

# Structural Damage Analysis of Steel Special CBF and Steel Self Centering CBF Buildings

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## ABSTRACT

Steel special concentrically braced frames (SCBFs) are effective, economic and stiff lateral force resisting systems for steel structures. However, they have limited ductility capacity because of brace buckling. Steel self-centering concentrically braced frames (SC-CBFs) have been developed to have the effectiveness, economy and stiffness of SCBFs with increased lateral drift capacity and reduced residual lateral drift. A probabilistic structural damage assessment of SCBF and SC-CBF buildings is presented in this paper. The assessment shows the differences between the earthquake performance of SCBFs and SC-CBFs, and verifies the expected earthquake performance of SC-CBFs. Structural damage assessment is an essential part of performance based earthquake engineering (PBEE). In an earthquake structural damage assessment, the earthquake response of the structure is related to physical damage states observed in the structure after the earthquake. In this research, structural damage is considered as a combination of building damage (collapse/non-collapse and demolition/no demolition events) and damage of the braces of an SCBF or SC-CBF (repair actions corresponding to the damage).

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## 1. Introduction

Special concentrically braced frames (SCBFs) are effective, economical and frequently used earthquake lateral force resisting systems for steel structures. The braces provide large lateral strength and stiffness, and are the critical components of an SCBF. Deterioration and fracture of the braces under earthquake loading limits the capability of SCBFs to undergo large inelastic deformations. This limited deformation capacity is associated with significant damage to the braces leading to permanent lateral deformation (such as residual lateral drift) in the system. Self-centering concentrically braced frames (SC-CBFs) have been developed to maintain the effectiveness, economy and stiffness of SCBFs, and to have increased lateral drift capacity before damage and reduced residual lateral drift (Roke et al., 2006). An SCCBF is designed to rock on its foundation, and the rocking action increases the lateral drift capacity of the system. An SC-CBF has vertically-oriented PT bars that provide restoring forces to self-center the frame during the earthquake (Roke et al., 2010). The main structural components are designed to remain undamaged under the design earthquake to permit the system to self-center. The controlled rocking action and the self-centering behavior of SC-CBFs enable the structural damage to be concentrated into a few replaceable elements and to eliminate significant residual drift [1].

The main difference between SCBFs and SC-CBFs is expected to be in the damage to the braces and in the residual drift. The probability of structural damage and residual drift of an SC-CBF is expected to be very low under the design level earthquake. To verify the expected earthquake performance of SC-CBFs, compared to the earthquake performance of SCBFs, and to show the differences between the two systems, an earthquake

structural damage assessment for these two systems is performed in this study. Structural damage assessment is an essential part of the performance based earthquake engineering (PBEE). In an earthquake structural damage assessment, the earthquake response of the structures is related to physical damage states observed in the structure after the earthquake. Necessary repair actions are determined for the corresponding damage states [2].

The relationship between the structural response and the damage is expressed as the probability of being in a specific damage state. Four model buildings are used in the earthquake damage assessment in this study. 4- and 9-story SCBF building are designed for earthquake loading according to current building codes. The earthquake performance of these SCBF buildings is compared to that of previously designed 4- and 9-story SC-CBF buildings (Chancellor, 2013). These buildings are office-type buildings designed for a site in Southern California. Two different building heights were selected to see the effect of building height on the earthquake performance of SCBFs and SC-CBFs. Nonlinear dynamic earthquake response analyses of the model buildings which were performed for two ground motion intensity levels, namely the design level earthquake (DBE) and the maximum considered earthquake (MCE) (Tahmasebi, 2014). The results of these analyses are used for the damage assessment. Damage criteria are developed for the buildings and the braces. A probabilistic methodology is developed and used for the damage assessment. The assessment results for the model buildings are compared with each other [3].

## Report Scope

To achieve the research objectives, the following work was completed. Two SCBF model buildings with different building heights were designed according to the seismic

design procedures given in ASCE 7-10 (ASCE, 2010). These SCBF model buildings are used to make comparisons with SC-CBF model buildings designed previously (Chancellor, 2013) using the design procedure developed by Roke et al. (2010). The nonlinear dynamic earthquake response analysis of the SCBF and SC-CBF model buildings were performed by Tahmasebi (2014) and by Chancellor (2013); the results of these analyses are used in this study to obtain the structural response parameters of the model buildings under earthquake loading. Building damage criteria are established for collapse and demolition conditions. Brace damage criteria are established from a previous experimental study. A probabilistic methodology is developed and used to relate structural response parameters to specified damage criteria [4].

