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Assessment of design parameters influencing seismic collapse performance of buckling-restrained braced frames

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ABSTRACT

Buckling Restrained Braced Frames (BRBFs) are widely used as seismic force-resisting systems due to their significant ductility and energy dissipation capacity. However, owing to their modest overstrength and relatively low post-yield stiffness, BRBFs subjected to seismic loading may be susceptible to concentrations of story drift and global instability triggered by *P*-Δ effects. Due to the use of simplistic methods that are based on elastic stability, current code-based design provisions do not address seismic stability rigorously and do not consider the unique inelastic characteristics of different systems. Supplemental strategies may be used to prevent undesirable seismic response of BRBFs, such as story drift concentration and large residual drift. This study employed the FEMA P695 framework to evaluate the response of BRBFs designed according to current codes in the United States and to study the effect on seismic stability of three additional parameters: BRBF column orientation, gravity column continuity, and dual systems. Results from nonlinear static and dynamic analyses provide insight into seismic behavior, and collapse performance evaluation quantifies the relative performance of various BRBF designs and supplemental strategies for enhancing seismic stability.

1. Introduction

Over the past two decades, buckling-restrained braced frames (BRBFs) have grown in popularity in the United States as a primary seismic force-resisting system (SFRS) and are now used extensively in new construction and retrofit applications due to their economy, design simplicity and stable symmetric cyclic response that provides significant energy dissipation capacity and ductility (e.g., [1–7]). Favorable seismic performance of BRBFs, which are concentrically-braced frames (CBFs) with buckling-restrained braces (BRBs), has been demonstrated through numerous numerical and experimental studies (e.g., [8–11]). As a SFRS, a BRBF must be capable of maintaining global stability when subjected to *P*-Δ effects that arise due to the gravity loads acting on the laterally deflected structure during seismic response. However, reliably ensuring seismic stability in buildings is not straightforward due to the complexity and uncertainty associated with input earthquake characteristics and inelastic dynamic structural response. Although research has expanded knowledge about the topic [12–15], it has not led to direct methods for use in seismic building codes. In the absence of a direct rigorous approach, current design provisions address *P*-Δ effects primarily from the perspective of elastic stability. Member forces from seismic loading are amplified by stability coefficients, which are based on static equilibrium of a simplified single degree-of-freedom model that is extrapolated on a per-story basis. Due to this simplification, the process does not directly address the complex response of a building in the inelastic range under *P*-Δ effects. Furthermore, it does not consider the unique inelastic characteristics of different SFRSs.

Despite the favorable seismic performance that BRBFs have exhibited in previous studies, BRBFs designed according to provisions in the U.S. may be vulnerable to dynamic instability. In multi-story buildings, dynamic instability is closely relate to story drift concentration, which can lead to a story mechanism that precipitates collapse. For BRBFs, this tendency is at least partially related to the modest overstrength and relatively low post-yield stiffness of the system, which can also lead to problematic residual drifts [8–10]. Moreover, the collapse capacity of BRBFs can be comparable to or even lower than that of conventional CBFs [11].

Fundamentally, to improve the seismic stability of BRBFs by reducing story drift concentration and residual drifts, secondary stiffness must be provided to the system so that positive global stiffness is maintained up to high levels of drift. Column flexural contributions [16,17] and dual systems [18,19] are potential strategies for providing this secondary stiffness. The research described in this paper

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investigates the impact of various aspects of current code provisions on the improvement of seismic stability of BRBFs. Beyond code-based stability provisions, three additional parameters were also evaluated: BRBF column orientation, gravity column continuity, and dual systems (DS) combining BRBFs and special moment-resisting frames (SMRFs). Moreover, two DS design approaches were studied. Nonlinear static (pushover) and nonlinear dynamic (response history) analyses were used for these evaluations. Finally, collapse performance was quantified with a procedure based on the FEMA P695 Methodology [20]. Although the focus here is on BRBFs, the concerns about rigorously addressing inelastic seismic response and seismic stability are largely system-independent and should be broadly revisited in the future in ASCE 7.

2. Building designs and models

2.1. Prototype buildings

The prototype buildings used for this investigation were based on a 9-story office building model developed for a previous study [21], with a basement level (3658 mm), a tall first story (5486 mm), and uniform story heights (3962 mm) above the first story. Using this base building, prototype systems were defined by varying the number of stories above ground (4, 9 or 15) and system configuration. To evaluate the impact of current code provisions for seismic stability, noncompliant BRBF designs that ignored the stability requirements of AISC 360–10 [22] were established as an initial reference (prototype A). Code-compliant designs including stability requirements via the B_2 amplifier were established as the primary baselines (prototype 1). The B_2 amplifier is an approximate method for considering the destabilizing effects of gravity (*P*-Δ effects) and determining internal member forces that are consistent with equilibrium formulated on the global elastic deformed geometry of the structure. In AISC 360–10 [22], the B_2 amplifier is defined as $B_2 = 1/(1\text{-}P_{story}/P_{e,story})$ where P_{story} is the total vertical load on the story and $P_{e,story}$ is the elastic critical buckling strength for the story. The B_2 amplifier does not account for inelastic deformed geometry.

