for shorter buildings. This trend is similar to that observed for reinforced

Table 4.2: Designed BRB sizes, for frames with height-varying size (mm²)

Frame	Story											
	1	2	3	4	5	6	7	8	9	10	11	12
X-2-E	4230	2280										
R-2-E		6970										
X-4-E	5540	4270	3390	1950								
R-4-E		7100	5550	3170								
X-6-E	6130	5100	4800	4150	3140	1730						
R-6-E		7100	6520	5610	4210	2320						
X-8-E	6380	5510	5520	5290	4810	4040	2970	1610				
R-8-E		7030	6900	6520	5880	4920	3610	1950				
X-10-E	6770	6020	6260	6340	6190	5820	5190	4280	3110	1660		
R-10-E		7350	7480	7480	7230	6710	5920	4880	3530	1880		
X-12-E	7160	6650	7160	7550	7740	7740	7420	6900	6080	4970	3570	1890
R-12-E		7940	8320	8650	8710	8580	8190	7550	6580	5370	3860	2040

Table 4.3: Designed BRB sizes, for frames with single BRB size (mm²)

Brace Pattern	Design Method	$N = 2$	$N = 4$	$N = 6$	$N = 8$	$N = 10$	$N = 12$
X	Overturning, $\gamma = 1.0$	3310	3840	4210	4540	5180	6250
	80% base shear rule	4740	5980	6650	7230	8190	9810
R	Overturning, $\gamma = 1.0$	6970	5260	5150	5260	5830	6900
S	Overturning, $\gamma = 1.0$	8390	19000	30900	44300	62900	91000
	Equivalent lateral force	8390	19000	30900	44300	62900	91000

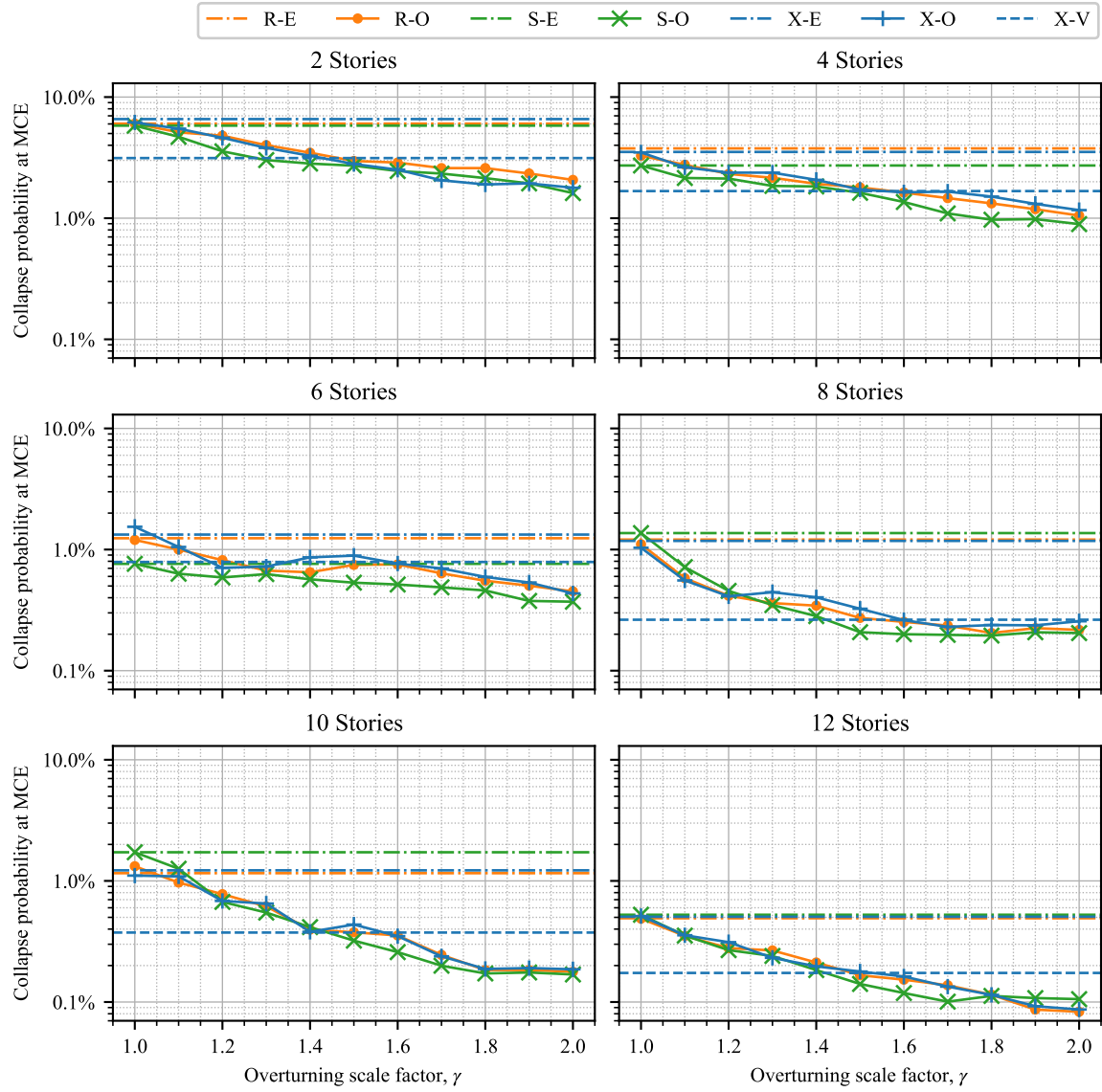


Figure 4.4: Collapse probability at MCE of frames designed and analyzed using the simple model

concrete shear walls, steel concentrically braced frames, and buckling-restrained braced frames (NIST 2010). Within each grouping of building heights, longer periods—i.e., less stiff, and having smaller BRBs—correspond to a higher probability of collapse (see Fig. 4.5).

BRB size was most impactful for shorter buildings. For $N = 2$, doubling the scale factor, and thus the BRB size, reduced the collapse probability at MCE by around 4 percentage points, approximately a 2/3 reduction; for $N = 4$, collapse probability was reduced by around 1.5 percentage points. For taller frames, however, the collapse probability was already quite low, and while increasing the BRB size reduced the collapse probability further, the absolute decrease was smaller. For $N = 8, 10$, and 12 , a leveling-off was observed where increasing BRB size provided minimal to no decrease in collapse probability. However, these results were obtained with a rigid strongback; designing the strongback members to achieve this level of performance may not be practical, especially for tall structures.

The results of this section indicate that the overturning method provides comparable collapse safety to more established design procedures, and that BRBs sized for ELF-level demands (as opposed to the increased demand used by the 80% rule) provide sufficient collapse capacity for strongback braced frames. However, the BRBs are only one part of a hybrid system, and the strength and stiffness of the strongback itself are critical components of the system response.

4.4 Evaluation by Nonlinear Frame Level Analysis

Frame-level design and analysis was performed on a subset of the buildings from the parametric study. This analysis was done (1) to validate that acceptable frame designs can be produced, especially for the R and S configurations, and (2) to provide further insight into the behavior of the resulting frames.

4.4.1 Methods

Numerical Model

OpenSees models (McKenna et al. 2010) were used for both design and analysis. The models were originally developed in other work (Talley et al. [Forthcoming](#)); further details may be found there.

For design by SMPA, the model shown in Fig. 4.6 uses the “PP” set of constitutive relations proposed

by Simpson and Rivera Torres (2021). In this configuration, the BRBs and shear tabs are modeled with elastic-perfectly plastic response. The wide flange members are modeled using force-based beam-column elements with fiber sections, with two elements per member and five integration points per element. The material response of the fibers is elastic; this setup is used to ensure that the initial stiffness of the design model is the same as that of the nonlinear analysis model. Tributary loads and mass are carried by leaning columns. Geometric nonlinearity is incorporated through the use of corotational truss elements for the BRBs and leaning columns, and corotational geometric transformations for the beam-column elements. Rigid offsets are incorporated for all elements based on assumed gusset plate geometry.

For the nonlinear time history analysis, the SMPA model was modified to include additional sources of nonlinearity. The material response of the BRBs was changed to a calibrated uniaxial material including asymmetric response (using the Steel4 uniaxial material) and low-cycle fatigue developed by Simpson (2018). The fibers in the wide flange members were changed to use a Giuffr -Menegotto-Pinto model (Steel02), with residual stresses represented using a Lehigh pattern (Galambos and Ketter 1959). The nonlinear time history model is not suitable for progressive collapse modeling, as it lacks important degradation modes such as fracture of the wide flange braces and lateral-torsional buckling of the columns. However, it does incorporate sufficient degradation through brace buckling, BRB fracture, and P-Δ effects to be sufficient for the purposes of evaluating non-simulated collapse states.

The GMMS design model is distinct from the SMPA and nonlinear time history models. Unlike these models it is entirely linear, with the BRBs and beam links replaced by equivalent stiffness springs, and other elements modeled using workpoint-to-workpoint members. Further details on the calculation of the equivalent stiffnesses may be found in Martin and Deierlein (2021) and Talley et al. (Forthcoming).

Design of Buckling-Restrained Braces

Sizes for the BRBs in the frame-level model were taken from those developed for the parametric study, with the exception of those designed proportional to ELF analysis since the frame-level model provides a more realistic estimation of the stiffness of the frame. For designing the BRBs using ELF, initial strongback member sizes had to be selected in order to perform the analysis. The initial member sizes for the strongback were taken from Talley et al. (Forthcoming). After performing the strongback member design, the BRBs were

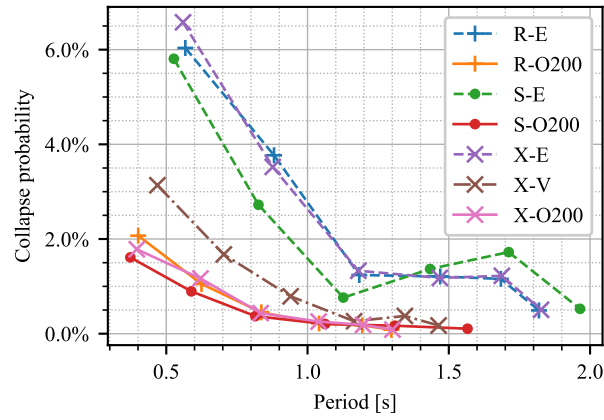


Figure 4.5: Collapse probability at MCE versus first mode period of simple models

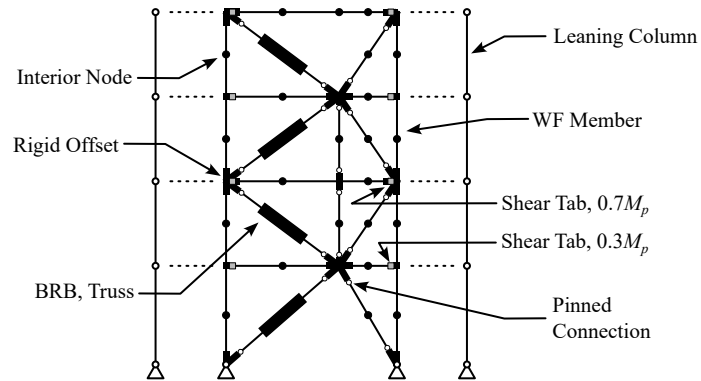


Figure 4.6: Schematic of pushover model for SMPA procedure

checked again to ensure they still met the strength requirements.

