Retrofit of Existing Steel Moment Connections with Floor Slabs

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Abstract

The effectiveness of retrofit methods on the deformation capacity of existing steel moment connections with concrete slabs were investigated. Five full-scale beam-to-column subassemblages were tested. Tests included one conventional specimen and four retrofit specimens. Three types of retrofit methods for composite beam connections introduced in detail: RRS, RBS & RSS details. These retrofit methods applied to the bottom flange of beams only. Cyclic and quasi-static loading tests were performed. Test results revealed that the RBS was not, by itself, sufficiently improved for the deformation capacity of existing steel moment connections. Test results also exhibited that both the RRS and RSS details move the plastic hinge away from the face of the column and reduce stress levels in the vicinity of the beam bottom flanges, so these retrofit details improved the deformation capacity.

Keywords: Retrofit, Reduced beam section (RBS), RBS reinforced by horizontal stiffener (RRS), RBS shape horizontal stiffeners (RSS), Deformation capacity

1. Introduction

Many steel buildings suffered from fracturing of beam-to-column connections in the Northridge earthquake and Kobe earthquake (Miller, 1998; Nakashima, 1998). It was also found that most of the connections that failed were due to fracturing at the weld between the bottom flange of the beam and column flange plate. Thus research on steel structures in the second half of 1990s has been strongly characterized by significant efforts toward understanding the causes of the observed damages, assessing the major parameters which affect the cyclic behavior of steel moment connections and suggesting improvements to the connection configurations.

Most of their test specimens, however, did not consider the composite action of a slab that probably increases the strength of the connection without accompanying improvement of ductility in the connection. This is probably because they assume that the composite connection will behave like a bare steel connection. Considering the effect of the composite action, the beam became unsymmetrical and the neutral axis moved toward the top flange. This caused the strain on the bottom flange to be much larger than that on the top flange. Higher strain demand on the bottom flange led to premature failure of the connection (Okada, 2001; Oh, 2001).

Previous research has concentrated on pre-Northridge connections and new constructions. Less research has been directed towards retrofit methods for existing moment connections. In addition, the influence of a concrete slab is not fully understood for the case of a laterally loaded structure. In this study, a series of experiments on large size beam-to-column subassemblies, including the three retrofit connections, were conducted to improving their seismic resistance characteristics.

2. Composite connection tests

2.1 Test specimens

Large-scale structural testing was performed to examine the seismic behavior of composite moment connections. The test specimen is shown in Fig.1. Specimen represented a structural subassemblage isolated from the inflection point of a structure. The beam selected was the hot-rolled wide flange
section RH-612×202×13×23. The beam length from the loading point to the column surface was 3275 mm, or 3500 mm to the centerline of the column. The column selected was a built-up square tube section of Box-450x450x22. A relatively strong column was used to ensure that the beams could initiate the development of plastic hinge mechanism during cyclic loading before damage develops in the column. Also, to see the slab effects clearly, the panel zone (Box-450x450x22x32) was designed to remain elastic range even under severe lateral loading. Two diaphragm plates were inserted between the three separate pieces and shop welded all around by complete joint penetration (CJP) groove welding and extended from the column flange by 28 mm. Beam flanges are connected to the diaphragm plates through CJP single bevel groove welds made with gas shielded metal arc welding (GMAW), and beam web is welded to a column flange directly with fillet welds. Steel material is SM490. The concrete slab was designed with common concrete having design strength of 24MPa, which was 200 mm thick and 2500 mm width. Also, the concrete slab was provided with longitudinal and transverse steel reinforcement. For reinforcement, D13 (13 mm diameter steel reinforcing bars) having design strength of 400MPa were placed transversely at 200 mm spacing over the entire length of the concrete slab and were located both at 30 mm from the top and bottom of the slab. To make fully composite action between the beam and the slab, two rows of studs were welded to the top surface of the top flange at 200 mm spacing. In addition, the mould plate was chosen so that the slab would contribute more strength to the connection (Fig. 1), thereby simulating a more severe strain condition.

The list of specimens is shown in Table 1. All of five specimens have conventional type of the weld access hole in their beam-to-column connections designed prior to the Kobe earthquake (Fig. 1). Four of these specimens included a retrofit connection, except for specimen CON01. The specimen CON01 is intended to demonstrate fractures of the conventional connections that occurred in the Kobe earthquake and to compare with other retrofit specimens the deformation capacity of composite beams for fracture.

Fig. 2 shows the designs of three retrofit connections. RBS02 provided an RBS cut in the beam bottom flange (Fig. 2(a)). For new construction, RBS cuts are typically provided in both the top and bottom beam flanges. However, when retrofitting existing connections, making an RBS cut in the top flange may prove difficult due to the presence of a concrete slab.