Prototype 1 had BRBF columns oriented for weak-axis bending in the plane of the BRBF (weak-axis orientation), and gravity columns did not contribute to lateral resistance. Additional variations away from this baseline were then examined for each building height: (a) prototype 2, with strong-axis orientation for BRBF columns and no gravity column contribution; (b) prototype 3, with weak-axis orientation for BRBF columns and continuous gravity columns; and (c) BRBF-SMRF dual systems with weak-axis orientation for BRBF columns and no gravity column contribution. Two DS alternatives where studied: a design in accordance with the ASCE 7–10 requirement, used in prototypes DS4, DS9 and DS15, and a proposed design based on the procedure described by Magnusson [23], used in prototypes DS4-P, DS9-P and DS15-P. Fig. 1 presents the building plan view, and an elevation for a 4-story BRBF-SMRF dual system. As shown in the figure, for the BRBF-SMRF dual systems, the BRBF and SMRF are placed in the second and fourth bays; for the isolated BRBF cases, however, the BRBF is located in the middle bay.

2.2. Seismic Design

The prototype buildings are located in Seattle, WA, United States, a zone that is exposed to crustal and sub-crustal earthquakes and seismic ground motions originating from the Cascadia subduction zone. It is assumed that the structure is constructed on firm soil (Site Class C). The design spectral response acceleration parameters are $S_{DS} = 0.91$ g and $S_{D1} = 0.458$ g. Per ASCE 7–10 [24], the response modification coefficient for BRBF and BRBF-SMRF DS is $R = 8$. The importance factor is $I_e = 1.0$ and the building is in Seismic Design Category D. The redundancy factor is $\rho = 1.3$.

All prototypes were designed using Modal Response Spectrum Analysis (MRSA) according to ASCE 7–10 [24] and AISC 341–10 [25], except for BRBF9-ELF, which was designed using the Equivalent Lateral Force (ELF) procedure and was used to evaluate the influence of design procedure (ELF vs. MRSA) on BRBF seismic performance. Key design parameters are compared in Tables 1–3. In these tables, base shear per the ELF procedure is *V*, and base shear per the MRSA procedure is 0.85 *V*, as permitted by ASCE 7–10. Further details are provided by Zaruma [26].

For dual systems DS4, DS9 and DS15, the BRBF was proportioned to resist the full design base shear and the SMRF was sized for 25% of the design base shear, in accordance with the requirement from ASCE 7–10. The full design base shear was used for the BRBF considering that it is much stiffer than the SMRF and, therefore, owing to deformation compatibility, the SMRF carries negligible base shear for elastic response. The BRBF designs considered *P*-Δ effects so the BRBF portions of these dual systems are identical to BRBF4-1, BRBF9-1 and BRBF15-1. Previous research on dual systems and initial dual system analyses for this research motivated the development of an alternate BRBF-SMRF DS design. A more detailed design procedure considering the interaction between the BRBF and the SMRF components was applied for DS4-P, DS9-P and DS15-P. This Proposed DS design consisted of the following three steps: (1) design the BRBF for the full design base shear using MRSA, without considering elastic stability requirements (i.e., $B_2 = 1$); (2) design the SMRF for 50% of the design base shear; and (3) combine systems in an elastic model and reduce the BRBF member sizes based on the relative stiffness of the BRBF to SMRF using MRSA so that exactly the design base shear is carried elastically. This is an iterative process in which reduction of the BRBF member sizes is performed until convergence is achieved for the relative stiffness between the BRBF and the SMRF. For the BRBF components of DS-4P, DS-9P and DS-15, the final proportions of base shear used in design are indicated in Tables 1–3, respectively.

2.3. Numerical model

For all prototypes, numerical models were developed using the OpenSees software platform [27]. The symmetry of the building floorplan allowed modeling only half of the building. All columns were included in the model and to account for *P*-Δ effects, rigid truss elements connected the gravity columns to one another and to the SFRS at each level. Concentrated plasticity models were used for beams and columns. The modified Ibarra-Medina-Krawinkler (IMK) deterioration material model [28,29] was used to define the rotational spring properties. For BRBFs, moment-resisting beam-column connections were modeled at the floor levels and pinned beam-column connections at the roof level. BRBs were modeled as corotational truss elements between the gusset plates using Steel4 and the Fatigue material. Calibrated parameters for Steel4 [30] were used and the ultimate strength was matched with results from large-scale experimental data [9]. The Krawinkler model [31] was adopted for panel zone behavior in SMRFs. Regarding non-simulated collapse modes, a maximum ductility limit for BRBs, $\mu_{max} = 30$, was determined based on results from experimental testing and included in the post-processing. Further details about the models are provided by Zaruma [26].

3. Nonlinear static analyses

Results from monotonic nonlinear static analysis of each frame are summarized through the pushover curves shown in Figs. 2–4 along with the response quantities presented in Tables 4–6. V_{max} is the maximum base shear capacity, δ_u is the ultimate displacement (per FEMA P695, the displacement where $0.8V_{max}$ is reached in the post-peak region of response), and Ω is the system overstrength. Story drift profiles at maximum base shear capacity are presented in Figs. 5–7, where the allowable story drift from ASCE 7–10 (2%) is included for reference.

The pushover curves for all prototypes exhibit negative stiffness due

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