Design of Strongback Members

Members in the strongback were primarily designed using simplified modal pushover analysis (SMPA), which accounts for SBF-specific behaviors, such as contributions from higher modes (Simpson and Rivera Torres 2021). SMPA determines the design member forces using a combination of a displacement-controlled 1st-mode pushover analysis and load-controlled higher-mode static analyses. SMPA may be used with a variety of modeling choices; here, the “PP” set of materials is used, where force-controlled actions are modeled elastically, and displacement-controlled actions are modeled using elastic-perfectly plastic materials. Expected strengths are used in the 1st mode, and nominal strengths are used in the higher modes.

SMPA has only been evaluated for use with SBFs in the X configuration. Because it directly models higher mode effects and force redistribution due to nonlinear actions, it was expected that SMPA would provide appropriate estimations of forces for the other brace configurations studied. As a secondary design method, the generalized modified modal superposition (GMMS) method (Martin and Deierlein 2021) was used to provide alternate frame designs for the X-150, R-150, and S-150 frames.

All frames were also evaluated using the equivalent lateral force method to ensure they met ASCE 7-22 story drift and stability coefficient requirements (ASCE 2022). A story drift limit of 0.02 and a stability coefficient limit of 0.1 were used for all frames and stories. Frame members were also checked for AISC 341-22 ductility requirements, with beams restricted to high ductility, and other members restricted to moderate ductility (AISC 2022).

Nonlinear Analysis

To evaluate the resulting designs, nonlinear static pushover and nonlinear dynamic analyses were performed following the recommendations in FEMA P695 (FEMA 2009). The same analysis procedure as was used for the simplified model was used for the frame-level model, with the following changes: nonlinear static pushover was used to verify that μ_T exceeds 8.0 for all designs, and the model rating was increased from “fair” (C) to “good” (B) due to the increase in model fidelity. The same 5% story drift limit was used to indicate collapse, as the model does not incorporate modeling of progressive collapse, and the same ground motion

scaling and ground motion set were used.

4.4.2 Results and Discussion

Frame Designs

Member sizes are tabulated in Tables 4.4 to 4.6.

When using ELF to design the BRBs, the resulting sizes are similar to the those determined using the simple frame for the R and S configurations. For the X configuration, the largest BRB is, unusually, in the second story. The total BRB area is 10–20% larger when using ELF with the frame model than when using ELF with the simplified model.

For the R and S configurations, when $\gamma < 1.50$, the smaller BRBs meant that designing the frames for strength alone was insufficient, as stability and drift limits were triggered in the first story when strongback members were designed solely for strength limits. The first story strongback brace and the tie braces were increased to meet the stability and drift requirements.

For S configurations, demands on the column opposite the strongback were significantly reduced, as no BRBs frame into it. S configurations see greater strength demands on upper-story strongback braces compared to R and X configurations, but even with these greater strength demands S configurations can have issues meeting story drift limits in the upper stories. As BRB size increases with increasing γ , the drift demand in the upper stories decreases, while strength demands on the first story brace and column increase.

For X configurations, increasing BRB size did not lead to large increases in strongback member sizes, with only the first-story strongback brace increasing across the range investigated. This is in part due to reserve capacity in the strongback members, resulting from the larger step in size between W12 members at the range investigated.

The frames designed using GMMS have significantly larger members than the corresponding frames designed using SMPA. To meet strength demands for the first story strongback brace, the GMMS frames had to be designed using W14 sections instead of W12. The increase in demand was largest for the first story strongback braces and the tie braces: for first story braces, GMMS predicts 85–100% greater axial demand than SMPA does for X-150 and R-150, and 50% greater axial demand for S-150. For X-150 and R-150, the increase in demand in the GMMS analysis is across all modes, while for S-150 the increase in demand is

associated with the first mode, with GMMS and SMPA providing similar predictions for contributions from higher modes.

4.4.3 Nonlinear Analysis

Collapse probability at MCE for the frame-level model is shown in Fig. 4.7.

For the X configuration, collapse probability is acceptable for all methods of sizing the BRBs, and is not highly sensitive to BRB size. Doubling γ reduces collapse probability from 6% to 5%, and this reduction in collapse probability may primarily be associated with the correspondingly small increases in strongback strength and stiffness.

For the R and S configurations, the collapse probability at MCE is much higher than for X configurations. R and S frames with BRBs designed using ELF had lower collapse probabilities than those designed using the overturning method, except for S-100 and S-125. S-100 and S-125 have a lower collapse probability than the S frames with larger BRBs, most likely due to the increased strongback member sizes required to meet drift requirements (as opposed to strength requirements).

For all frames, collapse (as measured by story drift) was preceded by failure of the strongback braces. At the collapse intensity, fracture of the BRBs was solely due to excessive strain (as opposed to low-cycle fatigue), and such fracture only occurred after failure of the strongback. Fig. 4.8 shows histograms of which stories triggered the drift-based collapse limit. For the R configuration, collapse predominantly occurred due to failure of the first story strongback brace, except for S-100 and S-150-GMMS, which had almost no first story failures and instead a higher proportion of failures in the second and third stories. These setups had relatively larger first story braces—S-100 due to ELF-based drift limits and S-150-GMMS due to increased demands from the GMMS analysis. For the S configuration, most failures occurred in the upper two stories, except for S-100, S-125, and S-ELF. S-100 and S-125, similar to R-100, had upper story members controlled by drift limits, leading to larger member sizes for those members relative to the rest of the frame. For the X configuration, failure was evenly split between the first and second stories, except for X-150-GMMS and X-ELF, which saw reduced numbers of first-story failures and a higher proportion of upper story failures, especially the third story.

Fig. 4.9 shows the story shear demands from different design methods, as well as the peak story shear from

Table 4.4: Member sizes for designed frames (“R” configuration)

Building	Story	BRB Size [in ²]	Beam	Left Column	Strongback Column	Strongback Brace	Tie Brace
R-100	1	—	W12×96	W12×170	W12×170	W12×279	—
	2	5290 (8.2)	W12×96	W12×170	W12×170	W12×58	W12×136
	3	5290 (8.2)	W12×96	W12×79	W12×79	W12×58	W12×136
	4	5290 (8.2)	W12×96	W12×79	W12×79	W12×79	—
R-125	1	—	W12×96	W12×190	W12×170	W12×210	—
	2	6650 (10.3)	W12×96	W12×190	W12×170	W12×58	W12×120
	3	6650 (10.3)	W12×96	W12×79	W12×79	W12×58	W12×120
	4	6650 (10.3)	W12×96	W12×79	W12×79	W12×79	—
R-150	1	—	W12×96	W12×210	W12×170	W12×230	—
	2	7940 (12.3)	W12×96	W12×210	W12×170	W12×58	W12×136
	3	7940 (12.3)	W12×96	W12×79	W12×96	W12×58	W12×136
	4	7940 (12.3)	W12×96	W12×79	W12×96	W12×79	—
R-175	1	—	W12×96	W12×210	W12×170	W12×230	—
	2	9230 (14.3)	W12×96	W12×210	W12×170	W12×58	W12×152
	3	9230 (14.3)	W12×96	W12×96	W12×96	W12×58	W12×152
	4	9230 (14.3)	W12×96	W12×96	W12×96	W12×79	—
R-200	1	—	W12×96	W12×230	W12×170	W12×252	—
	2	10600 (16.4)	W12×96	W12×230	W12×170	W12×58	W12×170
	3	10600 (16.4)	W12×96	W12×96	W12×96	W12×58	W12×152
	4	10600 (16.4)	W12×96	W12×96	W12×96	W12×79	—
R-ELF	1	—	W12×96	W12×190	W12×170	W12×210	—
	2	9810 (15.2)	W12×96	W12×190	W12×170	W12×53	W12×96
	3	5480 (8.5)	W12×96	W12×79	W12×79	W12×53	W12×87
	4	2770 (4.3)	W12×96	W12×79	W12×79	W12×53	—
R-150 (GMMS)	1	—	W12×96	W14×283	W14×176	W14×398	—
	2	7940 (12.3)	W12×96	W14×283	W14×176	W14×61	W14×257
	3	7940 (12.3)	W12×96	W14×109	W14×120	W14×68	W14×257
	4	7940 (12.3)	W12×96	W14×109	W14×120	W14×120	—

Table 4.5: Member sizes for designed frames (“S” configuration)

