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SEISMIC PERFORMANCE EVALUATION AND RETROFIT OF NONDUCTILE CONCENTRICALLY BRACED FRAMES

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Abstract

Concentrically braced frames (CBFs) are common seismic-force-resisting systems in steel construction. CBFs built prior to about 1990 predate the codification of capacity-based and other ductile design provisions intended to provide significant lateral deformation capacity and mitigate damage outside the braces and gusset plates. Many of these older and potentially nonductile CBFs (NCBFs) remain in use today in regions with high seismic risk. NCBFs employ a wide range of brace types, bracing configurations, and gusset-plate configurations, and they are expected to have strength and/or deformation capacities at the component and system levels which are lower than special CBFs (SCBFs). The impacts of these deficiencies on the seismic vulnerability of NCBFs is uncertain, and seismic retrofit may be required to ensure safety and functionality of these structures in large earthquakes.

The potential vulnerability of NCBFs and lack of existing engineering guidance for their retrofit motivated an extensive research program funded by the National Science Foundation consisting of integrated experimental and computational investigations. Twenty-two (22) large-scale tests of existing or retrofitted NCBFs were conducted at the University of Washington and National Center for Research on Earthquake Engineering (Taipei, Taiwan) to evaluate the effects of brace, connection, and chevron-beam vulnerabilities common to the infrastructure. These experimental data were used to develop advanced nonlinear modeling approaches using line and spring elements capable of simulating brace fracture, connection fracture, secondary connection yielding mechanisms, beam yielding, and column buckling. The modeling approaches were then implemented in *OpenSees* to analyze the seismic performance of three- and nine-story NCBF, retrofitted NCBF, and SCBF buildings located in Seattle. The buildings were subjected to five suites of ground motions selected and scaled to approximate discrete seismic hazard levels (i.e., a multiple-stripe analysis). The combined experimental and computational findings inform a seismic-retrofit priority and demonstrate the viability of relatively non-invasive retrofit schemes which balance yielding mechanisms and suppress severe failure modes. Finally, practical methods for modeling and evaluating NCBFs within the ASCE 41 framework are proposed for implementation in AISC 342.

Keywords: Steel; braced frames; retrofit; nonlinear analysis; evaluation.

1. Introduction

Steel concentrically braced frames (CBFs) employ diagonal braces along the building height to resist forces induced by wind and seismic events. These braces provide considerable strength and stiffness and can be arranged to accommodate a variety of architectural considerations. The design of CBFs in regions with high seismic risk has evolved substantially over the past several decades. In US construction, CBFs were designed with little consideration of ductility until the late 1980s. Capacity-based design provisions were eventually introduced in the *1988 Uniform Building Code* [1] to ensure beams, columns, and connections could sustain yielding of the braces in tension and buckling in compression. Provisions for special CBFs (SCBFs) were introduced about a decade later to ensure ductility of the braces and system [2]; these provisions remain similar in new construction today [3]. The key differences between SCBFs and their vintage counterparts are as follows:

- Capacity-based design is performed using (1) a load case in which all braces attain their expected capacities in tension and compression and (2) a load case in which all braces attain their expected capacities in tension but have degraded post-buckling compressive capacity;
- Lateral resistance must be adequately proportioned between compressive and tensile braces;
- Brace configurations are limited to avoid unacceptable inelastic deformations (i.e., K-bracing);
- Cross-sectional width-to-thickness ratios (or local slenderness ratios) are limited for braces, beams, and columns to delay and mitigate the severity of local deformations;
- Brace connections must be able to accommodate end rotation of the brace resulting from buckling or develop its fully moment capacity; and
- Welds connecting components in the seismic-force-resisting system must meet demand critical requirements, which include minimum Charpy V-Notch toughness.

As a consequence of these changes, the seismic behavior and performance of CBFs built prior to 1988 is expected to vary drastically from SCBFs. These older CBFs are thus termed nonductile CBFs (NCBFs). NCBFs are prevalent in existing building infrastructure in regions with high seismic risk in the US (with similar systems existing around the world) are expected to remain in service for years to come. Thus, these NCBFs have been and will be candidates for seismic rehabilitation to ensure their safety and functionality.