## 2. Literature review

Several research studies have been conducted on damage to buildings (i.e., on “building losses”) from earthquakes. The present research focuses on the potential for selfcentering systems to reduce earthquake losses. Probabilistic earthquake structural damage assessments will be conducted to compare the earthquake performance of SC-CBF and SCBF systems. The Pacific Earthquake Engineering Research Center (PEER) developed a performancebased earthquake engineering (PBEE) framework based on loss estimates, using conditional probability concepts and the total probability theorem (Moehle and Deierlein, 2004). An enhanced building-specific loss estimation procedure was developed by Miranda (2010) that considers building losses associated with collapse, repair of damage, and demolition. The Applied Technology Council (ATC) developed guidelines for performance assessment based on losses in the ATC-58-1 project “Seismic Performance Assessment of Buildings”. Other previous research has examined the performance-based design of SCBFs and SCCBFs. This chapter summarizes the concepts and results of recent research related to the PBEE methodology, and performance assessment of SCBFs and SC-CBFs [5].

The first generation PBEE approaches (e.g., FEMA 273, 350 and 351) tried to relate structural response parameters to performance objectives such as Immediate Occupancy (IO), Life safety (LS) and Collapse Prevention (CP). The design of a structure is considered to satisfy the performance objectives if deformations or forces in each component do not exceed the specified limits (Porter, 2003). Whittaker et al. (2003) listed the key shortcomings of the first generation PBEE approaches as follows [6]:

The structural response and demand are evaluated for the whole structure, whereas the performance assessment is done on the basis of damage of individual components (and most of the time, the weakest component's performance controls the structural performance)

Most criteria for the performance of structures in building codes are based on judgment instead of reliable data.

Most structural engineers presume that code-specified performance objectives are too conservative with respect to prescriptive criteria [7].

Specified performance levels do not consider the concerns of the stakeholders such as economic losses (in terms of repair costs), occupancy losses in damaged buildings, casualties, etc.

These shortcomings led to the development of an improved methodology that correlates structural response parameters to performance measures [8].

Miranda et al. (2004) proposed a performance-based approach that estimates the total loss in the building from damage as a summation of individual losses of building components. Analyses were done with an existing non-ductile seven story reinforced concrete building. It was stated that the total loss is the sum of losses associated with non-collapse and losses associated with collapse. In this approach, collapse (C) and non-collapse (NC) damage states are considered as mutually exclusive. The probability of collapse was estimated for two conditions: sidesway collapse and collapse as a result of loss of vertical carrying capacity. The analysis results showed that the second type of collapse is more critical for non-ductile structures (Aslani and Miranda, 2005) [9].

In another study, Zareian and Krawinkler (2006) developed a simplified PBEE approach. This simple procedure contains three domains: Hazard Domain, Structural System Domain, and Loss Domain. Building-specific economic losses are represented in a semigraphical way. The Structural System Domain and Loss Domain consider both noncollapse and collapse cases as separate sub-domains. This approach recommends to group building components into subsystems (at the story level or building level). By this way, components in the same subsystem can be related to a single response parameter. A comprehensive structural response database was established within the scope of this study. This database includes EDPs for various reinforced concrete moment-resisting frames and shear walls. This approach gives only the mean values of performance not a full probabilistic performance assessment. Mitrani-Reiser (2007) developed another performance assessment methodology that estimates economic losses in terms of repair costs, building downtime and human fatalities. An event-tree-based virtual inspection procedure was used to check the safety of buildings according to the current code-based guidelines. An analytical methodology was presented in the form of a toolbox (MatLab Damage and Loss Analysis) for damage and loss estimation [10].

A new reinforced concrete moment frame office building was analyzed in this study. Mean losses as a function of ground motion intensity level and expected total annual loss were estimated for different structural design approaches. For the components with available fragility functions, non-collapse losses were estimated in a component basis. The expected annual loss results indicated that the non-code conforming design had the worst performance. Additionally, an event-tree-

based fatality model was created to evaluate factors affecting human injuries and deaths. The fatality estimation results showed that there is no life safety risk in code-conforming building designs at all hazard levels.

In the seismic performance assessment approach of Ramirez and Miranda (2009), a story-based loss estimation is used. This approach estimates damage by directly relating structural response to loss of each story of a building through EDP-DV functions. There is no need to estimate component damage in this approach, because it is included in the building story losses. With this alternative approach, the performance assessment can be obtained in a more efficient manner. The total loss estimation was also modified to consider the losses associated with the demolition of a building that has not collapsed but cannot be repaired due to excessive residual deformations [11].