Building	Story	BRB Size [in ²]	Beam	Left Column	Strongback Column	Strongback Brace	Tie Brace
S-100	1	19000 (29.4)	W12×96	W12×79	W12×190	W12×252	—
	2	—	W12×96	W12×79	W12×190	W12×152	W12×210
	3	—	W12×96	W12×58	W12×136	W12×152	W12×210
	4	—	W12×96	W12×58	W12×136	W12×210	—
S-125	1	23700 (36.8)	W12×96	W12×79	W12×210	W12×279	—
	2	—	W12×96	W12×79	W12×210	W12×152	W12×136
	3	—	W12×96	W12×58	W12×120	W12×152	W12×136
	4	—	W12×96	W12×58	W12×120	W12×152	—
S-150	1	28500 (44.1)	W12×96	W12×79	W12×230	W12×279	—
	2	—	W12×96	W12×79	W12×230	W12×152	W12×136
	3	—	W12×96	W12×58	W12×120	W12×136	W12×136
	4	—	W12×96	W12×58	W12×120	W12×120	—
S-175	1	33200 (51.5)	W12×96	W12×79	W12×230	W12×305	—
	2	—	W12×96	W12×79	W12×230	W12×170	W12×136
	3	—	W12×96	W12×58	W12×120	W12×136	W12×136
	4	—	W12×96	W12×58	W12×120	W12×136	—
S-200	1	37900 (58.8)	W12×96	W12×79	W12×252	W12×336	—
	2	—	W12×96	W12×79	W12×252	W12×190	W12×152
	3	—	W12×96	W12×58	W12×120	W12×152	W12×152
	4	—	W12×96	W12×58	W12×120	W12×136	—
S-ELF	1	21200 (32.8)	W12×96	W12×79	W12×210	W12×252	—
	2	—	W12×96	W12×79	W12×210	W12×152	W12×152
	3	—	W12×96	W12×58	W12×136	W12×152	W12×152
	4	—	W12×96	W12×58	W12×136	W12×152	—
S-150 (GMMS)	1	28500 (44.1)	W12×96	W14×61	W14×370	W14×342	—
	2	—	W12×96	W14×61	W14×370	W14×211	W14×193
	3	—	W12×96	W14×38	W14×120	W14×193	W14×193
	4	—	W12×96	W14×38	W14×120	W14×145	—

Table 4.6: Member sizes for designed frames (“X” configuration)

Building	Story	BRB Size [in ²]	Beam	Left Column	Strongback Column	Strongback Brace	Tie Brace
X-100	1	3870 (6.0)	W12×96	W12×152	W12×152	W12×170	—
	2	3870 (6.0)	W12×96	W12×152	W12×152	W12×79	W12×87
	3	3870 (6.0)	W12×96	W12×79	W12×79	W12×79	W12×87
	4	3870 (6.0)	W12×96	W12×79	W12×79	W12×79	—
X-125	1	4840 (7.5)	W12×96	W12×152	W12×152	W12×170	—
	2	4840 (7.5)	W12×96	W12×152	W12×152	W12×79	W12×87
	3	4840 (7.5)	W12×96	W12×79	W12×79	W12×79	W12×87
	4	4840 (7.5)	W12×96	W12×79	W12×79	W12×79	—
X-150	1	5810 (9.0)	W12×96	W12×152	W12×152	W12×190	—
	2	5810 (9.0)	W12×96	W12×152	W12×152	W12×79	W12×87
	3	5810 (9.0)	W12×96	W12×79	W12×79	W12×79	W12×87
	4	5810 (9.0)	W12×96	W12×79	W12×79	W12×79	—
X-175	1	6770 (10.5)	W12×96	W12×170	W12×152	W12×190	—
	2	6770 (10.5)	W12×96	W12×170	W12×152	W12×79	W12×87
	3	6770 (10.5)	W12×96	W12×87	W12×79	W12×79	W12×87
	4	6770 (10.5)	W12×96	W12×87	W12×79	W12×79	—
X-200	1	7680 (11.9)	W12×96	W12×190	W12×152	W12×190	—
	2	7680 (11.9)	W12×96	W12×190	W12×152	W12×79	W12×87
	3	7680 (11.9)	W12×96	W12×87	W12×79	W12×79	W12×87
	4	7680 (11.9)	W12×96	W12×87	W12×79	W12×79	—
X-ELF	1	3870 (6.0)	W12×96	W12×152	W12×152	W12×190	—
	2	6770 (10.5)	W12×96	W12×152	W12×152	W12×50	W12×79
	3	4520 (7.0)	W12×96	W12×79	W12×87	W12×58	W12×79
	4	2580 (4.0)	W12×96	W12×79	W12×87	W12×79	—
X-V80	1	6000 (9.3)	W12×96	W12×170	W12×152	W12×190	—
	2	6000 (9.3)	W12×96	W12×170	W12×152	W12×79	W12×87
	3	6000 (9.3)	W12×96	W12×79	W12×79	W12×79	W12×87
	4	6000 (9.3)	W12×96	W12×79	W12×79	W12×79	—
X-150 (GMMS)	1	5810 (9.0)	W12×96	W14×211	W14×233	W14×257	—
	2	5810 (9.0)	W12×96	W14×211	W14×233	W14×109	W14×145
	3	5810 (9.0)	W12×96	W14×109	W14×109	W14×109	W14×145
	4	5810 (9.0)	W12×96	W14×109	W14×109	W14×109	—

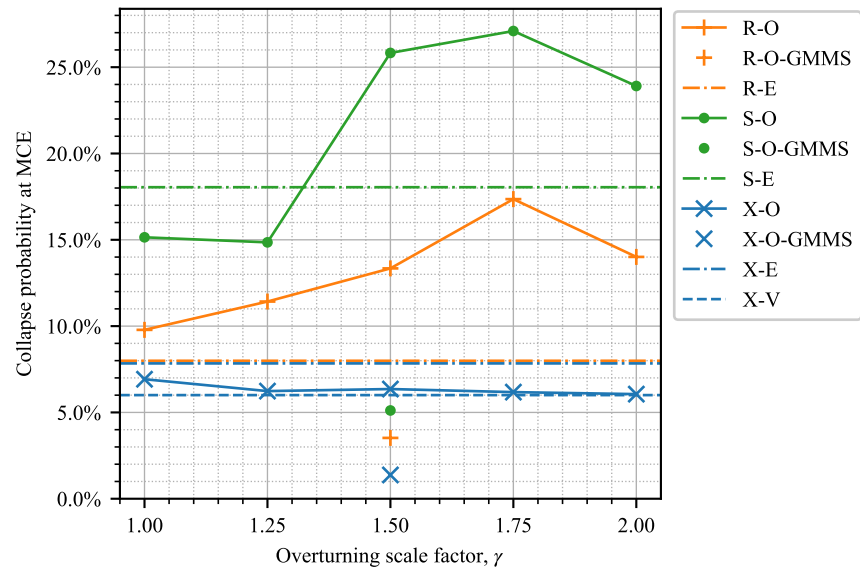


Figure 4.7: Collapse probability at MCE for frame-level models

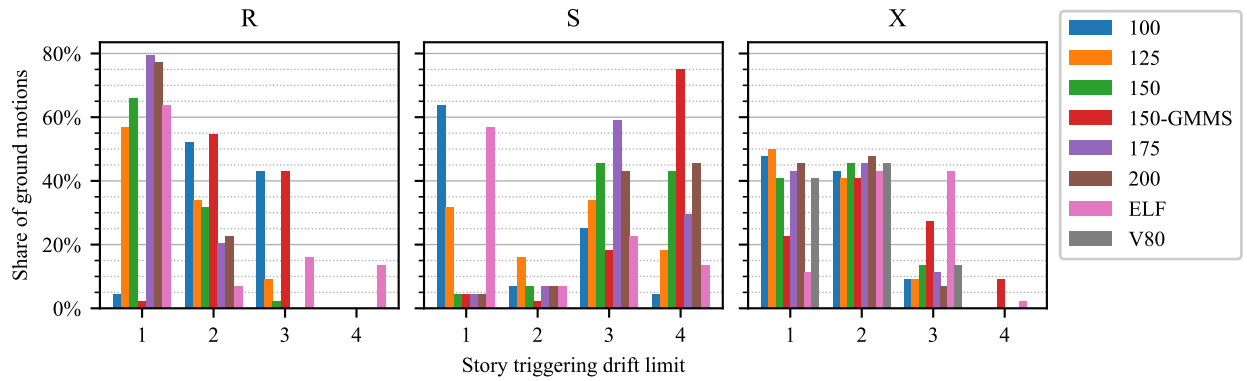


Figure 4.8: Collapse probability at MCE for frame-level models

the nonlinear time history analyses at MCE, for frames R-150, S-150, and X-150. The strongback members in these frames were designed using SMPA, but demands from the GMMS method are shown, as well as demands from using GMMS with a load combination factor of 1.0, instead of the 1.3 factor prescribed by Martin and Deierlein (2021). For all three frames, the SMPA story shear is less than the median MCE story shear; this under-prediction was lowest for X-150, which had the lowest collapse probability of the three. For R-150 and X-150, the GMMS demands exceeded the demands from the entire far-field ground motion set, and demands using a 1.0 load combination factor exceeded the 85th percentile of the MCE demands.

These results show that it is not necessary to meet the median MCE story shear demand to have sufficient collapse capacity: X-150 has a 6.5% probability of collapse at MCE, but the SMPA story shear demands used to design its strongback members are as low as 70% of the median MCE. This 70% mark appears to be a rough lower bound, as the SMPA demands for R-150 and S-150 both peak around this level; this also agrees with seismic design practice, where the design earthquake (DE) is 2/3 the MCE intensity (ASCE 2022). In contrast, the GMMS demands are significantly in excess of the MCE demands; designing with a lower scale factor would likely result in adequate collapse capacity even for S-150. GMMS more closely aligns with the median MCE demands for S-150 than for R-150 and X-150, indicating that GMMS has better predictive power for frames more similar to the rocking frame geometry that it was originally developed for.

Frames designed using GMMS had lower collapse probabilities than any frame designed using SMPA. This agrees with previous work indicating that GMMS is overall more conservative than SMPA for strongbacks (Talley et al. Forthcoming). While not conclusive from this data, GMMS does not appear to capture the differences between the frame configurations better than SMPA, nor does it appear to provide a more consistent collapse probability. Rather, differences between configurations remain, just at lower collapse probabilities, apparently due to GMMS being more conservative than SMPA.

4.5 Conclusions

Strongback braced frames can have non-standard BRB configurations. To realize these configurations in practice, design methods that support these non-standard configurations are necessary. In this work, a method for designing BRBs for SBFs with non-standard configurations is introduced. The method is based on providing sufficient overturning resistance to design earthquake demands. BRBs for frames in three

configurations and a range of heights were designed and evaluated using incremental dynamic analysis of a simplified model with an idealized rigid strongback. BRB designs were also developed using equivalent lateral force and the 80% base shear rule. The following observations were made:

- The overturning method introduced here provides consistent collapse performance for SBFs with missing BRBs.
- Using ELF for BRB sizing provides acceptable collapse performance for SBFs with missing BRBs.
- BRBs designed using the overturning method with a scale factor of 1.0 were sufficient for collapse performance.
- BRBs designed using the 80% base shear rule were approximately the same as those designed using a scale factor of 1.5, and were 50% larger than BRBs that provided sufficient collapse performance.