Despite their apparent vulnerability, current guidance for seismic retrofit of NCBFs is limited. The American Society of Civil Engineers (ASCE) provide modeling recommendations, performance acceptance criteria, and several retrofit strategies in *ASCE/SEI 41* [4]. However, these provisions do not address the potentially complex behavior of NCBF connections and system deficiencies expected to affect response. For example, all brace connection limit states are treated with equal severity, whereas recent research on SCBFs has shown that limited yielding of gusset plates can actually prove beneficial [5].

A research project funded by the US National Science Foundation entitled “Collaborative Developments for Seismic Rehabilitation of Vulnerable Braced Frames” (Grant No. CMMI-1208002) was established to investigate the seismic vulnerability of NCBFs and evaluate the efficacy of potential retrofit schemes. This research consisted of a series of large-scale subassemblage tests and a complementary computational research program to investigate system-level effects, and this paper focuses on the latter. The experimental observations are highlighted and new approaches for nonlinear modeling are developed. These modeling approaches are implemented in *OpenSees* [6] and used to evaluate the performance of archetype buildings with variations to represent different deficiencies. The results show the consequences of severe deficiencies on collapse probability and significant benefit of relatively economical retrofit schemes.

2. Experimental Highlights

Two large-scale testing programs were completed to understand the effects of various NCBF deficiencies and retrofits. The first set of tests investigated brace and connection deficiencies and retrofits using the test setup shown in Fig. 1a. These were tests of one-story NCBFs in the single-diagonal configuration. Twenty-two specimens (22) specimens were tested using the same beam and column sections but different braces and connection configurations (highlighted components in Fig. 1a). The reader is referred to Sen et al. [7] for detailed descriptions of these experiments. The second set of tests also investigated brace and connection deficiencies but in conjunction with beam deficiencies; that is, the braces were oriented in the chevron configuration and the beam was not strong enough to develop the expected unbalanced brace loads after buckling (second load case described above). This test setup was utilized for four (4) specimens and is depicted in Fig. 1b. These tests are fully described by Sen et al. [8]. Note that the primary study area depicted in Fig. 1b was the first story, as the second story beam was designed as intentionally stronger than typical for an NCBF in order to deliver the actuator force into the frame.

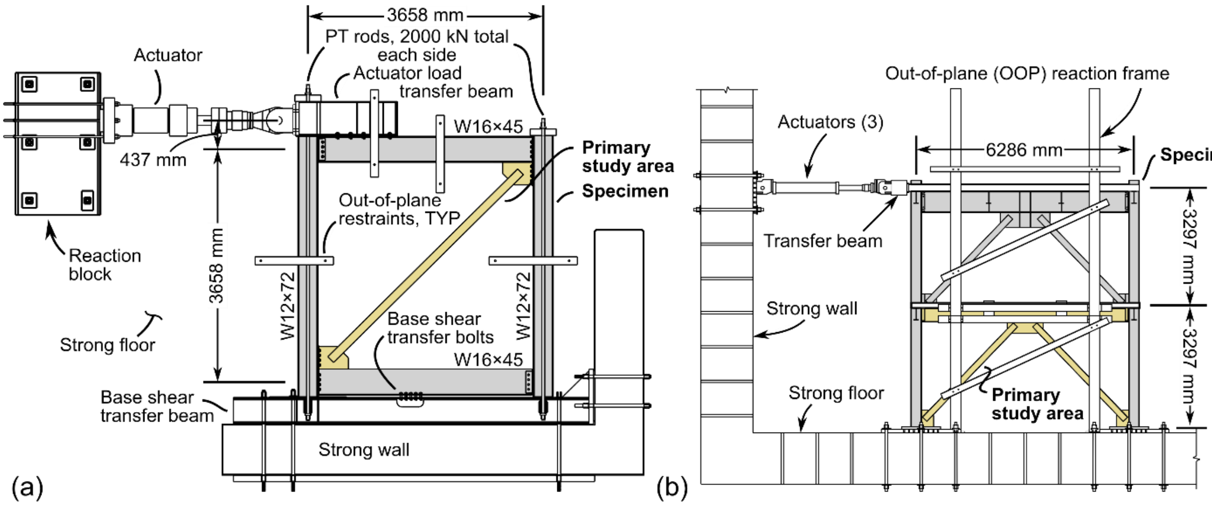

Fig. 1 – Experimental setup (not to scale): (a) one-story specimens, (b) two-story specimens

