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Assessing the Effect of Design Variations on Seismic Stability of Steel Special Concentrically Braced Frames

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ABSTRACT

Building seismic stability is provided when a structural system is capable of carrying gravity loads while developing large inelastic deformations and associated lateral displacements, but this complex nonlinear behavior is challenging to distill into a straightforward design approach. Code-based consideration of seismic P-Delta effects is generally based on simplistic elastic models, and specific to steel buildings and the design framework in the United States, the current fundamental approach for stability design was developed and calibrated for non-seismic scenarios where the structure has modest overstrength, and the ultimate strength (stability point) of the structure occurs prior to significant inelastic deformation. The results described in this paper are part of a comprehensive study that is seeking to identify the most critical lateral system parameters that affect seismic stability and to develop a rigorous yet simple methodology whereby these parameters can be considered in the design. This paper focuses on a set of steel special concentrically braced frames that is evaluated using nonlinear static analysis to assess the relative influence of several parameters – bracing configuration and analysis type used in the design process – on inelastic seismic stability behavior.

Introduction

Although modern performance-based seismic design employs advanced analysis to rigorously consider stability effects, seismic stability considerations in pragmatic code-based procedures are still relatively simple. In the United States, most buildings are designed for seismic loads using reduced forces prescribed in ASCE/SEI 7-16 [1] and elastic analysis considering P-Delta effects. Focusing on steel buildings and ANSI/AISC 360-16 [2], stability design is conducted with the Direct Analysis Method. In a ductile steel seismic lateral force-resisting system (LFRS), such as a special concentrically braced frame (SCBF), the design-level forces and resulting nominally-elastic deformations are not consistent with the ultimate strength state of the system. Although ANSI/AISC 341-16 [3] contains rigorous requirements related to capacity-based proportioning and ductile detailing, global seismic stability is not directly considered. This paper summarizes a portion of a study that is evaluating seismic stability design of steel frames. SCBFs were designed and evaluated with nonlinear static (pushover) analysis to observe differences in behavior and to evaluate implications of design variations on response.

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Prototype Building Designs

The prototype buildings and seismic design parameters for this study were based on the 8-story and 16-story office buildings from the National Institute of Standards and Technology (NIST) Technical Note 1863-2 [4]. The LFRS studied in this paper are 2-bay SCBFs designed by ASCE/SEI 7-16 using either the equivalent lateral force (ELF) analysis procedure or the modal response spectrum analysis (RSA) procedure. According to the Direct Analysis Method of Design in ANSI/AISC 360-16, geometric nonlinearity is considered via second-order elastic analysis, and softening due to initial yielding is considered via a stiffness reduction factor. The design strengths of the structural elements were determined in accordance with ANSI/AISC 360-16 and the capacity design procedures in ANSI/AISC 341-16. The gusset plate connections were designed by the Uniform Force Method to eliminate bending moments at the connection interfaces.

Variations in general analysis type (first-order or second-order), seismic effect analysis procedure (ELF or RSA), and bracing configuration define the extents of the design matrix, as shown in Table 1. The two bracing configurations considered are inverted-V (IV) and split-X (2X). For both 8-story and 16-story frames, first-order analysis and second-order analysis resulted in the same member selections, indicating that elastic second-order effects are negligible for these stiff braced frame systems. Similarly, for the 8-story frames, ELF and RSA resulted in the same member selections owing to the predominantly first-mode elastic response and member width-to-thickness design requirements. For the 16-story frames, some small changes in member sizes occurred between ELF and RSA, with the RSA designs being slightly lighter (Table 2).

Table 1. Design matrix of special concentrically braced frames

Analysis Type	Bracing Configuration	8-Story		16-Story	
		ELF	RSA	ELF	RSA
Second-Order	Split-X (2X)	08-2X	-	16-ELF-2X	16-RSA-2X
First-Order	Split-X (2X)	-	-	-	-
Second-Order	Inverted-V (IV)	08-IV	-	16-ELF-IV	16-RSA-IV
First-Order	Inverted-V (IV)	-	-	-	-

Numerical Building Models

Two-dimensional numerical building models were developed using the OpenSees [5] platform. A concentrated plasticity approach was used for beam and column models, with zero-length rotational springs and the modified Ibarra-Medina-Krawinkler hysteretic model used in plastic hinge regions [6,7]. A distributed plasticity approach was used for brace elements, with displacement-based beam elements and fiber sections following the recommendations of Karamanci and Lignos [8]. Gusset plate connections were modeled using nonlinear rotational springs [9]. The leaning column was a single representative column used to simplify the modeling of the entire gravity framing system and capture global P-Delta effects associated with lateral displacements.

