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Bracing systems for seismic retrofitting of steel frames

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Abstract

The present study assesses the seismic performance of steel moment resisting frames (MRFs) retrofitted with different bracing systems. Three structural configurations were utilized: special concentrically braces (SCBFs), buckling-restrained braces (BRBFs) and mega-braces (MBFs). A 9-storey steel perimeter MRF was designed with lateral stiffness insufficient to satisfy code drift limitations in zones with high seismic hazard. The frame was then retrofitted with SCBFs, BRBFs and MBFs. Inelastic time-history analyses were carried out to assess the structural performance under earthquake ground motions. Local (member rotations) and global (interstorey and roof drifts) deformations were employed to compare the inelastic response of the retrofitted frames. It is shown that MBFs are the most cost-effective bracing systems. Maximum storey drifts of MBFs are 70% lower than MRFs and about 50% lower than SCBFs. The lateral drift reductions are, however, function of the characteristics of earthquake ground motions, especially frequency content. Configurations with buckling-restrained mega-braces possess seismic performance marginally superior to MBFs despite their greater weight. The amount of steel for structural elements and their connections in configurations with mega-braces is 20% lower than in SCBFs. This reduces the cost of construction and renders MBFs attractive for seismic retrofitting applications.

Keywords: Bracing; Buckling restrained braces; Steel frames; Concentrically braced frames; Moment resisting frames; Ductility; Seismic retrofitting; Performance assessment; Time history analyses

1. Introduction

Damage experienced during past earthquakes worldwide demonstrates that steel multi-storey building structures generally exhibit adequate seismic response (e.g. [1]). This is due to the favourable mass-to-stiffness ratio of base metal and the enhanced energy absorption of structural ductile systems employed. Nonetheless, relatively recent earthquakes, e.g. those in the 1994 Northridge (California), 1995 Kobe (Japan) and 1999 Chi-Chi (Taiwan), have shown that poor detailing of connections (e.g. beam-to-column, brace-to-beam, brace-to-column and column-to-base) and buckling of diagonal braces can undermine the seismic performance of the structure as whole (*see*, for example, [2–6]). Fig. 1 shows the distribution of damage level and the damage to structural members and connections with respect to structural type as surveyed in the aftermath of the 1995 Hyogoken-Nanbu (Kobe) earthquake [7]. Damaged buildings are classified as having unbraced (UFs) or braced (BFs) frames. Thus, considering the two principal framing orientations of a building, the surveyed structures include the following designations: UF-UF (unbraced frames in two horizontal directions), UB-BF (unbraced frames in one horizontal direction and braced frames in the other direction), and BF-BF (braced frames in both horizontal directions). Beams consisted almost exclusively of wide-flange sections, either rolled or built-up. For columns, wide-flange (H) sections were used most extensively; square-tube (S) sections were also utilized in some structural systems. Considering the 988 damaged steel buildings, 432 (43.7%) are UF-UF, 134 (13.6%) are UF-BF and 34 (3.4%) are BF-BF, with 388 (39.3%) having unidentified framing systems. These statistics indicate that the majority of damaged buildings had unbraced moment resisting frames (MRFs) as earthquake-resistant system. Fig. 1 also displays the location of damage, namely columns, beams, beam-to-column connections, braces and column bases, as a function of frame type. Major observations from the collected data are as follows [8]: (i) columns in UFs suffered the most damage relative to other frame elements (in terms of the number of

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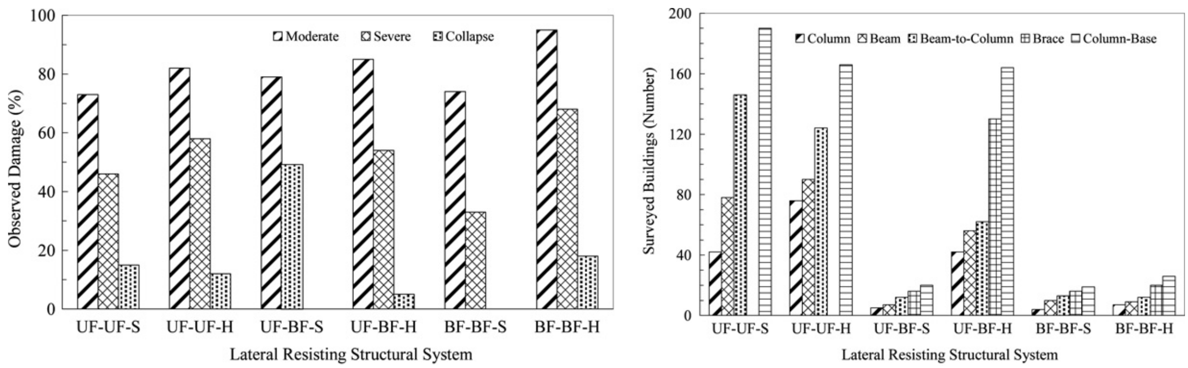

Fig. 1. Distribution of damage level (*left*) and damage to structural members and connections (*right*) with respect to structural type. *Key*: UF = Unbraced frame; BF = Braced frame; H = Wide flange sections; S = Square tube sections.