### 3. Seismic design and dynamic analysis

In this study, the seismic performance of SCBFs and SC-CBFs is evaluated from a damage assessment and compared. As part of this study, engineering demand parameters (EDPs) such as story drift and component deformation are estimated by dynamic analysis. These EDPs are then related to damage states to quantify the necessary repair actions. Four buildings are analyzed in this study: 4- and 9-story SCBF buildings (4SCBF and 9SCBF, respectively), and 4- and 9-story SC-CBF buildings (4SC-CBF and 9SC-CBF, respectively). These four model buildings are designed using design procedures described in this chapter. Analytical models of these four frames are created in OpenSees and the earthquake response of these models is determined from nonlinear dynamic earthquake response analysis. The peak story drift ( $\theta_m$ ), the peak residual story drift ( $\theta_r$ ) and the normalized residual out-of-plane displacement of the braces ( $\Delta_{or}$ ) of the CBFs are the EDPs extracted from the dynamic analysis results [12].

#### Design of SCBF

SCBFs consist of beams, columns, and diagonal bracing. The design of an SCBF starts with the determination of the base shear and the earthquake lateral forces. Member forces are then obtained from a structural analysis of the structure under the earthquake lateral forces. The brace design forces are taken from these analysis results. For the beams and columns, however, the design method for SCBFs from the AISC Seismic Provisions (AISC, 2010b) considers the maximum expected brace forces together with the gravity loads acting on the beams and columns (Powell, 2009). 2-dimensional SAP2000 models were used for the design of the 4- and 9-story SCBFs termed 4SCBF and 9SCBF, respectively. These SAP2000 models include only the main structural members (beams, columns, and braces) and a lean-on column connected to the left-hand side of the SCBF. The lean-on column is modeled as weightless and connected to the SCBF with weightless rigid link elements. The tributary seismic mass for one frame (which is one quarter of the total building mass) is applied to the lean-on column. The lean-on column in the model considers the P-delta effects due to the

total gravity load of the tributary area for one SCBF (corresponding to one quarter of the total floor area). Twenty five percent of the live load, as well as the full dead load, within the one quarter of the total floor area associated with one SCBF are applied to the lean-on column.

The connections between the beams and columns are modeled as full-moment connections without any moment release, but the brace connections are modeled as pin connections by releasing the moment at the ends of the braces. The columns are assumed to be fixed at the base and modeled accordingly [13].

Member Selection the 4- and 9-story SCBFs are designed for both ELF and RSA forces. The most economical design among the two is used for further analysis. Members are designed for the most critical load combination by using the SAP2000 Steel Design tool in accordance with the current AISC Design Specification. The SCBFs with section sizes are given in Figure 1 through Figure 2. Figure 3. shows the 4SCBF. There is only one design for 4SCBF because both the ELF procedure and RSA give the same design base shear and member section sizes for the 4SCBF [14].