Additionally, design and analysis was performed using a more detailed model with actual member sizes. These frames were designed using simplified modal pushover analysis (SMPA), and a subset were also designed using generalized modified modal superposition (GMMS). Incremental dynamic analysis of these frames indicates:

- Using a scale factor of 1.0, the overturning method provides acceptable collapse performance for standard SBF configurations. Using the 80% base shear rule, the BRBs are 50% larger and the strongback weighs 5% more while providing similar collapse performance.
- For standard SBF configurations with BRBs at every level, SMPA provides a consistent probability of collapse for a broad range of BRB sizes.
- SMPA may not be appropriate for designing SBFs with missing BRBs, as it underestimates demands and leads to high collapse probabilities. GMMS provides a more appropriate estimate of the demands for configurations with missing BRBs, though this may be due to it being a more conservative method in general.
- Collapse of the SBFs studied here was preceded by buckling of the strongback braces. BRB fracture did not occur until after failure of the strongback.

These results support the usage of SBFs with non-standard BRB configurations, provided that the strong-back can be adequately designed. Further investigation of the difference in predicted demand between SMPA and GMMS is warranted in order to develop a method for designing protected elements in SBFs that is robust for configurations with missing BRBs. Such work would enable the design flexibility of SBFs, and allow broader adoption of these systems.

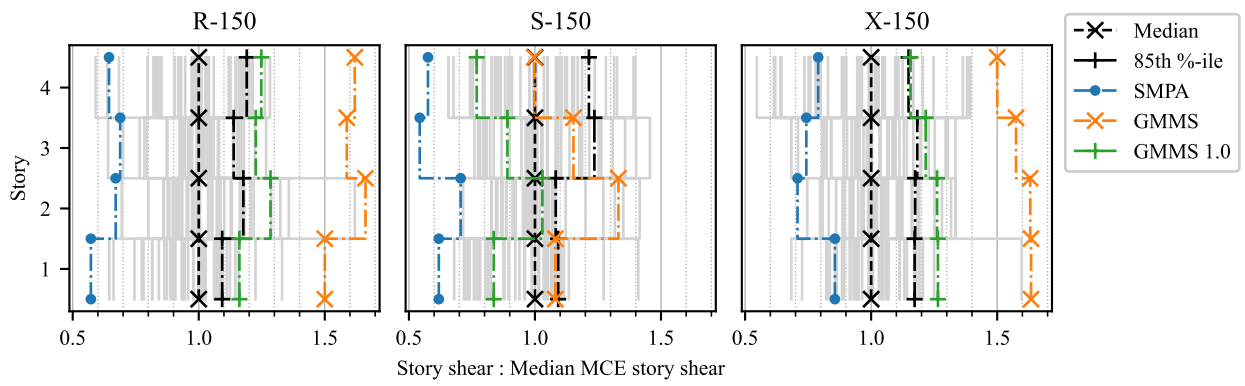


Figure 4.9: Predicted story shear demands for frames designed using $\gamma = 1.50$ for the BRBs and SMPA for the protected elements, relative to the median peak story shear demand at MCE. Light grey lines indicate peak story shear from individual ground motions. GMMS 1.0 indicates the predicted demand from GMMS using a load combination factor of 1.0, instead of the 1.3 factor prescribed by Martin and Deierlein (2021)

CHAPTER 5

Segmented Spines and Supplemental Control Devices for Strongback Braced Frames

5.1 Introduction

Steel concentrically braced frames are susceptible to story mechanisms during earthquakes that can lead to instability and collapse. One class of systems that seeks to remedy this issue is the spine systems, which are hybrid systems that combine energy dissipating elements with a stiff, strong spine that provides vertical continuity and redundant load paths for earthquake demands. Within the spine systems, the strongback braced frame (SBF) has drawn attention for its adaptation of spine system behavior into typical braced frame geometry (Lai and Mahin 2014; Simpson 2018; Tremblay 2003).

Spine systems have their own challenges. In particular, the spine must resist higher mode demands that are not limited by yielding of the frame. These demands can be reduced by using multiple spine segments instead of a single continuous spine. This technique has been studied for use in several different spine systems (Eatherton et al. 2014; Tremblay et al. 2014; Wiebe and Christopoulos 2009), but has not been evaluated for SBFs.

The gap between spine segments offers a convenient location for supplementary vibration control devices. These devices could include conventional hysteretic and viscous dampers, which directly dissipate seismic energy through yielding or fluid action, or inerter-based devices. However, it remains uncertain if useful amounts of energy dissipation can be achieved from devices located at the joint between segments, and if that dissipation can be achieved without losing the benefits of introducing the joint on the required strengths of the strongback.

The objective of this work is to quantify the effects of linking spine segments with supplementary vibration control devices. This is achieved through nonlinear response history analyses of an SBF with a single segment, with two segments, and with two segments connected by various vibration control devices. The results identify promising strategies to improve the structural performance of multi-segment SBFs.

5.2 Methods

All analyses in this work are performed on the same building and same SBF, but with variations in one member. The frame was selected to be seven stories tall so that the segments, created by the removal of a single tie brace, would be of roughly the same height. An elevation view of the frame is shown in Fig. 5.1.

The BRBs were designed to be the same size at each level, with each BRB designed to resist a story shear equal to 80% of the base shear (Simpson 2018).

The other members of the frame were designed assuming that the strongback is formed with a single segment and using generalized modified modal superposition (GMMS) (Martin and Deierlein 2021). Using members from a single-segment design for the analysis of the two-segment strongbacks in this study isolates the differences between configurations to the joint and is a practical necessity especially considering that SBF design methods have not been verified two-segment strongbacks and have not been developed for two-segment strongbacks with vibration control devices at the joint. The member sizes for a single segment strongback are expected to be larger than those required for a multiple-segment strongback. For this study, the use of larger members is acceptable since the strongback is intended to be stiff and strong and remain essentially elastic, however, potential material and cost savings from reduced demands on the strongback braced frame are not evaluated. Similarly, GMMS was used as it results in larger member sizes than other SBF design methods (Talley et al. Forthcoming). Member sizes are listed in Table 5.1. The top-story columns experienced low demands, and so were selected to be the lightest W14 in the same flange width group as the column beneath.

Two classes of supplementary device were analyzed: hysteretic dampers (HD) and rotational inertia viscous dampers (RIVD). HDs dissipate energy similarly to BRBs, through stable hysteretic yielding. RIVDs dissipate energy through viscous damping, and have a supplemental inerter component that provides additional inertia to the structure.

5.2.1 Model

The analysis model was developed in OpenSees (McKenna et al. 2010) and consists of centerline beam-column elements and truss elements. A schematic of the model is shown in Fig. 5.2. The wide flange beams, braces, and columns are represented by beam-column elements using the force-based formulation with fiber sections. The braces and columns were modeled as elastic, as they are intended to remain elastic, and the focus of the study was on overall frame response. The beams were modeled with nonlinear fiber sections, with a Lehigh residual stress pattern (Galambos and Ketter 1959). Shear tabs connecting the beams to the columns were represented using zero-length springs with bilinear response. The shear tab yield rotation was a constant 0.005 radians, and the yield strength was $0.3M_p$ for shear tabs unrestrained by brace gusset plates and $0.7M_p$

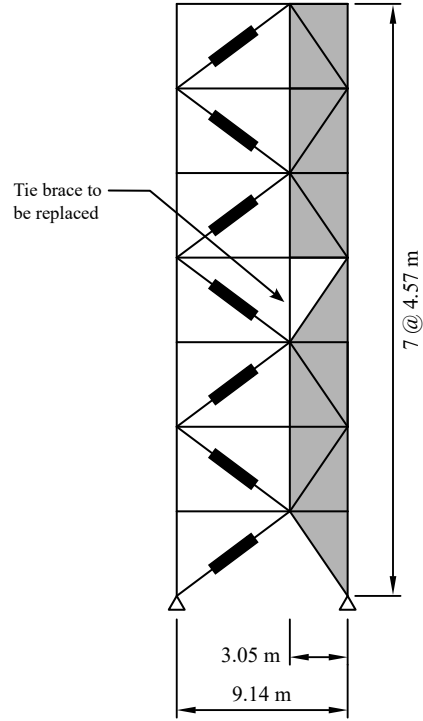


Figure 5.1: Elevation view of studied strongback braced frame

Table 5.1: Designed member sizes

Story	BRB Size [in ²]	Beam	Left Column	Strongback Column	Strongback Brace	Tie Brace
1	7030 (10.9)	W12×120	W14×426	W14×550	W14×342	—
2	7030 (10.9)	W12×120	W14×426	W14×550	W14×233	W14×311
3	7030 (10.9)	W12×120	W14×311	W14×455	W14×193	W14×311
4	7030 (10.9)	W12×120	W14×311	W14×455	W14×145	W14×342
5	7030 (10.9)	W12×120	W14×159	W14×257	W14×176	W14×342
6	7030 (10.9)	W12×120	W14×159	W14×257	W14×145	W14×145
7	7030 (10.9)	W12×120	W14×109	W14×109	W14×132	W14×132

for restrained shear tabs, where M_p is the plastic moment strength of the attached beam (ASCE 2017). The buckling-restrained braces use a calibrated uniaxial material incorporating asymmetric response and low-cycle fatigue (Simpson 2018) to represent their hysteretic response. Rigid offsets are incorporated throughout the model based on member geometry.

The hysteretic dampers were modeled using corotational truss elements with bilinear response with 1% kinematic hardening. The RIVDs were modeled as linear inerters with parallel viscous damping, using the inerterTruss2d element (Talley et al. 2021).

5.2.2 Analysis

The frame was analyzed using nonlinear response history analysis at the maximum considered earthquake (MCE). The FEMA P695 far-field ground motion set was used (FEMA 2009). The ground motions were scaled according to FEMA P695 Appendix A:

$$\ddot{x}_g = \frac{S_{MT}}{\hat{S}_{NRT}} \cdot NM \cdot \ddot{x}_{g,recorded} \quad (5.1)$$

where S_{MT} is the spectral intensity of the MCE at the fundamental period of the building, $T = 1.38$ s, equal to $0.653 g_0$; $\hat{S}_{NRT}$ is the median spectral intensity of the normalized record set at the fundamental period of the building, equal to $0.261 g_0$; and NM is the FEMA P695 normalization factor for the specific ground motion (FEMA 2009).

