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Numerical investigation on design requirements for steel ordinary braced frames

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A B S T R A C T

The strength of beams and columns can significantly influence on the seismic performance of steel ordinary concentrically braced frames (OCBFs) due to the unbalanced forces that occur after buckling of braces. Current seismic provisions have required specific design requirements for beams and columns in steel OCBFs to sustain the loads induced under post-buckling limit state. To analyze the influence of the design requirements on collapse responses of steel OCBFs, this study carried out nonlinear static and dynamic analyses of 5- and 10-story prototype frames designed considering different design requirements relating to strengths of beams and columns. The analysis results showed that all prototype buildings satisfied the limitation of an inter-story drift ratio of 0.02 while significant differences are observed between the collapse capacities of each prototype building according to the design criteria of unbalanced forces. The collapse capacities of prototype frames were also evaluated using incremental dynamic analyses and FEMA P-695 methodology. It is observed that the collapse capacities of the prototype frames mainly depended on axial force-carrying capacities of the columns in the lower stories rather than those of the beams and that the use of strong columns improves the collapse capacities of the prototype frames.

1. Introduction

In contrast to steel moment resisting frames resisting to lateral loads using the flexural capacities of structural members, steel braced frames mainly carry them with the axial capacities of braces. The large axial stiffness of the braces in a steel braced frame can efficiently reduce seismic displacement demands. In a severe earthquake event, however, braces could buckle in compression resulting in quick stiffness and strength degradation. Such buckling phenomenon of compression braces in steel concentrically braced frames (CBFs) causes unbalanced forces illustrated in Fig. 1 in which *C* and *T* are, respectively, the compression and tension forces applied to the connection beam from the braces. This mechanism makes a softening story where the braces buckle and, in turn, a building suffers severe structural damage.

In order to prevent the undesirable behavior of steel CBFs, steel special concentrically braced frames (SCBFs) should be designed such that connecting beams should elastically resist to unbalanced forces inducing the yielding of tensile braces after the buckling of compressive braces [1]. Unlike the steel SCBFs, steel ordinary concentrically braced frames (OCBFs) are permitted to form the plastic hinges of connecting beams before tensile braces reach the yield strengths. For this reason, several experimental and analytical researches indicate that the seismic performance of steel OCBFs could be improved with employing stronger beams under the assumption that the columns must essentially remain elastic during strong ground motions [2–4]. However, the current design practices for steel OCBFs have stipulated that the columns have adequate axial strength to prevent global buckling without the consideration of concurrent bending moments. It provides no assurance that columns do not yield due to the combination of axial loads and bending moments. Tremblay and Robert [5] analytically demonstrated that the high flexural demands of columns are observed in steel OCBFs with more than 4 stories. Such high bending moments can lead to brittle failure of the columns in steel OCBFs. Since the unbalanced forces resulting from buckling of the braces are transferred to the columns, compression forces in the columns significantly increase [6,7]. The high compression forces usually reduce the ductility capacities of columns and, consequently, cause catastrophic building collapse. This phenomenon is more critical for the columns in the lower stories of higher buildings under a maximum considered earthquake (MCE). Furthermore, current seismic provisions, such as ASCE/SEI 7-10 [8] and KBC2009 [9] implicitly require that ordinary building structures under MCEs should be designed to satisfy the collapse prevention

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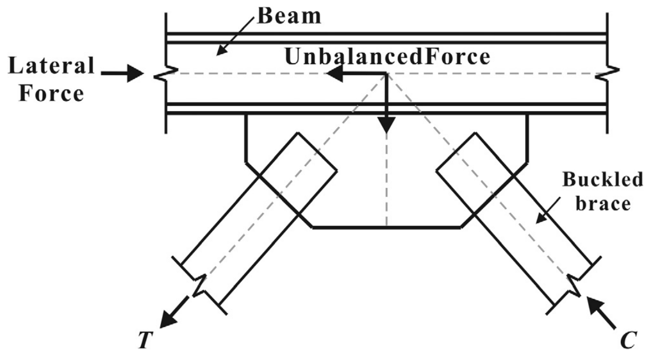

Fig. 1. Unbalanced forces in steel concentrically braced frames.

seismic performance. In other words, they provide low likelihood of collapse against very rare strong ground motion events. Nevertheless, there are very limited researches on the collapse capacities of steel OCBFs under MCEs and the influences of their seismic design requirements on the collapse mechanism. To address current issues relating to steel OCBFs, this study intends to review the design requirements for their beams and columns throughout broad analyses.