The tests revealed four main patterns of behavior across the different braces, brace configurations, connection configurations, and retrofit strategies:

- Type A: Premature failure of a connection leading to beam-to-column connection failure and potential loss of vertical-load-resisting capacity.
- Type B: Premature failure of a connection or brace with residual strength, stiffness, and ductility from retained beam-to-column connections.
- Type C: Premature failure of a connection or brace with significant residual strength, stiffness, and ductility due to retained load path to brace via secondary yielding mechanism.
- Type D: Failure of brace after significant cyclic deformation (i.e., SCBF behavior).

These categories are described graphically with representative backbone curves in Fig. 2. Note that the ultimate response of each category is loss of the beam-to-column connection, but this type of failure was only rarely observed in the experimental program. Type A behavior is the most severe because the initial failure mode results in loss of beam-to-column connection resistance. In Type B, C, and D behavior, the initial failure mode results in total or partial loss of the brace load path. Total loss of the brace load path may be onset by brace fracture, brace-to-gusset weld fracture, or gusset-to-beam/column weld or bolt fracture. These failure modes occur prematurely when the brace local slenderness is too high (e.g., greater than the specified maximum local slenderness ratio for highly ductile members in the AISC *Seismic Provisions* [3]) or the connection elements (welds or bolts) are undersized for the expected brace forces. Partial loss of the brace load path was observed for connection configurations where the gusset plate was bolted to the column via a shear plate or double angles and welded directly to the beam. In these cases, gusset-to-beam weld fracture was the initial failure mode, and secondary yielding was possible through bolt-hole elongation (shear plate connections) or angle yielding (double-angle connections) on the retained gusset-plate interface. Importantly, the weak beam tested in the two-story specimens was not directly responsible for the development of a given behavior type; the brace and connection properties were more critical, and Type D behavior was achieved when brace and connection deficiencies were eliminated in retrofit.

Retrofit strategies tested included concrete in-fill of rectangular HSS braces, gusset-to-beam/column weld reinforcement, gusset-to-column bolt reinforcement, and brace replacement using in-plane buckling braces or buckling-restrained braces. Concrete in-fill successfully increased the deformation capacity of a rectangular HSS brace with a local slenderness ratio 2.3 times that permitted by the AISC *Seismic Provisions* [3]. The concrete delayed and mitigated the effects of local deformation in the brace. This retrofit was successful because the concrete fill was terminated before the gusset plate, and hence the strength of the brace (and thereby demand on the connections) was not increased. Weld reinforcement successfully increased the deformation capacity of gusset plates which bend about their weak axis due to out-of-plane brace buckling. These welds were reinforced with filler metal meeting demand critical requirements to develop the full tensile strength of the plate in conjunction with the existing (and non-demand-critical) filler metal.