Nonlinear Static Analyses

Nonlinear static pushover analyses were used to quantify the effect of various design parameters on the nonlinear behavior of the numerical models. The global inelastic behavior of the numerical SCBF models is represented by pushover curves (normalized base shear, V / V_{design} , vs. roof drift ratio) as shown in Figure 1. All of these curves display essentially linear elastic response followed by a short region of softening up to the peak strength, and then a region of negative stiffness driven by global P-Delta effects. The following three significant points are noted on the pushover curves: a) end of the linear range, b) peak base shear (V_{max}), and c) 20 % strength loss ($0.8V_{\text{max}}$). Response quantities from pushover analyses are summarized in Table 2, where W_{SCBF} is the steel self-weight for a single SCBF; V_{design} is the base shear for strength design; $\delta_{y,\text{eff}}$ is the effective yield displacement calculated per FEMA P695; δ_u is the ultimate displacement defined as the roof displacement at $0.8V_{\text{max}}$; Ω is the overstrength defined as $V_{\text{max}} / V_{\text{design}}$; and μ is the ductility defined as $\delta_u / \delta_{y,\text{eff}}$.

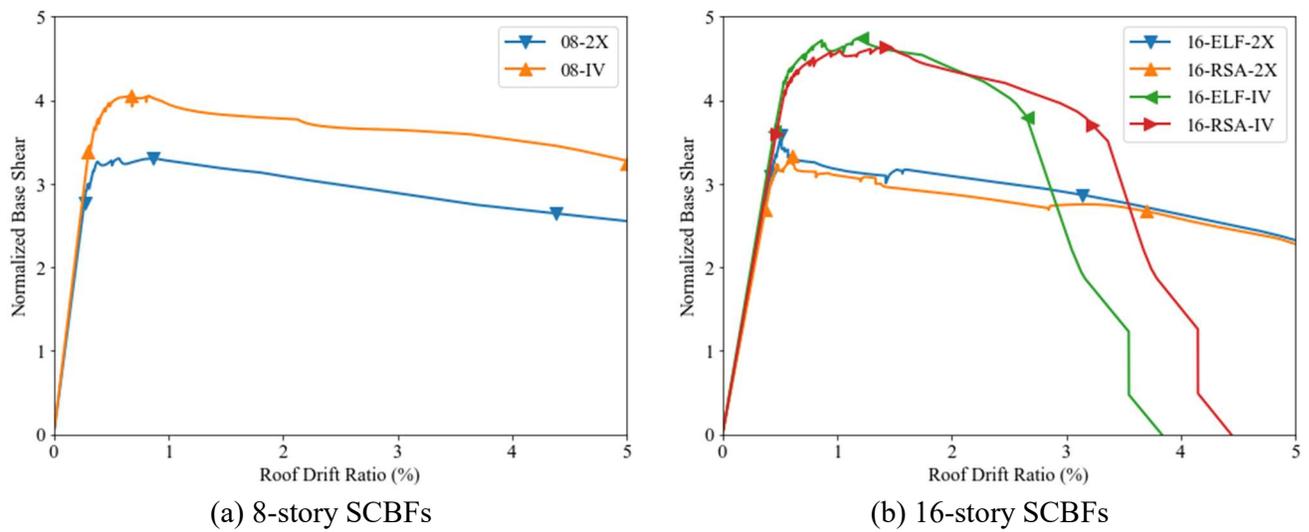


Figure 1. Normalized monotonic static pushover curves

Table 2. Response quantities of SCBFs from pushover analyses

Case Name	W_{SCBF} (kips [kN])	V_{design} (kips [kN])	V_{max} (kips [kN])	$\delta_{v,eff}/h$ (%)	δ_u/h (%)	Ω	μ
08-2X	123 [547]	682 [3030]	2260 [10100]	0.32	4.32	3.32	13.6
08-IV	214 [951]	696 [3100]	2830 [12600]	0.36	5.00	4.07	13.8
16-ELF-2X	328 [1460]	753 [3350]	2700 [12000]	0.40	3.14	3.58	7.9
16-RSA-2X	324 [1440]	753 [3350]	2510 [11200]	0.36	3.70	3.34	10.3
16-ELF-IV	518 [2300]	783 [3480]	3710 [16500]	0.45	2.66	4.74	5.9
16-RSA-IV	514 [2290]	783 [3482]	3630 [16100]	0.47	3.23	4.64	6.9

