Engineering Structures 150 (2017) 622–635

Contents lists available at ScienceDirect

Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct

Seismic behavior and fragility curves of replaceable steel coupling beams with slabs

Xiaodong Ji [a,*], Yandong Wang [a], Junshan Zhang [a], Taichiro Okazaki [b]

[a] Key Laboratory of Civil Engineering Safety and Durability of China Education Ministry, Department of Civil Engineering, Tsinghua University, Beijing 100084, China
[b] Graduate School of Engineering, Hokkaido University, Sapporo, Hokkaido 060-8628, Japan

ARTICLE INFO

Article history:
Received 30 December 2016
Revised 26 May 2017
Accepted 14 July 2017

Keywords:
Replaceable steel coupling beam
RC slab
Cyclic behavior
Slab damage
Crack
Fragility curve

ABSTRACT

Replaceable steel coupling beams (RSCB) have been proposed as an alternative to conventional reinforced concrete (RC) coupling beams for enhanced seismic resiliency of coupled wall systems. This paper presents a series of quasi-static tests conducted to examine the seismic behavior of RSCBs with RC slabs and to identify reasonable slab configurations that can minimize the damage to RC slabs. A total of five large-scale specimens were designed and tested. The first four specimens adopted the same end plate link-to-beam connection but adopted different types of RC slabs, including a composite slab, bearing slab, isolated slab or slotted slab. The fifth specimen adopted splice plate link-to-beam connection and a bearing slab. The test results indicate that all specimens developed a large inelastic rotation capacity of more than 0.05 rad with stable hysteretic response. The presence of RC slabs is found to have limited effect on the shear strength and inelastic rotation capacity of RSCBs. Some types of RC slabs increased the initial elastic stiffness of RSCBs, but in the plastic stage, none of the slabs affected the loading or unloading stiffness. Among those four types of slabs, the composite slab suffered the most significant damage, as a result of pulling out of shear studs and subsequent pouching failure of the slab. Compared with the bearing slab or slotted slab, the isolated slab developed much fewer and smaller cracks, which should allow for easier repair. Based on the observations of this test and previous tests, four damage states for RSCBs were identified, corresponding to different repair methods. Fragility curves of RSCBs at various damage states were developed, which can provide the criteria for seismic performance assessment of RSCBs.

1. Introduction

Coupled wall systems are often used in high-rise buildings due to their superior strength and stiffness against wind load and earthquake action. Coupling beams distributed along the building height are designed as the components that undergo inelastic deformation and dissipate energy when the coupled wall systems are subjected to strong seismic motions. If detailed appropriately, reinforced concrete (RC) coupling beams show adequate seismic performance, but once damaged, post-earthquake repair of these components requires significant cost in expense and time. For enhanced seismic resiliency for buildings, there is a clear need to develop innovative coupling beams that are easy to repair or replace after being damaged. Recently, various types of replaceable coupling beams have been proposed and recognized as an alternative to RC coupling beams [1–5].

Fig. 1 shows a type of replaceable steel coupling beam (RSCB), which comprises a central "fuse" shear link connected to steel beam segments at its two ends. By appropriately proportioning the strengths of beam segments and the shear links, inelastic deformation and damage are expected to concentrate in the "fuse" shear links when the coupled wall is subjected to severe ground motion. Past studies (e.g., Kasai and Popov [6], Dusicka et al. [7], Ji et al. [8]) indicate that short shear link with proper detailing can provide large inelastic deformation, stable hysteretic responses and significant energy dissipation. 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 in presence of residual drifts [4]. Large-scale tests by Ji et al. [4] have demonstrated the excellent seismic performance and replaceability of the RSCBs. Nevertheless, the previous tests were on bare RSCBs, and they did not include the slabs above the coupling beams. The presence of RC slabs might influence the strength and deformation capacity of the beneath RSCBs. Moreover, RC slabs possibly suffer severe

* Corresponding author.
E-mail address: jixd@mail.tsinghua.edu.cn (X. Ji).