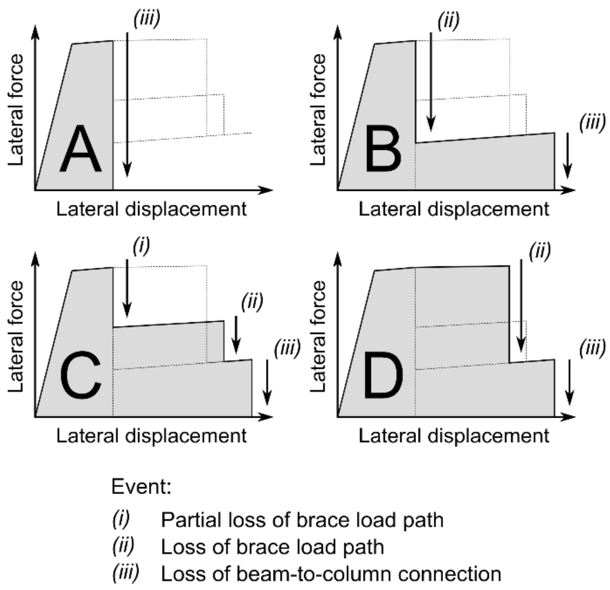

Fig. 2 – Backbone curves representative of typical CBF behavior categories

3. Nonlinear Model Development

Accurate representation of the responses of existing and retrofitted NCBFs is important for understanding their seismic performance. Nonlinear modeling approaches for SCBFs are well established; in particular, significant developments have been made in *OpenSees* [6] to simulate buckling and fracture of the braces using fiber-based elements, flexural yielding of gusset plates using rotational springs, and strength and stiffness contributions of the beams and columns using fiber-based elements [9-12]. However, the observations of the experimental research program show that NCBFs and their retrofits do not exhibit behavior similar to SCBFs (Type D) in many cases. While fracture of brace-to-gusset welds has been successfully modeled in the literature [13-14], further developments are necessary to simulate premature fracture of braces and other connections and estimate the effects of concrete in-fill in HSS braces.

Several models for predicting and simulating brace fracture on a fiber-by-fiber basis in *OpenSees* [6] have been developed in previous research [9, 11-12]. In this research, the brace type is limited to rectangular HSS and the maximum strain range fracture model developed by Hsiao et al. [11] is extended to ensure fracture of braces with high local slenderness and asymmetric loading histories typical of chevron braced frames with yielding beams can be well predicted. In this updated model, an individual fiber in the brace fails (strength and stiffness reduced to nearly zero) when the total strain range in tension and compression exceeds Eq. 1, where

b/t is the local slenderness ratio, L_c/r is the global slenderness ratio, E is the elastic modulus, and F_y is the yield stress, $\delta_{c,max}$ is the maximum compressive deformation of the brace, and $\delta_{t,max}$ is the maximum tensile deformation of the brace. This equation was calibrated using linear regression in logarithmic space based on a data set developed from numerical simulation of 59 experiments of braces or braced frames.

$$MSR = 0.554\left(\frac{b}{t}\right)^{-0.75}\left(\frac{L_c}{r}\right)^{-0.47}\left(\frac{E}{F_y}\right)^{0.21}\left(\frac{\delta_{c,max}}{\delta_{t,max}}\right)^{0.068} \tag{1}$$

To account for the effects of concrete fill in rectangular HSS braces, ten additional tests were simulated. It was found that the maximum strain range at fracture did not significantly vary with the slenderness, material, or deformation history variables used in Eq. 1. Therefore, the geometric mean of the data, 0.0505, is used for such braces.

A model to simulate failure of gusset-to-beam/column welds due to weak-axis bending of the plate was also developed. This simulates Type B behavior described in Fig. 2 (i.e., without a secondary yielding mechanism to achieve Type C behavior). Similar to the brace fracture failure mode, numerical simulation of experiments was conducted and the numerical gusset-plate rotation at fracture in the test was extracted. Eq. 2 is the predicted rotation at fracture based on linear regression in logarithmic space of these data, where L_{clear} is the gusset-plate elliptical clearance [15], t_p is the gusset-plate thickness, and DCR is the weld demand-to-capacity ratio based on the tensile strength of the plate [5]. This failure mode is simulated in *OpenSees* [6] by removing the zero-length element which connects the brace to the frame when the gusset-plate rotation exceeds the value from Eq. 2.

$$\theta_f = 0.11\left(\frac{L_{clear}}{t_p}\right)^{0.33} DCR^{-0.57} \leq 0.257 \text{ rad} \tag{2}$$

4. Seismic Performance Evaluation

Simulation of building systems was performed in *OpenSees* using the newly developed nonlinear modeling approaches to quantify the vulnerability of existing NCBFs and the potential performance enhancement of retrofitted NCBFs. Four building archetypes were developed for a site in Seattle, WA, USA (47.619°N, 122.333°W) with Site Class C soil (V_{s30} of 537 m/s). The archetypes included (1) a 3-story paired single-diagonal CBF, (2) a 3-story chevron CBF, (3) a 9-story paired single-diagonal CBF, and (4) a 9-story chevron CBF. The building elevations are shown in Fig. 3 and were based on drawings of existing NCBFs in the US obtained from practicing engineers [16]. A full description of the building designs and modeling approaches are available in Sen [17].

