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Seismic performance evaluation of a high-rise building with novel hybrid coupled walls

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ABSTRACT

In recent years, a novel type of hybrid coupled wall (HCW), which consists of reinforced concrete (RC) wall piers and replaceable steel coupling beams (RSCBs), has been proposed for enhancing the seismic resilience of high-rise buildings. This paper presents the assessment of the seismic performance of an 11-story building located in a highly seismic area and designed per modern Chinese codes. The building adopts the frame-shear wall interacting system. For comparison, two cases are considered: one using the novel hybrid coupled walls and another using conventional RC coupled walls (RCW). The dynamic response of the buildings under high intensities of ground motion shaking is obtained from nonlinear dynamic analysis in OpenSees. The seismic performance, expressed in terms of repair cost and time, is assessed based on the FEMA P-58 method. The results indicate that most of the damage is concentrated in the coupling beams and nonstructural components. The use of novel HCWs instead of conventional RCWs results in maximum interstory drifts up to 24.5% lower in the HCW building than the RCW building when subjected to a maximum considered earthquake (MCE) intensity. Novel HCWs result in a reduction of 50.8% and 60.5% in the median building repair cost and time, respectively, under MCE, due to less damage to coupling beams and RC frames and easy replacement of RSCBs after a damaging earthquake. It is also noted that HCWs have limited influence on the peak floor accelerations, and thus the repair cost and time for acceleration-sensitive non-structural components are similar for both the HCW building and the RCW building.

1. Introduction

Recent major earthquakes including the 2010 Maule earthquake (Chile, Magnitude of 8.8), 2011 Tohoku earthquake (Japan, Magnitude of 9.0) and 2011 Christchurch earthquake (New Zealand, Magnitude of 6.3), have demonstrated that our built environment and infrastructure, particularly in the urban context, need to be more resilient to earthquakes. In order to ensure minimal disruption to everyday life and business in urban society, prompt post-earthquake recovery of buildings is a clear need.

Coupled wall systems are often used in high-rise buildings due to their superior lateral strength and stiffness. In such a system, coupling beams are designed to undergo inelastic deformation and dissipate seismic energy, as shown in Fig. 1. Once damaged, traditional reinforced concrete (RC) coupling beams are expensive and time-consuming to repair. As a result, various types of replaceable coupling beams have been recently proposed as an alternative to traditional RC coupling beams (e.g., Fortney et al. [1], Christopoulos et al. [2], and Ji et al. [3]). Among them, the replaceable steel coupling beam (RSCB), as shown in Fig. 2, appears to be very promising.

The RSCB comprises of a central "fuse" shear link connected to steel beam segments at its two ends. By means of capacity design principles, the inelastic deformation can concentrate in the "fuse" shear links, while the steel beam segments remain elastic. Extensive studies [4,5] have indicated that a short shear link with proper detailing can provide very stable and ductile behavior under cyclic shear loading. In addition, specialized link-to-beam connections have been developed which can ensure adequate shear and flexural strength of the connections and allow the damaged shear link to be replaced easily, even in presence of residual drifts [3]. The RC slab above the RSCBs is elevated by a distance as proposed by Ji et al. [6]. Large-scale test results indicate that such "isolated slab" has very limited influence on the initial stiffness, shear strength and hysteretic performance of the RSCB, and that damage to the slab is minimal even under an inelastic rotation of the coupling beam of 0.05 rad, thus enabling quick and easy repair. The RSCBs and RC wall piers form a novel hybrid coupled wall (HCW) system whose superior structural performance against seismic action was demonstrated in a previous study [7].

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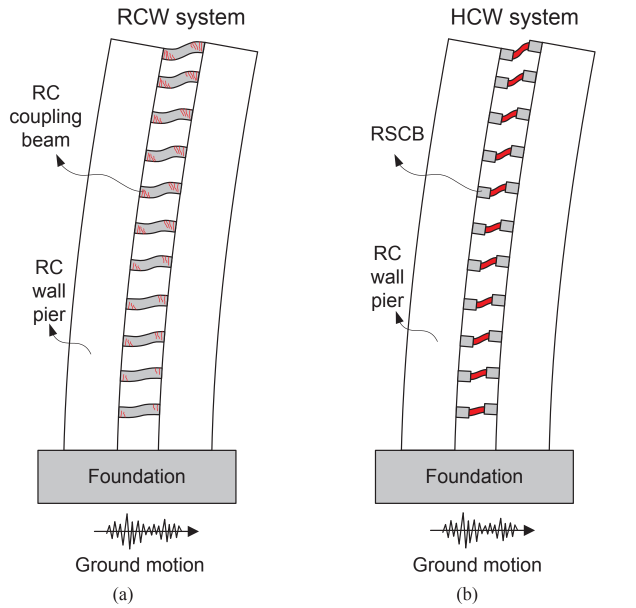

Fig. 1. Sketch of coupled walls. (a) RC coupled wall. (b) Hybrid coupled wall.

The objective of this paper is to evaluate the impact of adopting this novel HCW system on overall building performance, including damage to structural and non-structural components, as well as post-earthquake repair cost and time. A realistic 11-story office building is considered for a case study. To illustrate the superior performance and benefit in reparability of the high-rise building with HCWs using RSCBs (HCW building), its performance is compared against a building with commonly-used RC coupled walls (RCW building). To this end, Section 2 describes the structural design of the buildings considered. Section 3 presents the development of nonlinear numerical models and ground motion selection for nonlinear dynamic analysis. Section 3 also summarizes global responses of the HCW and RCW buildings when subjected to high intensities of ground motion shaking. Section 4 assesses seismic performance of the buildings in terms of repair cost and repair time based on the FEMA P-58 method.