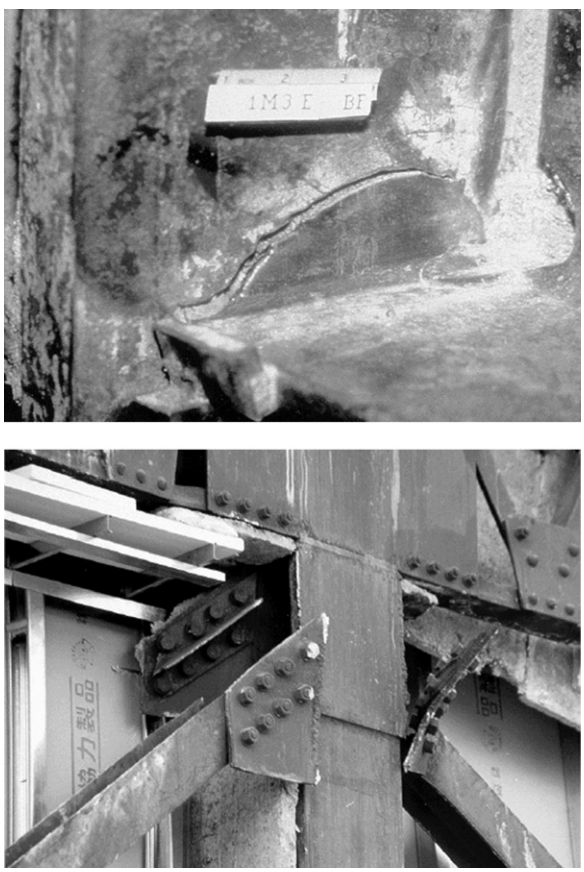

Fig. 2. Fracture in beam-to-column connections in the Northridge earthquake (*top*) and web tear-out in bolted brace-to-column connections during the 1995 Kobe earthquake (*bottom*).

buildings), while braces in BFs were the most frequently damaged structural element; (ii) damage to beam-to-column connections and column bases was also significant in UFs; (iii) damage to beam-to-column connections was most significant for UFs employing hollow section (square-tube) columns; and (iv) damage to columns was most significant for UFs with wide-flange members. The discussion of the above surveyed data is representative of typical structural response of steel buildings damaged by moderate-to-severe earthquake ground motions.

The occurrence of buckling, often in the plastic range in multi-storey buildings, erodes the capacity of the structure and may lead to sudden changes in the dynamic characteristics of the lateral resisting structure system. Brittle fractures, as for example, those depicted in Fig. 2 for beam-to-column and brace–column connections, impair the global ductile response of frames and hence their energy dissipation capacity under earthquake loads. As a result, beam-to-column connections and braces may be inefficient in ductile MRFs or concentrically braced frames (CBFs) if they are not adequately capacity-designed (e.g. [9–12] among many others).

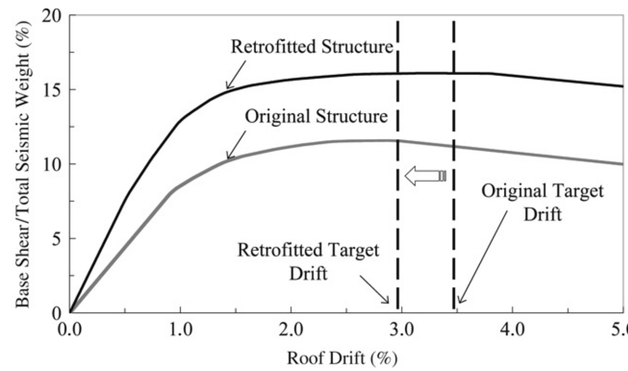

Fig. 3. Characteristics of global intervention approaches in seismic retrofitting of structures.

Bracing is a very effective global upgrading strategy to enhance the global stiffness and strength of steel UFs. It can increase the energy absorption of structures and/or decrease the demand imposed by earthquake loads whenever hysteretic dampers are utilized. Structures with augmented energy dissipation may safely resist forces and deformations caused by strong ground motions. Generally, global modifications to the structural system are conceived such that the design demands on the existing structural and nonstructural components are less than their capacities (Fig. 3). Lower demands may reduce the risk of brittle failures in the structure and/or avoid the interruption of its functionality and, in turn, the downtime due to the retrofitting, which are key features in the earthquake loss assessment [13,14]. The attainment of global structural ductility is achieved within the capacity design framework by forcing inelasticity to occur within dissipative zones (plastic hinges in MRFs and braces in CBFs) and ensuring that all other members and connections behave linearly. Diagonal braces can be aesthetically unpleasant where they change the original