For each archetype, NCBF buildings were designed using the *1979 Uniform Building Code* [18], with the exception that only brace and gusset-plate interface weld deficiencies were present. Hence, the designs intentionally isolated these failure modes. The NCBF buildings were varied parametrically to investigate the effects of brace local slenderness (hollow brace versus concrete-filled brace) and gusset-to-beam/column weld fracture rotation (θ_f between 0.075 and 0.200 rad). For comparison, SCBF buildings were designed using *ASCE/SEI 7-16* [19].

Each building was subjected to 5 suites of 30 ground-motion record sets selected from the NGA-West2 database [20]. The 5 suites represented hazard levels with intensity return periods of 43, 475, 975, 2,475, and 4,975 years. The ground motions for each hazard level were selected and scaled such that their geometric-mean intensity matched the site's uniform hazard spectrum on average in a period range between $0.5T_1$ and $5T_1$, where T_1 is the period of the building's first mode. This procedure was used to ensure the ground motions were sufficiently intense at longer periods, since the fundamental period of the structure elongates due to the change in stiffness after brace buckling and fracture.

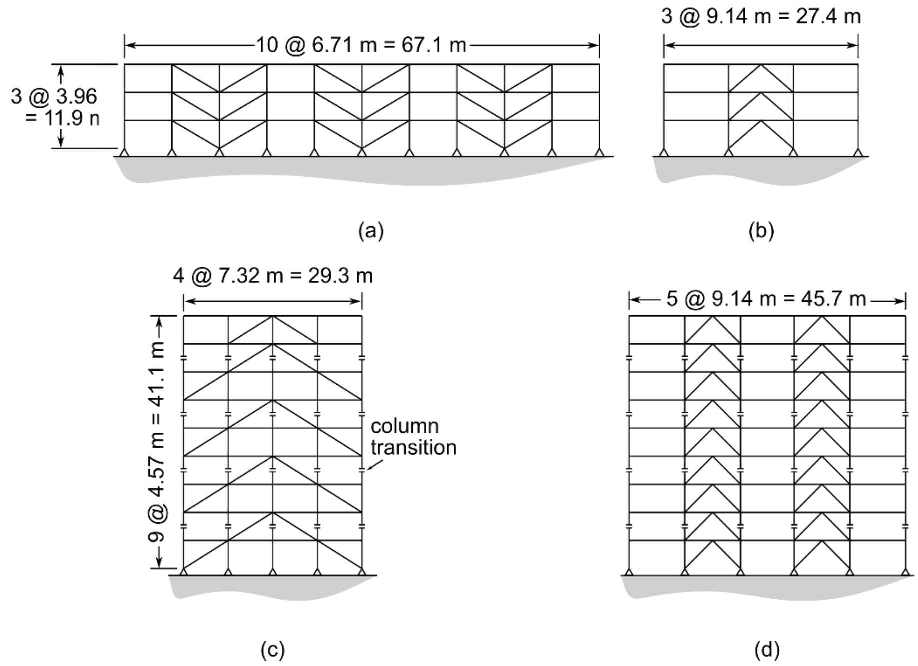

Fig. 3 – Archetype building elevations (not to scale): (a) 3-story paired single diagonal, (b) 3-story chevron, (c) 9-story paired single diagonal, (d) 9-story chevron