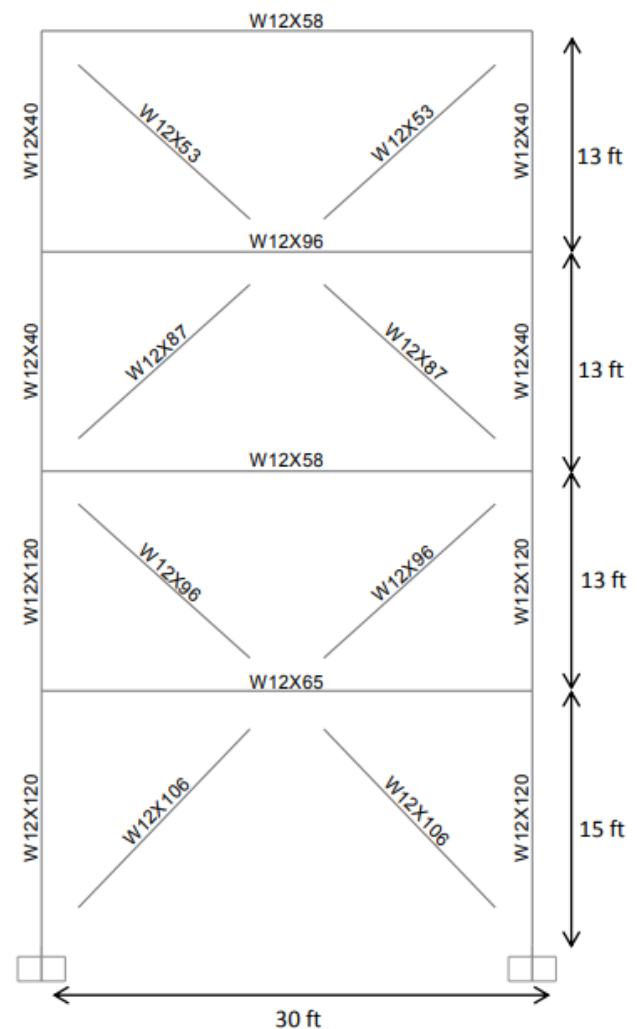


Figure 1: 4SCBF with designed section sizes

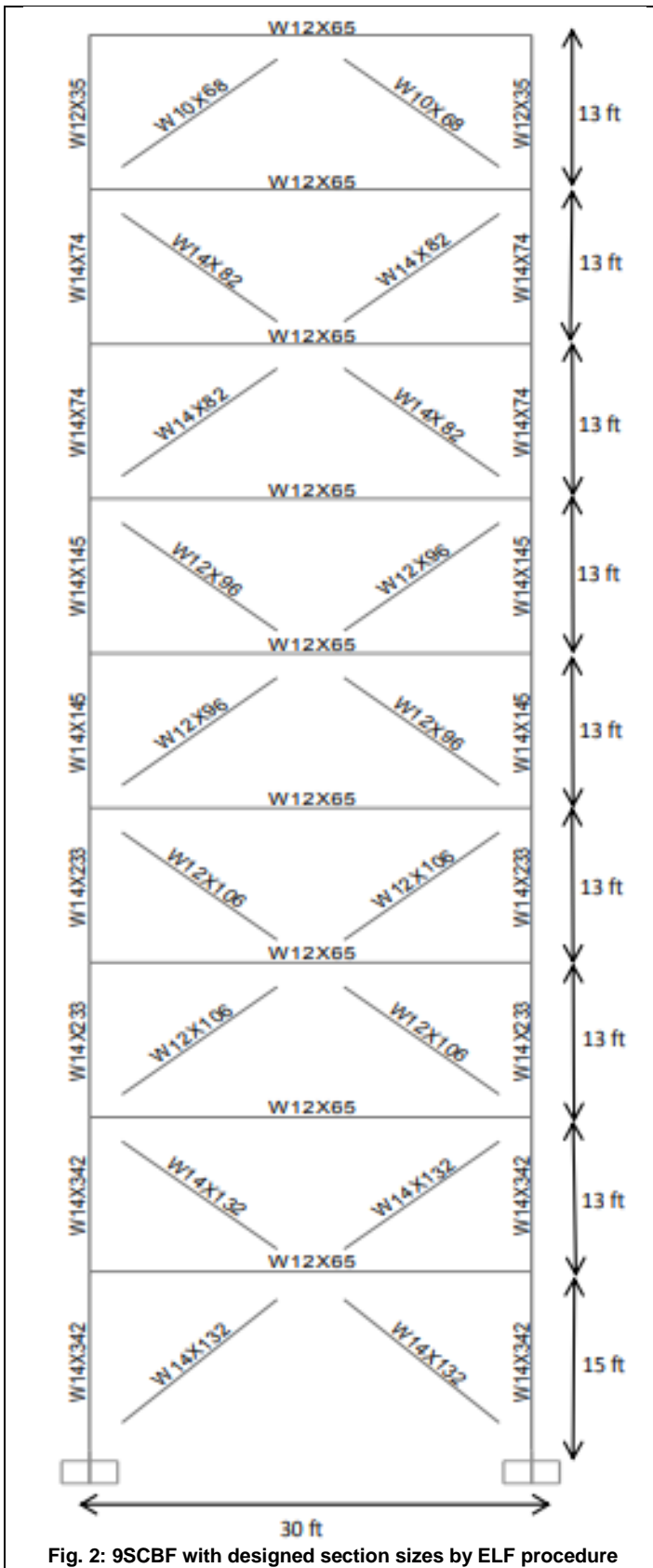


Fig. 2: 9SCBF with designed section sizes by ELF procedure

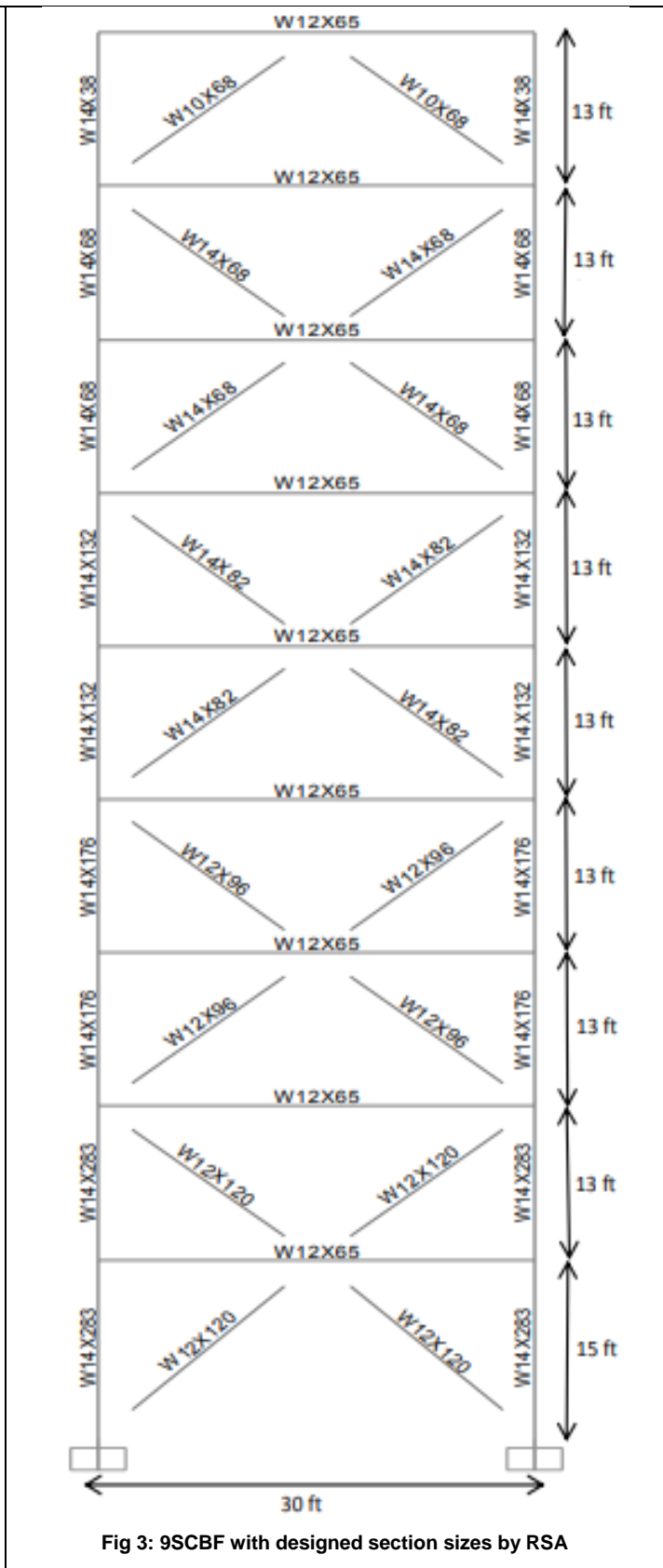


Fig 3: 9SCBF with designed section sizes by RSA

Nonlinear Numerical Models As mentioned earlier, nonlinear numerical models for the 4- and 9-story SCBF and SCCBF model buildings were created in OpenSees for the dynamic analyses by Tahmasebi (2014) and by Chancellor (2013). The nonlinear numerical models for the SCBFs include the frame members (beams, columns, braces) and a lean-on column (Tahmasebi, 2014). Beam-column

connections are modeled as rigid connections. The brace connections are modeled as moment-free pin connections. In a real building, buckling of the beams is restrained by the floor system and gravity load framing. The beams in the SCBF model are modeled without buckling to reflect this restraint. Column bases are modeled as fixed base. The nonlinear numerical models for SC-CBFs include the frame members

(beams, columns, and braces), two gravity columns adjacent to the SC-CBF columns, PT bars and a lean-on column (Chancellor, 2013). Beam-column connections are modeled as rigid connections. Gravity loads are applied to the lean-on column according to the corresponding tributary area of both the SCBFs and SC-CBFs [15].