2. Structural design

The prototype building is an 11-story office building located in Beijing, as shown in Fig. 2. It adopts a RC frame-shear wall interacting system. The total height of the structure is 48.5 m, and the plan dimension is 48.6 m by 14.4 m at the first floor, and 48.6 m by 17.65 m at other floors. The structural configuration and details of the prototype building are slightly modified from its as-built configuration, and are consistent with the representative floor plan shown in Fig. 2(b). The dead load of each floor including the self-weight of the floor slabs and the superimposed dead load varies from 5.5 kN/m^2 to 6.5 kN/m^2. The live load is 2.5 kN/m^2.

Based on the configuration of the prototype building, the structure using HCWs is designed according to the Chinese code for seismic design of buildings (GB 50011-2010) [8] and Chinese technical specification for concrete structures of tall buildings (JGJ 3-2010) [9]. The resulting dimensions of the beams in the frame are 250 × 700 mm, and the dimensions the columns range from 700 × 700 mm to 900 × 900 mm (see Fig. 2(b)). The thickness of shear walls is 300 mm. The dimensions of RSCBs are shown in Fig. 2(c). Linear response spectrum analysis of a three-dimensional structural model is performed to determine the design forces of the structural components and the deformation of the structure under the service level earthquake (SLE, with a probability of exceedance of 63% in 50 years), which has a peak ground acceleration (PGA) of 0.07 g. In this analysis, a damping ratio of 5% is assumed for all modes, and an accidental eccentricity of 5% is considered in each direction to account for torsional effects. In accordance with GB 50011-2010, the elastic stiffness E_cI_g is used for the RC wall piers and columns, as their deformations are very small under SLE (interstory drift ratio limit of 1/800). The stiffness of RC frame beams is taken as 1.5 E_cI_g (for exterior beams) or 2.0 E_cI_g (for interior beams), to account for the increased stiffness associated with the RC slabs above the beams. The structure is designed as a dual system, and the design shear force for the RC frames in each floor is adjusted to be over 0.2 V_0 or 1.5 V_{fmax} in accordance with GB 50011-2010 [8], where V_0 denotes the calculated total base shear force in the building and V_{fmax} denotes the maximum value of the calculated floor shear force in the RC frames across all stories.

The first three natural periods of the HCW building are 1.60 s, 1.51 s and 1.35 s, corresponding to the vibration modes of translation in the x direction (i.e., longitudinal direction), translation in the y direction (i.e., transverse direction) and the torsional mode, respectively. Fig. 3 shows the interstory drift ratios and shear-to-gravity coefficients of the structure under SLE. Note that the shear-to-gravity coefficient of a story denotes the ratio of the shear force of the story over all the gravity loads above this floor. The maximum interstory drift ratio is less than the upper limit 1/800 required in GB 50011-2010 [8], and the minimum

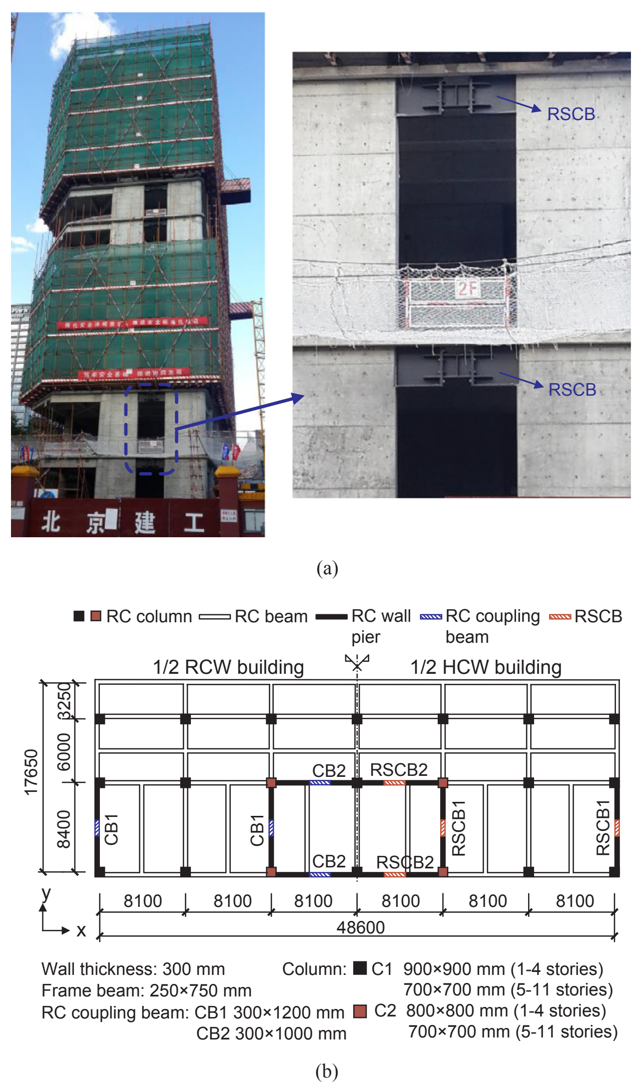

Fig. 2. Prototype structure. (a) Photograph of prototype building under construction. (b) Plan view (Units in mm). (c) Elevation view of HCW (Units in mm).