Fig. 4 shows piecewise-linear collapse fragility curves derived from the analysis results for the NCBF (hollow brace) and NCBF-R (concrete-filled brace) building archetypes. The potential-collapse performance state was determined to be reached when any of the following conditions were satisfied: (1) any story exceed a drift ratio of 8%, (2) any beam-to-column connection fractured based on the modeling recommendations of Liu and Astaneh-Asl [21], or (3) the analysis did not reach a converged state after the prescribed iteration procedure. The fragility curves of comparable SCBFs (same configuration and height) are superimposed as dotted lines in Fig. 4. It is noted that the minimum seismic design loads in ASCE-7 [19] provide an anticipated reliability of 10% probability of collapse in the MCE_R earthquake, which is similar to the 2,475-year return period hazard level examined here. The SCBFs in this study meet or only slightly exceed a 10% probability of collapse at this hazard level.

In general, the NCBF and NCBF-R buildings are more vulnerable to collapse than their SCBF counterparts. This result is expected because the NCBF and NCBF-R buildings are able to develop premature failure modes in the brace or connection. It important to note, however, that the NCBF and NCBF-R buildings only rarely reached the potential-collapse performance state at return periods at or below 475 years. This result is in agreement with real building performance observed in such relatively more-frequent but less-intense earthquakes. Most potential-collapse cases at these hazard levels were a result of analysis nonconvergence, which are indicative of numerical instability but not necessarily structural collapse.

In most cases, the NCBF and NCBF-R buildings did not achieve the anticipated reliability of ASCE-7 [19] based upon the 2,475-year return period hazard level results. However, existing and retrofitted buildings may not necessarily be required to achieve this level of performance, and even economical retrofits to increase brace or connection deformation capacity can result in substantial performance improvements.

By comparing the fragility curves for each building across the specified fracture rotation, it is apparent that the effect of gusset-plate interface weld fracture was more pronounced in the NCBF-R buildings (larger variability between fragilities) rather than the NCBF buildings (smaller variability between fragilities). The brace fracture failure mode dominated the response of the NCBF buildings, and hence there was little difference in behavior between buildings with low gusset-plate interface weld fracture rotations and locally slender braces. When the braces are filled with concrete, the latter failure mode is effectively suppressed, and the fracture rotation has a significant effect on seismic performance.