The primary results from each response history analysis are: the maximum story drift ratio; the story drift concentration, measured as the maximum “marginal drift”; the energy dissipated by the BRBs, beams, and dampers, as well as by Rayleigh damping; peak story accelerations; peak axial and moment demands in strongback members; and peak base shear and overturning moment. The energy dissipated by elements was calculated as the work done by each element’s forces over the element’s deformation. For the BRBs, beams, and hysteretic dampers, this means that a small portion of the energy is attributable to elastic deformation that is not strictly dissipated, but instead stored in the element. Energy dissipated by Rayleigh damping was calculated similarly, as the work done by the Rayleigh damping reaction forces at each node over the displacement of the node.

5.3 Results and Discussion

5.3.1 Hysteretic Damper

Figs. 5.3 to 5.9 show the mean response from the 44 ground motions, for varying damper stiffness, k , and yield strength, P_y . A total of 441 combinations of k and P_y were evaluated (21 each). The “x” indicates k and P_y corresponding to that of the original tie brace in the frame:

$$k_0 = \frac{AE}{L} \quad (5.2)$$

$$P_{y0} = F_y A \quad (5.3)$$

where A is the gross cross-section area of the tie brace, equal to 65 200 mm² (101 in²); E is the elastic modulus of the tie brace, equal to 200 GPa (29 000 ksi); F_y is the yield strength of the tie brace, equal to 345 MPa (50 ksi); and L is the workpoint-to-workpoint length, equal to the story height of 4.57 m (15 ft). Each figure shows, from left to right: the response value, the response value relative to the response of the single segment SBF, and the response value relative to the two segment SBF. The relative response quantities were calculated as the relative difference from the corresponding base frame:

$$r_{rel}(\%) = 100 \frac{r_{damper} - r_{base}}{r_{base}} \quad (5.4)$$

where r_{damper} is the response of the frame with the damper and r_{base} is the response of the one- or two-segment frame.

Fig. 5.3 shows the maximum story drift ratio. Generally, replacing the elastic tie brace with a hysteretic damper results in increased drift. For HDs that are both stiffer and stronger than the original brace, no change is seen in the drift response compared to the original frame. A band of minimal response is seen for HDs slightly weaker than the tie brace, with $P_y \approx 2000$ kip, and $k > 6000$ kip/in, with slight reductions in drift around 2.5% compared to the one-segment frame. HDs reduce drift in all cases compared to the two-segment SBF, converging to the two-segment case for very low strength and stiffness dampers.

Fig. 5.4 shows the drift concentration, calculated as the maximum marginal story drift ratio. Drift

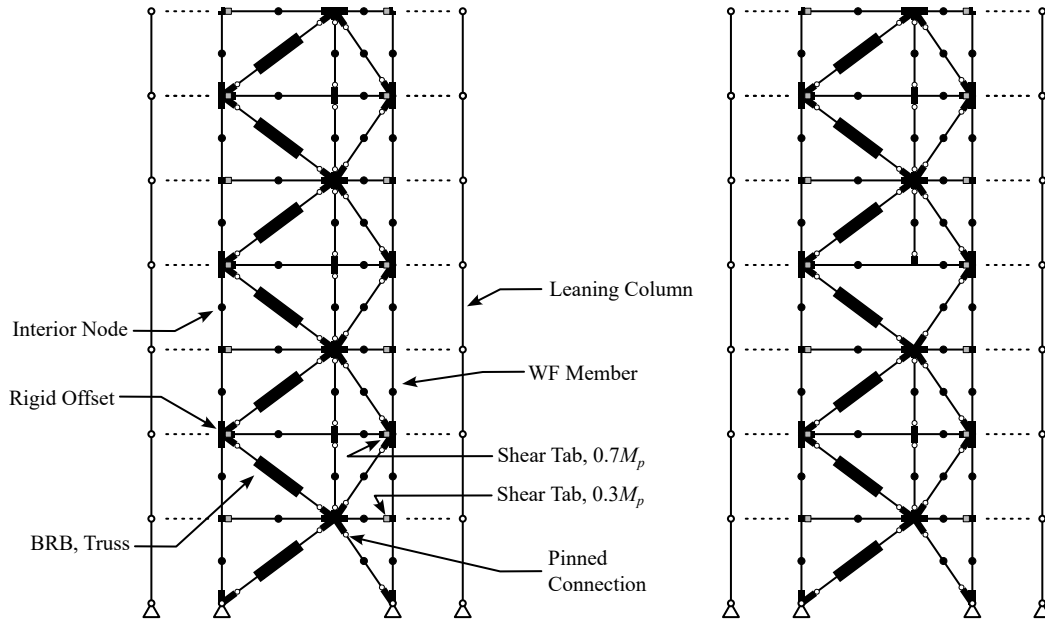


Figure 5.2: Schematic representation of the models. Left: single segment SBF. Right : two-segment SBF.

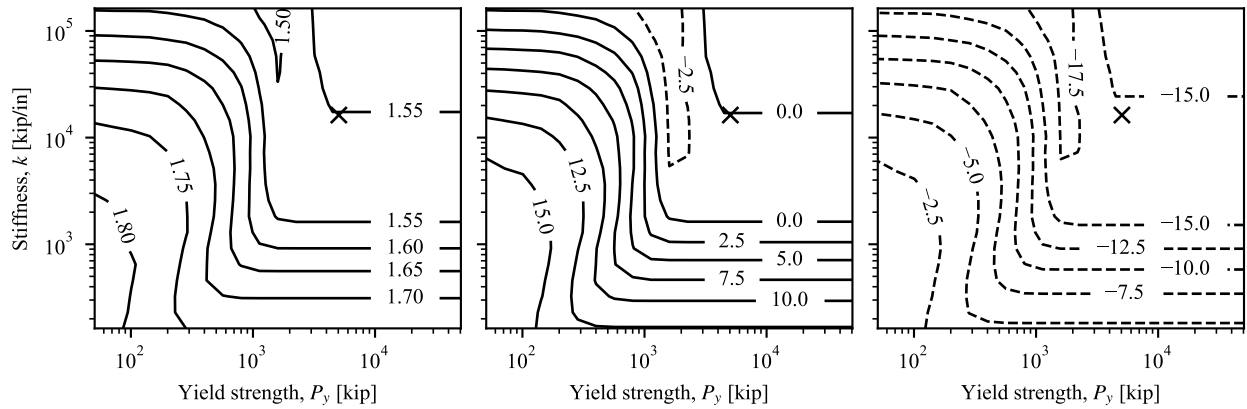


Figure 5.3: Mean value of the maximum story drift ratio (%), for different hysteretic dampers. Left-to-right: mean value, relative to one-segment SBF, relative to two-segment SBF. Dashed contours indicate negative values. × indicates stiffness and strength corresponding to the original tie brace.

concentration remains low for all HDs, indicating that the strongback effectively distributes demand in all cases examined. A band of minimal response is present, similar to maximum drift, but extending to lower stiffnesses.

Fig. 5.5 shows the energy dissipated by the HD, as a percentage of the energy dissipated by the BRBs. A clear peak is seen, though this peak does not align with the minimum drift seen in Fig. 5.3, indicating that optimizing for HD utilization alone would not lead to reductions in overall demands.

Fig. 5.6 shows the mean peak story acceleration. Peak story accelerations are relatively insensitive to the damper parameters, but do increase with stiffer and stronger HDs. HDs that are weaker or less stiff than the original brace reduce peak story accelerations. All frames with HDs had greater peak story accelerations than the two-segment frame.

Fig. 5.7 shows peak axial demand in the strongback braces, and Fig. 5.8 shows the peak moment demand in the strongback columns. Brace axial demands decrease with weaker and less stiff HDs. HDs stiffer and stronger than the original brace increase strongback brace demands. Brace axial demands are lowest for the two-segment spine. Column moment demands greatly increase when using HDs weaker and less stiff than the original brace, and are very high for the two-segment spine, but it should be noted that column moment demands are not capped by yielding in this model, and could be reduced by an explicit pin joint between the two segments.

Fig. 5.9 shows the peak base shear, as a ratio of the ELF base shear $V = 1830 \text{ kN}$ (411 kip). Peak base shear increases with increasing HD stiffness and strength. Using HDs always increases the base shear compared to the two-segment strongback response, converging to the two-segment response for low strength and stiffness.

Overall, adding HDs to two-segment SBFs provides a continuous range between single-segment and two-segment response. A potential “sweet spot” for HD parameters exists for HDs that are stiffer but have slightly reduced yield strengths compared to the tie brace designed for the single-segment frame, where peak story drifts and accelerations, as well as axial demands in the strongback braces are reduced compared to the single-segment SBF. This combination of parameters does lead to large increases to the strongback column demands, which could be mitigated through explicit detailing of the joint connecting the segments. HDs also provide benefit over the two-segment SBF. The two-segment frame shows the greatest reduction in strongback brace demand, base shear, and peak story accelerations, but has greater peak story drift and

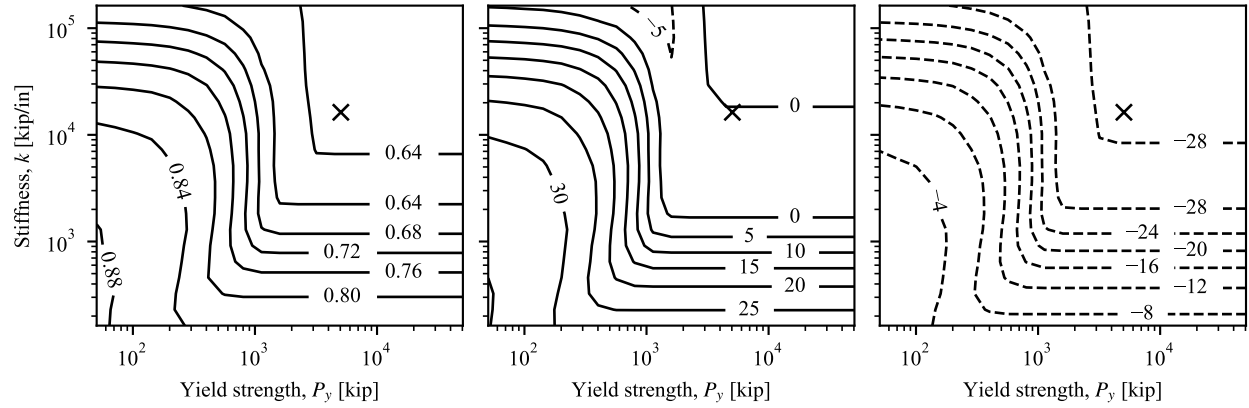


Figure 5.4: Mean value of the maximum marginal story drift ratio (%), for different hysteretic dampers. Left-to-right: mean value, relative to one-segment SBF, relative to two-segment SBF. Dashed contours indicate negative values. $\times$ indicates stiffness and strength corresponding to the original tie brace.