http://dx.doi.org/10.1016/j.engstruct.2017.07.045
0141-0296/

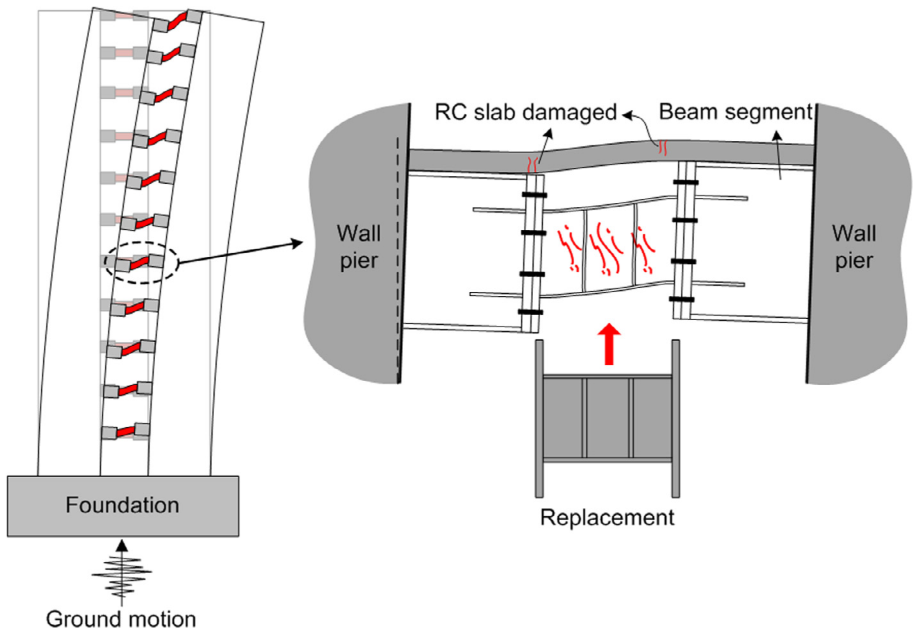

Fig. 1. Replaceable steel coupling beam with RC slab.

damage at large inelastic rotation of the RSCBs, which would influence the post-quake recovery of buildings.

To this end, the objectives of this paper are to examine how the seismic behavior of RSCBs is affected by the presence of RC slabs and to identify reasonable slab configurations for RSCBs that can minimize slab damage. This paper presents four types of RC slabs, i.e., composite slab, bearing slab, isolated slab and slotted slab. Large-scale specimens of RSCBs with different types of RC slabs were tested under cyclic loading. The next section presents the specimen design and experimental program. The third section details the test results, including hysteretic responses and damage to the RSCBs and above RC slabs. The fourth section examines the influence of RC slabs on the stiffness, strength and inelastic rotation capacity of RSCBs. Finally, based on the results in this test program and past tests, the fifth section develops fragility functions for RSCBs with RC slabs which provide the criteria for seismic performance assessment of RSCBs.

2. Experimental program

2.1. Test specimens

2.1.1. Replaceable steel coupling beams

Ji et al. [4] have reported cyclic shear tests of four bare RSCBs representative of 5/6-scale prototype coupling beams adopted in an eleven-story building. Except for addition of RC slabs, the specimens in this paper were identical in scale and dimension to the bare RSCB specimens in previous tests. Fig. 2 shows the geometry and details of the specimens. Two types of link-to-beam connections were adopted, i.e. end plate connection for Specimen CBS1 through CBS4 and splice plate connection for Specimen CBS5. For the end plate connection, the shear key set in the end plate was used to transfer all shear force and the high-strength bolts were designed to resist the bending moment. For the splice plate connection, the flange splices were designed to resist all the moment at the centerline of the splice and the web splices were design to resist all shear force acting at the centerline of the splice. Ji et al. [4] reported that these two types of specialized link-to-beam connections can ensure excellent seismic performance of RSCBs and allow easy replacement of damaged links.

Table 1 summarizes design parameters of the specimens. The shear links were built-up sections of H $350 \times 170 \times 10 \times 12$ (depth $\times$ flange width $\times$ web thickness $\times$ flange thickness, unit: mm) for Specimens CBS1 through CBS4 and H $350 \times 170 \times 10 \times 16$ for Specimen CBS5. The link flanges and web were welded by complete-joint-penetration (CJP) groove welds. Hybrid sections with lower yielding strength steel in web are used for the shear links to promote early yielding in shear and to increase their inelastic rotation capacity. The link web was low-yield-strength steel LY225 (nominal yield strength $f_y = 225$ MPa), the link flanges were Q345 steel ($f_y = 345$ MPa), and the stiffeners were Q235 steel ($f_y = 235$ MPa). Table 2 lists the material properties determined based on tensile coupon tests. Both the link flange and web satisfied the compactness requirement for highly ductile shear links by the AISC 341-10 provisions [9].

All links had a length ratio, $e/(M_p/V_p)$, smaller than 1.6 and, therefore, they were expected to yield primarily in shear. The stiffeners of shear link were full depth, welded to the link web and to both link flanges using fillet welds, and placed on one side of the web only. The stiffener spacing satisfied the limit for shear links specified by AISC 341-10 [9]. To delay web fracture at the region where the flange-to-web CJP groove weld and the stiffener-to-web fillet welds meet, the vertical fillet welds were terminated at a distance of no less than five times the web thickness from the flange-to-web weld [10,11].

To ensure that the beam segments remain elastic when the shear link fully yields and strain-hardens, their strength was designed to exceed the strength demand corresponding to the overstrength of shear link. Considering the very small length ratio of approximately 0.7, the overstrength factor of shear links was taken as 1.9 as suggested by Ji et al. [8]. Similarly, the strength of link-to-beam connection was designed to exceed the overstrength capacity of shear links. The details of link-to-beam connection design can be found in Ref. [4].

2.1.2. RC slab design

The slabs of all specimens had identical dimensions and reinforcement, as shown in Fig. 3. The slab width was 1500 mm, much larger than the effective flange width of composite beams which is 630 mm based on Chinese Code for Design of Concrete Structures