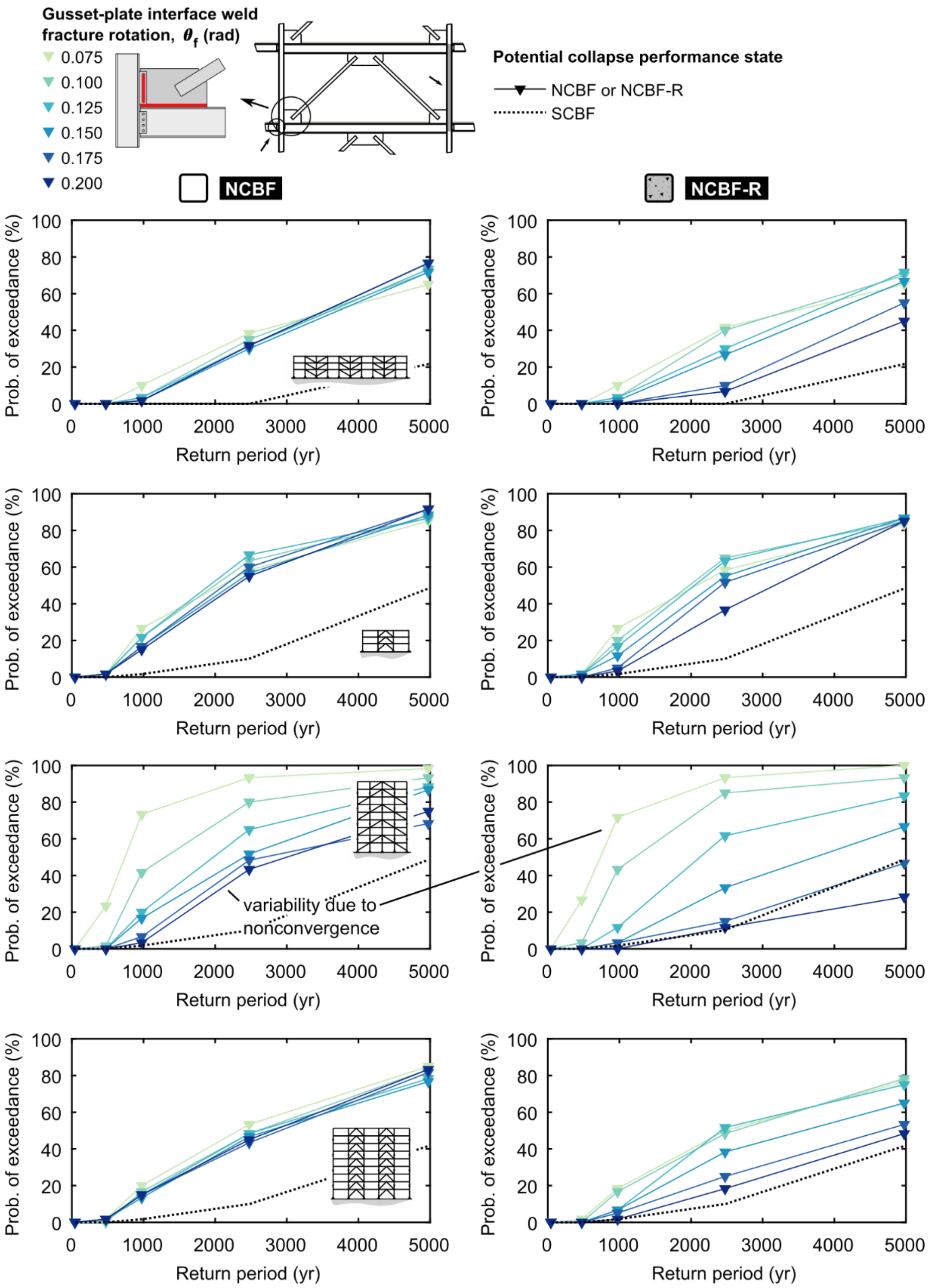

Fig. 4 – Collapse probabilities for NCBF and NCBF-R buildings with gusset-plate interface weld deficiencies

Especially high variability in collapse performance of the 9-story, paired single-diagonal configuration NCBF and NCBF-R buildings is noted in Fig. 4. These building were prone to nonconvergence, and thus the reader is cautioned from interpreting these archetypes as more vulnerable than others; further work is required to improve the robustness of these analyses.

5. Conclusions

Many NCBFs remain in service today around the world and are widely considered to be vulnerable to damage in moderate-to-large earthquakes. A large research program was initiated to quantify this vulnerability and develop impactful rehabilitation schemes through large-scale experimental testing and state-of-the-art, system-level nonlinear analysis. Testing conducted in the first phase of the research identified brace local slenderness and gusset-plate interface weld deficiencies as critical issues which affect seismic response. These deficiencies were shown to be mitigated in retrofit using concrete in-fill of tubular braces and gusset-plate weld reinforcement. In the second phase of the research, new approaches for modeling these deficiencies were developed and implemented in *OpenSees* using data collected from the present study and prior work in the literature. Finally, building models of NCBFs, retrofitted NCBFs, and SCBFs were analyzed probabilistically to evaluate collapse performance at multiple hazard levels.

The results indicate the clear collapse vulnerability of NCBFs in large earthquakes with intensities exceeding that associated with the 475-year return period. However, the collapse vulnerability of NCBFs with braces that are not vulnerable to premature fracture (i.e., compact or concrete-filled sections) can be significantly reduced by ensuring adequate rotational capacity of the gusset-plate interface welds. In the archetypes analyzed here, rotational capacities on the order of 0.175 rad or above were shown to provide performance expected of new construction in the US.

It is recognized that the results presented here are limited in scope, especially in view of the wide range of deficiencies in NCBFs; notably, the effects of beam and column strength deficiencies have not been examined in this paper. The effects of these other deficiencies in isolation and in interaction with those discussed here are the subjects of additional work in progress by the authors.

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