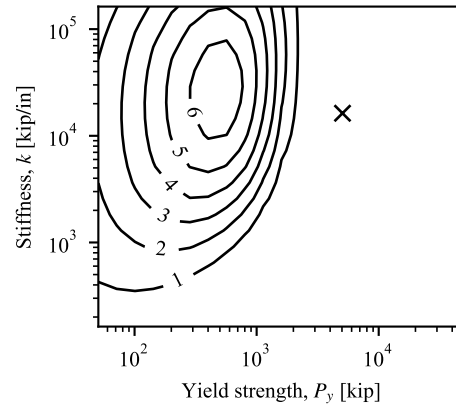


Figure 5.5: Mean value of the energy dissipated by the damper (% of BRB dissipation), for different hysteretic dampers. $\times$ indicates stiffness and strength corresponding to the original tie brace.

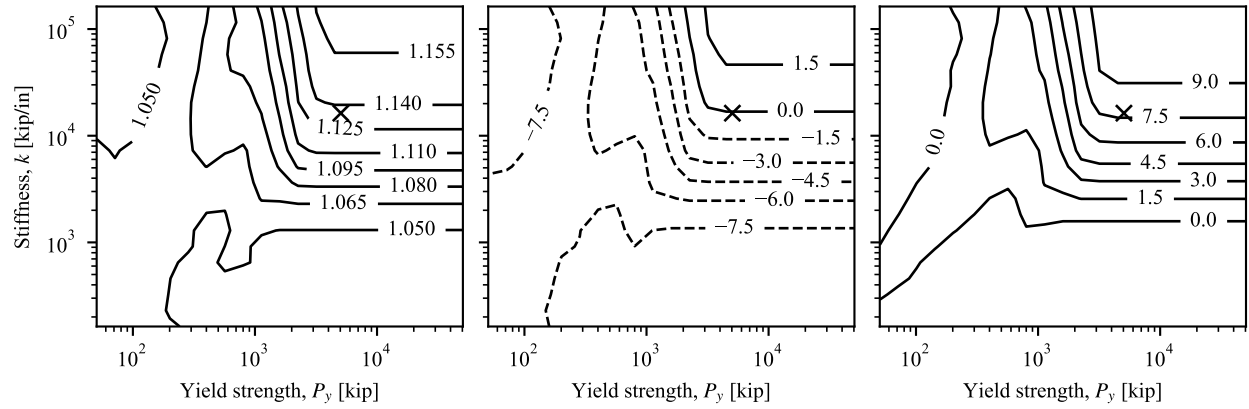


Figure 5.6: Mean value of the peak story acceleration (g), for different hysteretic dampers. Left-to-right: mean value, relative to one-segment SBF, relative to two-segment SBF. Dashed contours indicate negative values. $\times$ indicates stiffness and strength corresponding to the original tie brace.

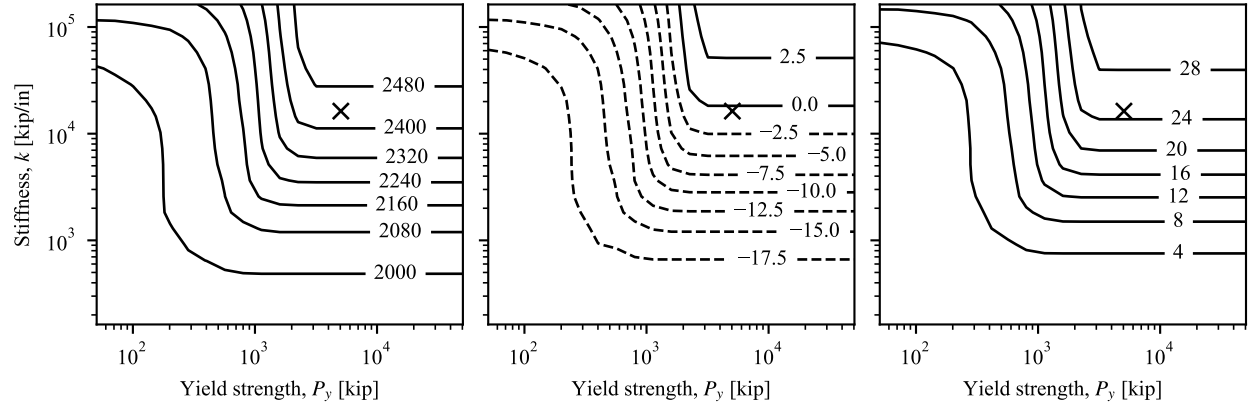


Figure 5.7: Mean value of the peak axial demand (kip) in the strongback diagonal braces, for different hysteretic dampers. Left-to-right: mean value, relative to one-segment SBF, relative to two-segment SBF. Dashed contours indicate negative values. $\times$ indicates stiffness and strength corresponding to the original tie brace.

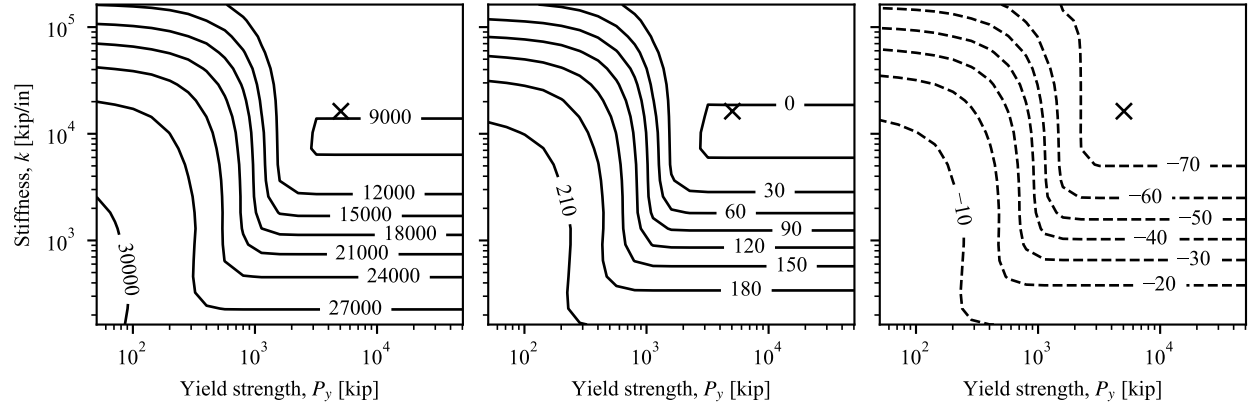


Figure 5.8: Mean value of the peak moment demand (kip-in) in the strongback column, for different hysteretic dampers. Left-to-right: mean value, relative to one-segment SBF, relative to two-segment SBF. Dashed contours indicate negative values. $\times$ indicates stiffness and strength corresponding to the original tie brace.

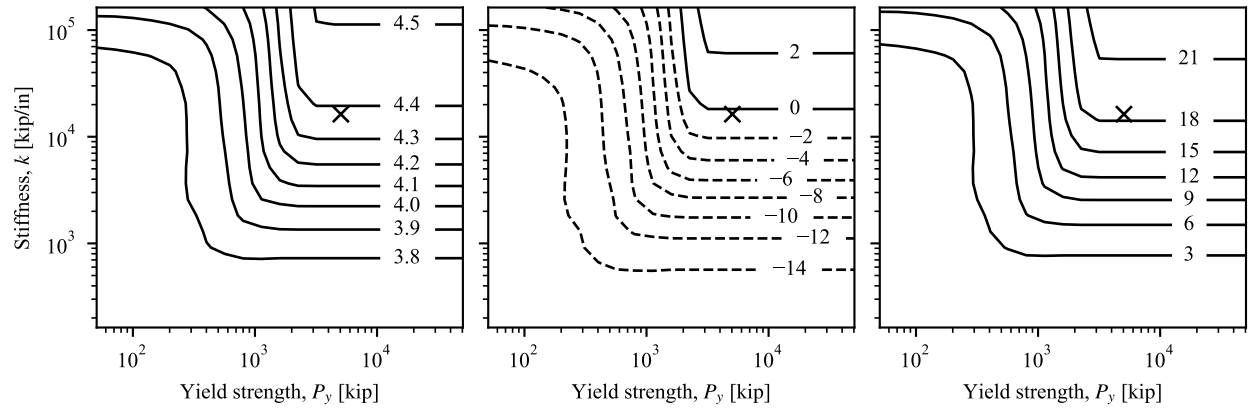


Figure 5.9: Mean value of the peak base shear (ratio of ELF base shear V), for different hysteretic dampers. Left-to-right: mean value, relative to one-segment SBF, relative to two-segment SBF. Dashed contours indicate negative values. $\times$ indicates stiffness and strength corresponding to the original tie brace.

strongback column moment demands than the frames with HDs.

5.3.2 Linear Inerter

Figs. 5.10 to 5.16 show the mean response from the 44 ground motions for varying viscous damping, c , and inertance, b . A total of 441 combinations of c and b were evaluated (21 each). As with the HDs, the figures show the mean response as well as the relative difference compared to the one-segment and two-segment SBFs calculated using Eq. (5.4).

Fig. 5.10 shows the maximum story drift ratio for different RIVD parameters. Replacing the brace with an RIVD provides no reduction in maximum story drifts over the entire range of parameters studied. For very high damping values, $c > 1000$ kip s/in, the response becomes very similar to that of the single segment frame, as the damping provided by the RIVD locks up the joint, preventing it from behaving as two segments. With low damping, $c < 10$ kip s/in, and moderate inertance, $b = 20$ kip s²/in, adding an RIVD can reduce drift performance compared to the two-segment spine. The same pattern is observed in the drift concentration shown in Fig. 5.11.

Fig. 5.12 shows the energy dissipated by the RIVD. A peak is seen around $b = 15$ kip s²/in and $c = 200$ kip s/in.

Fig. 5.13 shows peak story accelerations for different RIVD parameters. Compared to the single-segment SBF, replacing the tie brace with an RIVD always reduces peak accelerations. Reductions in peak accelerations compared to both the single-segment and two-segment frames are seen around the same parameter values that resulted in peak energy dissipation.