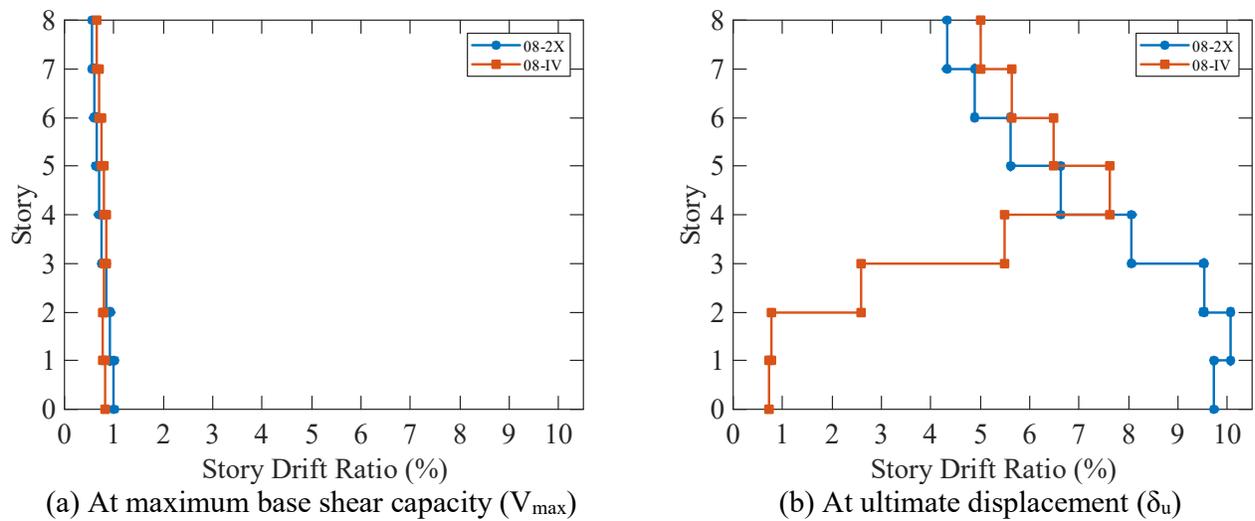


Figure 2. Effect of design variations on story drift profiles of 8-story SCBFs from pushover analyses

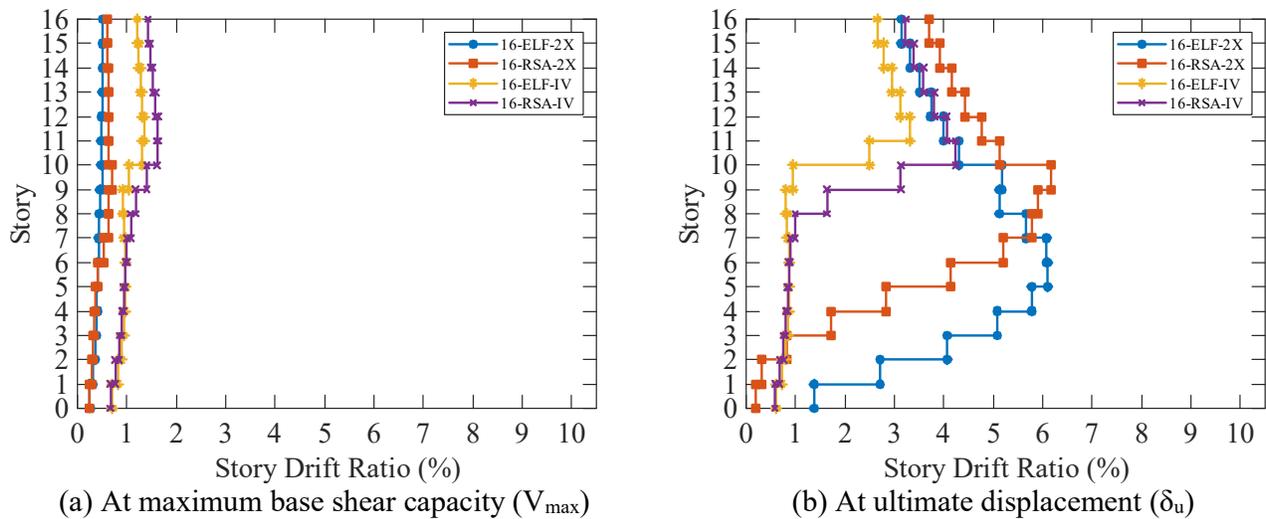


Figure 3. Effect of design variations on story drift profiles of 16-story SCBFs from pushover analyses

Comparing ELF to RSA for the 16-story frames, as shown in Table 2, design base shear (V_{design}) is the same for each brace configuration pair, and the overstrength (Ω) is nearly identical. Little difference is observed in the global response between ELF and RSA (Figure 1), although there are differences in the story-wise distribution of inelastic response (Figure 3). The RSA designs exhibit more than 20 % greater ductility (μ) than the ELF designs, meaning that their strengths degrade to $0.8V_{\text{max}}$ at larger drifts. Comparing split-X to inverted-V bracing, the inverted-V designs are 50 % heavier than the split-X design due largely to the unbalanced force capacity design requirements for beams in the inverted-V configuration. This also translates into noticeable differences in V_{max} and Ω , where the inverted-V configuration is stronger. Figure 2 and Figure 3 show the effects of design variations on the story drift profiles of SCBFs at V_{max} and δ_u . All prototypes have relatively uniform distribution of drift at maximum base shear capacity. Comparison of designs with different bracing configurations at the ultimate displacement point indicates that the split-X configuration leads to a greater concentration of inelastic demands in the lower stories. In contrast, the inverted-V configurations have larger drifts in the upper stories, while the inelastic response is more limited in the lower stories.

Conclusions

Global destabilizing effects of gravity (P-Delta effects) are a fundamental consideration for seismic stability design, yet the braced frame examples presented in this paper demonstrate that P-Delta effects do not influence the braced frame proportions using current U.S. design provisions. Due to inherent braced frame stiffness, first-order and second-order elastic analyses used in the design process led to the same member size selections. Modest differences in inelastic response are observed when comparing braced frame designs proportioned with equivalent lateral force and response spectrum analysis procedures. The most significant differences in inelastic braced frame response were observed when comparing split-X and inverted-V configurations, where the inverted-V configurations are approximately 50 % heavier and 30 % stronger.

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