To achieve the purposes of this study, 5- and 10-story prototype buildings with steel inverted V-type OCBFs had been first designed according to KBC2009 similar to ANSI/AISC 341-10 [10] and nonlinear static analyses were then carried out to observe their monotonic behavior including the lateral resistances with resultant deformation capacities. Nonlinear dynamic analyses were also performed to investigate of their earthquake response and collapse modes using selected 40 historical earthquake records. Finally, the collapse capacities of steel OCBFs under the MCE intensity levels were evaluated using incremental dynamic analyses [11] and the probability based evaluation method of FEMA P-695 [12]. Based on the collapse capacities of the prototype steel OCBF buildings, the design requirements for structural members are discussed.

2. Seismic design and analytical model of steel ordinary concentrically braced frames

The floor plan and elevation of 5- and 10-story prototype office buildings are presented in Fig. 2. The prototype buildings have three bays in both principal directions and the story height at every floor is 3.6 m. Inverted V-type steel OCBFs located at the middle bay were designed as a seismic-force-resisting frame (SFRS) of the prototype buildings. The beam-column joints in the braced frame were assumed as "Welded Unreinforced Flange-bolted web (WUF bolted web)" [13] except for interior steel frames that were designed with pinned connections to carry only gravity loads consisting of a dead load of 5.0 kN/m^2 and a live load of 4.0 kN/m^2. Although well designed gravity frames would develop beneficial effects on the post-yield behavior of an overall structural system [14,15], this study neglected the contributions of gravity frames to the seismic performance of the prototype buildings to emphasize the seismic response of steel OCBFs.

The prototype building is assumed to be located at stiff soil in Seoul, Korea of which the MCE structural response acceleration is 0.22g and the site class is assigned to D. Table 1 summarizes the design spectral acceleration parameters in the short period range (S_{DS}) and at a period of 1.0 s (S_{D1}), and seismic design parameters, including the response modification factor (R), deflection amplification factor (C_d), overstrength factor (Ω_o) and importance factor (I). Note that, unlike ASCE/SEI 7-10, there is no height limitation although the prototype buildings are classified into the seismic design category D.

2.1. Member design of prototype buildings

The 5- and 10-story buildings with the inverted V-type steel OCBFs were designed according to the seismic design requirements prescribed in KBC2009 similar to the AISC provision. The structural members of the prototype buildings were preliminarily designed for the required strengths which are calculated using design gravity and seismic loads without the specific consideration of unbalanced forces. In the preliminary design, the brace members were sized to meet the requirements for slenderness ratios and the width-thickness ratios provided by the AISC provision. Taking advantage of the fact that the lateral displacements of a braced frame mainly depend on the stiffness of braces, inter-story drifts are, in advance, checked with the preliminary-designed braces and arbitrary assumed other structural members in order to minimize design iterations.

Table 2 summarizes the cross section area (A_g), nominal yield strength (P_y) and nominal buckling strength (P_{cr}) of the braces in the prototype buildings. According to the AISC provision, the beams in the steel CBFs need to be strong enough to resist unbalanced forces. To determine the maximum unbalanced force, the provisions requires that the force in a tensile brace is assumed to equal its expected yield strength, R_yP_y where R_y is a ratio of the expected yield stress to the nominal yield stress (F_y) of steel. On the other hand, the assumed force in a compressive brace must be larger than 30% of the nominal buckling strength, $0.3\phi P_{cr}$ where ϕ is a resistance factor of 0.9, which could represent the residual strength after buckling. Thus, the horizontal and vertical components of the maximum unbalanced force acting on a beam can be respectively obtained from:

$$Q_v = (R_yP_y - 0.3\phi P_{cr})\sin\theta \quad (1)$$

$$Q_h = (R_yP_y + 0.3\phi P_{cr})\cos\theta \quad (2)$$

in which θ is the inclined angle between a brace and a connecting beam.

Instead of the maximum unbalanced force calculated by means of the above equations, the provision also allows it to be computed by the maximum tensile force, $T_{\max}$ which is obtained by performing a simulation, such as a pushover analysis. In designing a connecting beam, the value of $T_{\max}$ is often used, since it is generally smaller than R_yP_y although this procedure is somewhat cumbersome in that an additional complicated simulation should be carried out. The vertical and horizontal components of the maximum unbalanced force can be calculated by plugging $T_{\max}$ instead of R_yP_y into Eqs. (1) and (2).

The beams in the prototype buildings are re-designed using both procedures for determining the unbalanced forces. In this paper, the first procedure is called as an empirical unbalanced force (E_{BE}) procedure and the other is called as a simulated unbalanced force (S_{BE}) procedure. Horizontal and vertical unbalanced force components together with gravity load (w_u), as shown in Fig. 3, are applied to the center of each floor beam with ideal fixed-ends, and significantly increase its axial force and flexural demand. The beams in the prototype buildings are designed to properly resist such unbalanced forces and their cross sections are summarized in Table 3.

With the axial forces added from unbalanced forces in beam members, this study considers two design approaches to determine the required axial strengths of columns in the prototype buildings. The first approach is to obtain the axial force demand of a column using the load combinations including the amplified seismic loads. On the other hand, the second approach is to design