Fig. 5.14 and Fig. 5.15 show the peak brace demand and peak column moment demand respectively. Compared to the single-segment SBF, the SBF with RIVDs always has higher moment demands. These demands trend downward with increased damping. As with the drift response, adding an RIVD with a particular combination of low damping and medium inertance can result in column moment demands greater than the two-segment spine alone.

Fig. 5.16 shows the peak base shear for the SBFs equipped with RIVDs. The peak base shear closely follows the peak strongback brace demand, increasing when damping and inertance are both increased, but also having a peak at low damping and medium inertance.

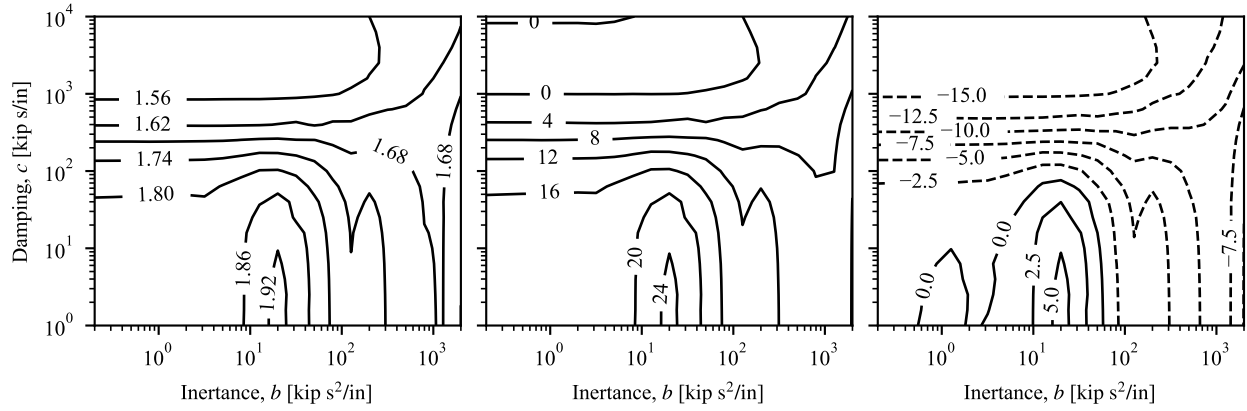


Figure 5.10: Mean value of the maximum story drift ratio (%), for different RIVD parameters. Left-to-right: mean value, relative to one-segment SBF, relative to two-segment SBF. Dashed contours indicate negative values.

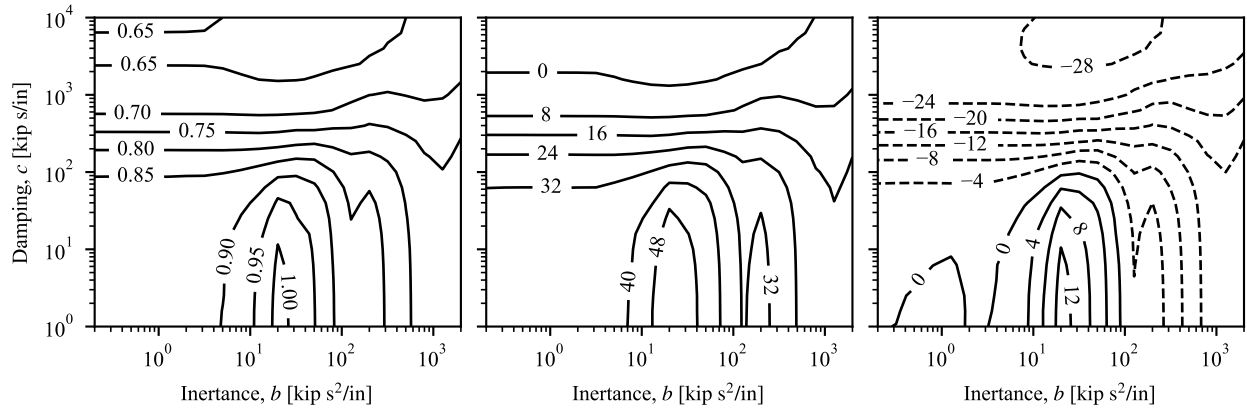


Figure 5.11: Mean value of the maximum marginal story drift ratio (%), for different RIVD parameters. Left-to-right: mean value, relative to one-segment SBF, relative to two-segment SBF. Dashed contours indicate negative values.

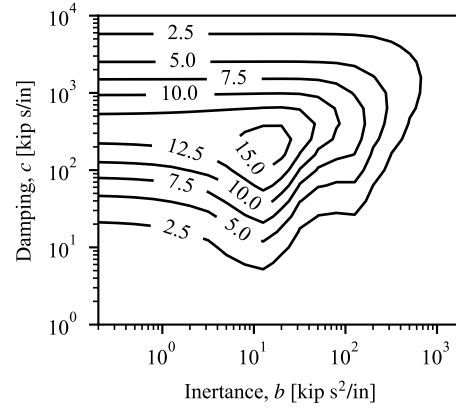


Figure 5.12: Mean value of the energy dissipated by the damper (% of BRB dissipation), for different RIVD parameters.

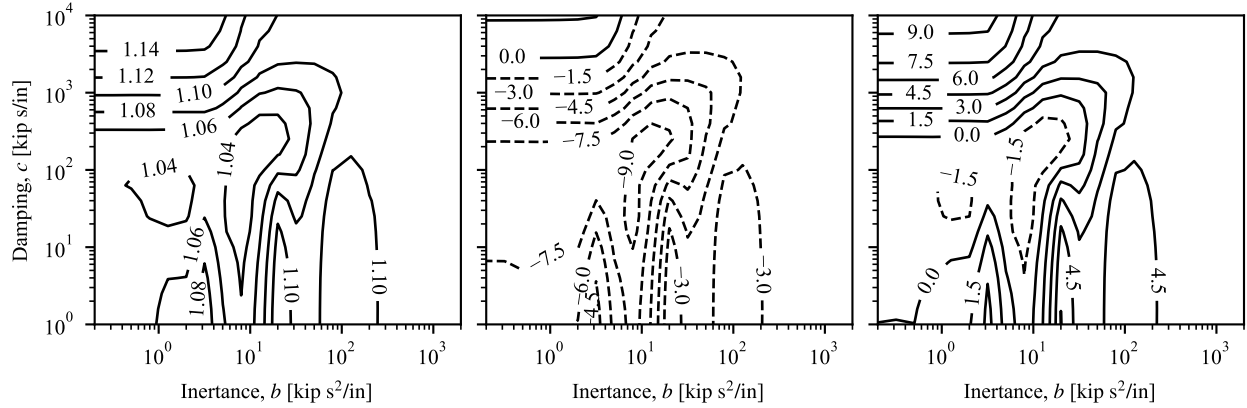


Figure 5.13: Mean value of the peak story acceleration (g), for different RIVD parameters. Left-to-right: mean value, relative to one-segment SBF, relative to two-segment SBF. Dashed contours indicate negative values.

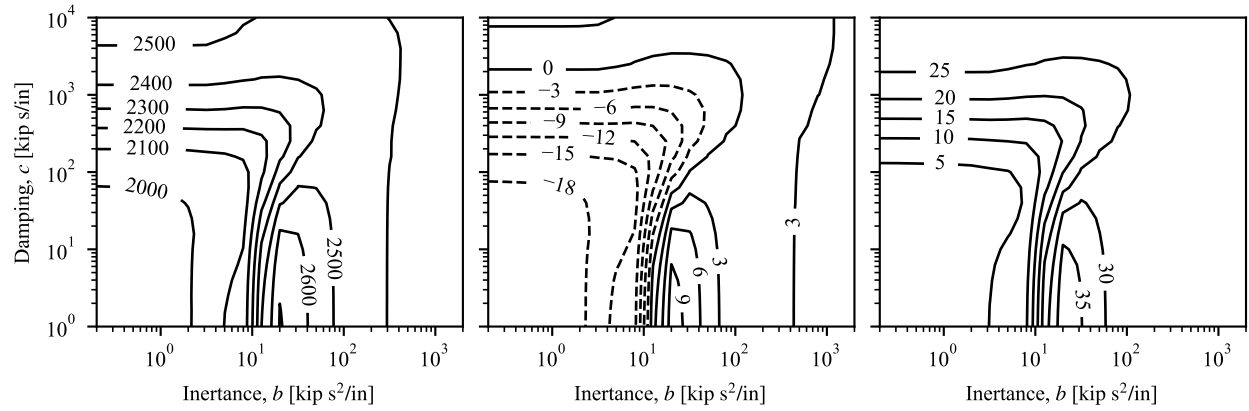


Figure 5.14: Mean value of the peak axial demand (kip) in the strongback diagonal braces, for different RIVD parameters. Left-to-right: mean value, relative to one-segment SBF, relative to two-segment SBF. Dashed contours indicate negative values.

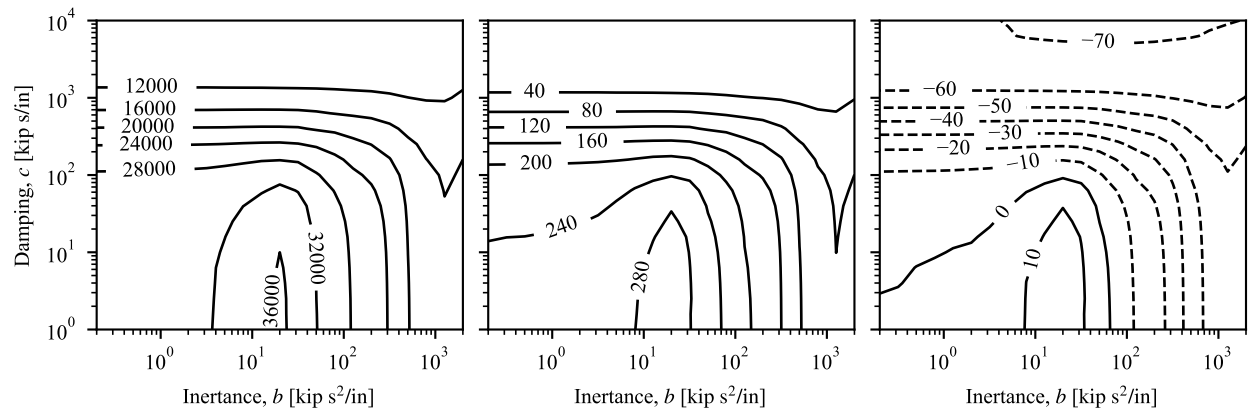


Figure 5.15: Mean value of the peak moment demand (kip-in) in the strongback column, for different RIVD parameters. Left-to-right: mean value, relative to one-segment SBF, relative to two-segment SBF. Dashed contours indicate negative values.