#### 4. Building damage assessment

In performance based earthquake engineering (PBEE), "building losses" are estimated from a damage assessment. As mentioned in Chapter 2, Miranda (2010) stated that the total expected loss in a building is the summation of the losses associated with the building collapse case and the losses associated with the building non-collapse case. According to Miranda (2010), losses associated with the non-collapse case consist of the losses associated with the case when the damage is repairable, and the losses associated with the case when the damage is considered irreparable so that the building is demolished. In this study, the damage assessment is similar in concept to that of Miranda (2010). Building damage is classified into two damage categories: damage corresponding to building collapse, i.e., a total building loss, and damage corresponding to building noncollapse. Damage associated with the non-collapse case are specified further and classified into two damage categories: repairable damage and irreparable damage. Irreparable damage corresponds to building demolition which leads to total building loss as in the collapse case. The peak story drift ( $\theta_m$ ) is used as the structural engineering demand parameter (EDP) for the collapse assessment, and the peak residual story drift ( $\theta_r$ ) is used as the EDP for the demolition assessment. Building damage is expressed in terms of probabilities, such as probability of collapse/non-collapse and probability of demolition/no demolition.

#### Building Collapse Assessment

A critical objective of the seismic design of buildings is protection against collapse. Local collapse or global collapse may occur as a result of earthquake loading. Local collapse, which is also known as vertical collapse, occurs when a vertical load carrying component fails, or when the shear transfer between horizontal and vertical components is lost. Global collapse occurs if most components in the system experience local collapse or if a single story displaces so extensively that P-delta effects overcome gravity load resistance. Deterioration in the stiffness and strength of the components should be considered in a collapse assessment.

Therefore, the ability to represent stiffness and strength deterioration is an essential part of the model used in the nonlinear dynamic earthquake response analysis. In the numerical models used in this study, buckling of braces is modeled. The model accounts for deterioration of the braces in compression to some extent. Low-cycle fatigue and fracture of the braces is not considered in the models. This deterioration can be seen in results shown in Chapter 5. Beams and columns are modeled with fiber sections having a

steel material model with the Bauschinger effect. No deterioration is included in the models for the beams and columns [16].

#### Development of Collapse – Non-Collapse Fragility Functions

There are two approaches to develop collapse fragility functions: One approach is the IM-based approach. In this approach, the collapse capacity of a structure subjected to a given ground motion is defined as the ground motion intensity, IMc, at which the dynamic instability is observed. Incremental dynamic analysis (IDA) is performed to find IMc. IMc is often defined as the intensity at which a small increment of intensity causes a large increment in the lateral displacement (Krawinkler et al., 2009). IMc values are obtained for a large number of ground motions and a statistical evaluation of IMc is performed (Krawinkler et al., 2009). A probability distribution is fit to the IMc values to develop the collapse fragility function. The second approach is the EDP-based approach. In this approach, an EDP limit value, EDPc, is used as the collapse indicator. EDPc is defined to be the minimum value of the EDP corresponding to collapse. When the EDP value obtained from the dynamic analysis equals or exceeds EDPc, the building is considered to be in the collapse condition (i.e., the collapse event has occurred). In words, the probability of collapse when EDP has the given value EDPi equals the CDF for EDPc evaluated at EDPi. More succinctly,  $P(C|EDP) = CDF_{EDPc}(EDP)$ , that is, the probability of collapse for a given value of EDP is estimated using the CDF for EDPc, so the CDF for EDPc is the collapse fragility function.

#### 5. Conclusion

A probabilistic earthquake structural damage assessment of SCBFs and SC-CBFs was presented in this paper. The objective is to develop a better understanding of the earthquake performance of SC-CBFs by comparing this performance with that of SCBFs. An SC-CBF is designed to rock on its foundation, and the rocking action increases the lateral drift capacity of the system. SC-CBFs exhibit a capability to soften with little structural damage and residual drift. Therefore, the earthquake performance of an SCCBF is expected to be better than the earthquake performance of an SCBF because an SC-CBF has greater lateral drift capacity before damage of individual members begins. This paper summarizes the seismic design procedures for SCBFs and SC-CBFs. Structural response parameters (EDPs) obtained from nonlinear dynamic earthquake response analyses of model SCBF and SC-CBF buildings. A probabilistic building damage assessment of the model buildings, and a detailed discussion of brace damage and the probabilistic brace damage assessment for the model buildings are presented. Comparisons of the damage assessment for the SCBF and SC-CBF model buildings are given. The peak story drift ( $\theta_m$ ), the peak residual story drift ( $\theta_r$ ) and the normalized residual brace OOP displacement ( $\Delta_{or}$ ) were extracted from the dynamic analysis results. These EDPs were taken as random variables in the probabilistic damage assessment.

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