In contrast to the hysteretic dampers, optimizing the RIVD parameters for energy dissipation could provide the most benefit, as this location provides reduced peak accelerations compared to both the single-segment and two-segment frames, while providing reduced maximum drifts and strongback column moment demands compared to the two-segment frame.

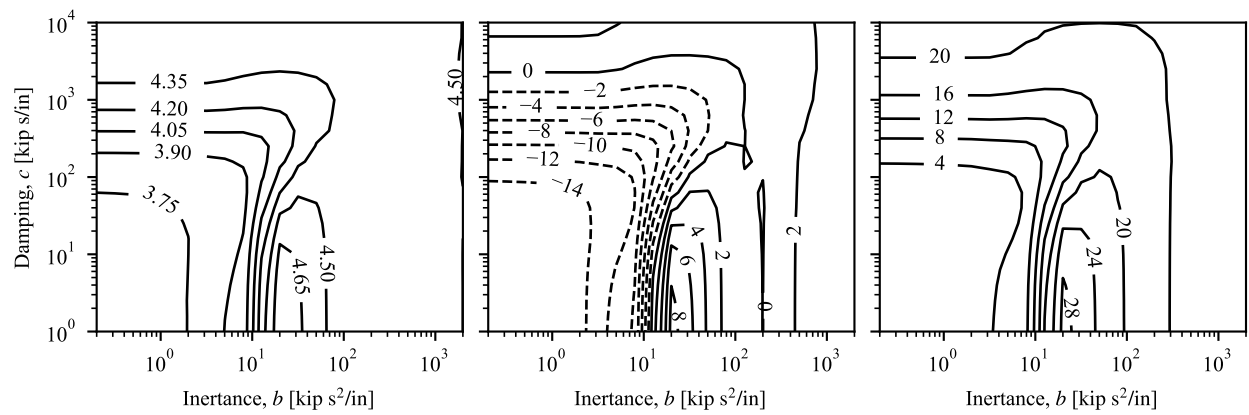


Figure 5.16: Mean value of the peak base shear (ratio of ELF base shear V), for different RIVD parameters. Left-to-right: mean value, relative to one-segment SBF, relative to two-segment SBF. Dashed contours indicate negative values.

CHAPTER 6

Conclusion

The strongback braced frame (SBF), which combines the energy-dissipating capability of buckling-restrained braces (BRBs) with a stiff and strong spine to distribute demands, is a relatively new seismic force-resisting system and one that has several potential advantages. They reduce the possibility of soft story mechanisms forming. SBFs can be part of resilient design as they are intended to distribute and reduce damage, leading to less costly repairs and less red-tagging. They can allow for the use of fewer BRBs. They increase redundancy within a bay. They can achieve these benefits while maintaining an essentially traditional form that is familiar to the steel construction industry.

However, design approaches for SBFs are lacking. While they can be designed using nonlinear response history analyses, this method is time and resource intensive and requires expertise. A more efficient and user-friendly design approach is necessary before SBFs can be practically realized at scale. Furthermore, important behavioral knowledge is missing, particularly regarding the effect of various strongback parameters and the location of optimal points in the design space.

In Chapter 1, two options for practical design methods for SBFs, simplified modal pushover analysis (SMPA) and generalized modified modal superposition (GMMS), were compared to use of current code design provisions using response spectrum analysis and traditional first-mode capacity design (RSAC). To evaluate the required strength and stiffness of the strongback, the designed frames were analyzed using nonlinear response history analysis while varying the strength and stiffness of the strongback members. The results show:

- Both SMPA and GMMS are generally well-calibrated, and provide value over the simpler RSAC procedure.
- Frames designed according to SMPA and GMMS have acceptable probability of collapse, and the strongback behaves as expected, with minimal drift concentration and yielding.

- GMMS is more conservative than SMPA, resulting in a heavier strongback but also improved performance.
- The response of SBFs is more sensitive to a lack of strength than a lack of stiffness in the strongback itself. Reductions in strongback member strength lead to rapid loss of collapse resistance, and large increases in drift concentration and strongback inelasticity.
- Increased strongback member stiffness reduces story drift concentration in SBFs, but is also associated with increased strongback inelasticity.
- The BRBs in an SBF provide the vast majority of the energy dissipation. The beams in SBFs, while characterized as secondary energy dissipators, do not provide large amounts of energy dissipation.

In Chapter 2, a method for designing BRBs for SBFs using overturning moments was introduced. The method provides a simplified procedure for selecting BRB sizes for SBFs, including SBFs with non-standard configurations of BRBs. The method is tunable, using a scale factor on the demand. The method was compared to using ELF to size the BRBs, and using 80% of the base shear to size the BRBs. A range of scale factors were investigated, first in a parametric study evaluating buildings of multiple heights using a simplified model with a rigid strongback, and then a focused study of four-story buildings using a more detailed model with strongback members designed using SMPA. The results from the simplified model show:

- The overturning method provides consistent collapse performance for SBFs with missing BRBs.
- Using ELF for BRB sizing provides acceptable collapse performance for SBFs with missing BRBs.
- BRBs designed using the overturning method with a scale factor of 1.0 provide acceptable collapse performance.
- BRBs designed using the 80% base shear rule were approximately the same as those designed using a scale factor of 1.5, and were 50% larger than BRBs that provided sufficient collapse performance.

Analysis using the more detailed model shows:

- Using a scale factor of 1.0, the overturning method provides acceptable collapse performance for standard SBF configurations. Using the 80% base shear rule, the BRBs are 50% larger and the strongback weighs 5% more while providing similar collapse performance.
- For standard configurations with BRBs at every level, SMPA provides a consistent probability of collapse for a broad range of BRB sizes.
- SMPA may not be appropriate for designing SBFs with missing BRBs, as it underestimates strongback demands in these cases and leads to high collapse probabilities. GMMS provides more accurate estimates of the demands for configurations with missing BRBs, though this may be due to it being a more conservative method in general.
- Collapse of the four-story SBFs studied was preceded by buckling of the strongback braces. BRB fracture did not occur until after failure of the strongback.

In Chapter 3, the behavior of a seven-story SBF with multiple segments was investigated, along with the potential use of tuned energy-dissipating members in the gap between the segments. The frames were analyzed using nonlinear dynamic analysis, with the strongback modeled as elastic. The analysis shows that a two-segment SBF with the same members as a single-segment SBF has lower peak base shear, peak story acceleration, and axial demands in the strongback braces, but experiences higher moment demands in the strongback column and higher maximum story drifts and greater story drift concentration. Adding a hysteretic damper to the two-segment spine provided a broad variation of response ranging between that of the single-segment SBF and the two-segment SBF. Adding a linear inerter to the two-segment spine reduced peak accelerations and strongback column demands compared to both the single-segment and two-segment SBFs, and reduced maximum story drifts and drift concentration compared to the two-segment SBF.

Overall, this dissertation contributes to knowledge on the behavior and design of strongback systems through the introduction of a general approach for sizing the BRBs in an SBF, critically assessing proposed strongback design methods, identifying strengths, weaknesses, and areas for improvement, exploring the broad design space for SBFs. The data and analysis tools used in this work are publicly archived and available online.

The research has also identified and highlighted areas in need of further investigation. These areas include:

- Further work is needed to clarify the role of the beams in SBFs and to develop criteria for sizing beams in SBFs, including the appropriate design ductility level.
- Further investigation of the difference in predicted demand between SMPA and GMMS for protected elements, for both standard configurations with BRBs at every level and non-standard configurations with missing BRBs.
 - GMMS provides more conservative designs than SMPA. This could be due to the 1.3 factor on the GMMS demands.
 - For configurations with missing BRBs, SMPA generally fails to provide an acceptable probability of collapse. GMMS, in contrast, does provide acceptable collapse probability, but it is uncertain as to whether this is due to GMMS being a more conservative design method in general or if GMMS provides a better model of the strongback behavior in these configurations.
 - As the strongback gets closer to an idealized rigid spine, less total BRB area is needed in order to provide sufficient collapse performance. For both SMPA and GMMS, however, the predicted demands, and thus designed size of the strongback, are positively correlated with BRB size.
- A full FEMA P695 study on SBFs to identify appropriate seismic performance factors is needed. The current literature largely assumes that the performance factors for BRBFs transfer to SBFs, but even this is not universal, with studies using a range of values for R .
- The nonlinear analyses in this dissertation have identified that the first story strongback brace is critical for the stability of the system, but the proposed design methods for SBFs do not include the ASCE 7 redundancy factor $\rho = 1.3$ in the design of the strongback members, even for buildings with only one SBF bay in each line of framing.
- Evaluating the feasibility of SBFs for taller buildings. SBFs and spine systems more broadly have primarily been investigated for low- to mid-rise buildings. Increasing the height further begins pushing up against the limits of available hot-rolled sections for the strongback members, requiring modifications to the spine such as using multiple pivoting segments.

- Three-dimensional behavior and the effects of torsion and out-of-plane demands have largely not been investigated for SBFs.
- SBFs as described have asymmetric response, even in 2-D analysis. In particular, pushover analyses demonstrate asymmetric response due to the asymmetric tension and compression behavior of the BRBs. FEMA P695 pushover analysis, used as part of the collapse evaluation, does not provide guidance for evaluating frames with asymmetric response.
- A first-mode pushover analysis, as prescribed by FEMA P695, may not be adequate to characterize the ductility of the system, as SBFs have complex interactions with higher mode demands.
- Adding self-centering to SBFs. Self-centering has been identified as an important part of spine systems for limiting residual drift. SBFs are capable of reducing maximum drifts by averaging demands, but large residual drift can remain due to yield deformation of the BRBs.
- The resiliency benefits of SBFs should be evaluated and quantified. Limited evaluation has been done of the actual cost incurred by the distributed yielding of the components of SBFs. It is expected that less damage in more locations is more advantageous than allowing a large concentration of damage in a single story, but this has not been rigorously evaluated for a large range of configurations.

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