In specimen RRS03, the RBS was reinforced by horizontal stiffeners at the beam-to-column connection to even further reduce stress levels as compared those of specimen RBS02 (Fig. 2(b)). Specimen RSS04 and RSS05 were designed with RBS shape horizontal stiffener. Fig. 2(c) and Fig. 2(d) show the sketches of these connections. Difference between RSS04 and RSS05 is distance from column face to minimum section of horizontal stiffener. Using this retrofit scheme, an enlarged plastic zone can be obtained in the pre-selected area as shown in Fig. 2(e).

In previous study (Kim, 2003), because of rigid horizontal stiffener, the strengthened section remained primarily elastic, so stress concentration occurred at the tip of horizontal stiffener. In this fact, the concept of RSS series differs from the previous one.

![Fig. 1. Test specimen](image)
Note that access to the connection may be limited, especially by the presence of a concrete floor slab which may limit or preclude any retrofits to the top flange. As such, this test was conducted on specimens that typically involved retrofits only to the bottom flange.

2.2 Test setup and procedure

The test, as shown in Fig. 3, simulates the boundary conditions of the subassemblage. A lateral support near the actuator was provided to prevent lateral deformation of the beam. Quasi-static cyclic loading was imposed by applying a predetermined cyclic incremental displacement history at the beam tip.

Imposed displacement amplitudes of $\theta$, which represented the rotation angle of only the pure beam. This rotation angle was incremented at $2\theta_p$, $4\theta_p$, $6\theta_p$, and so on after loading at the elastic rotation. Here, $\theta_p$ represents the rotation corresponding to plastic moment of bare steel beam.

3. Test Results

3.1 Hysteresis behavior

For the purpose of discussion of test results, moment versus rotation angle relationships ($\Theta$) were plotted in Fig. 4, where $\Theta$ is the rotation angle of only the pure beam. ▼, △ and ↓ shows the point where failure, local buckling and crack initiation of connections occurred, respectively.

As intended, specimen CON01 failed in the beam bottom flange, which was similar to the damage occurred during the Kobe earthquake (Fig. 5). The maximum plastic rotation was 0.012 rad. That is not adequate to survive a severe earthquake. During the cycles of $4\theta_p$, the initial ductile crack was located in the heat affected zone (HAZ) on the beam flange side. The crack progressed through the flange width during successive loading, followed by a sudden break of the entire beam flange.

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**Table 1. List of specimens**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Connection type</th>
<th>Retrofit method</th>
<th>Location of retrofit</th>
</tr>
</thead>
<tbody>
<tr>
<td>CON01</td>
<td>Conventional</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>RBS02</td>
<td>Retrofit</td>
<td>Reduced Beam Section</td>
<td>Bottom flange</td>
</tr>
<tr>
<td>RRS03</td>
<td>Retrofit</td>
<td>RBS reinforced by horizontal stiffener</td>
<td>Bottom flange</td>
</tr>
<tr>
<td>RSS04</td>
<td>Retrofit</td>
<td>RBS shape horizontal stiffener</td>
<td>Bottom flange</td>
</tr>
<tr>
<td>RSS05</td>
<td>Retrofit</td>
<td>RBS shape horizontal stiffener</td>
<td>Bottom flange</td>
</tr>
</tbody>
</table>
As shown in Fig. 5(b), failure of the web weld as well as the beam flange was observed. In Fig. 5(b), another interesting observation is the deformation of the column flange. The web of the box column was significantly less effective in transferring bending moment due to the out-of-deformation of the column flanges and the loss of the web sections by weld access hole as compared that of the H-section column. It was also found that the ductility capacity of the composite beam decreased as its joints efficiency degrades as compared that of bare steel beam (Kim, 2003; Okada, 2003). These facts would serve as other reason for the fracture occurring near the bottom flange.

In specimen RBS02, significant yielding of bottom flange was observed during the $4\Theta_p$ cycles. Yielding of beam bottom flanges extended from the column face to the narrowest beam section (Fig. 6). However, beam bottom flange of specimen RBS02 experienced brittle fracture before the cycles of $6\Theta_p$ was completed. Although the presence of an RBS cut delayed the beam fracture, the maximum plastic rotation of RBS02 was 0.019 rad.

Specimen RRS03 exhibited excellent performance. Significant yielding of bottom flange was observed during the $8\Theta_p$ cycles (Fig. 7). Specimen RSS02 developed a good plastic rotation of 0.039 rad. This amount of plastic rotation was about two times higher that of RBS02. The above results suggested that RBS connection only which has no additional modifications was detrimental.

Significant yielding of RSS04 occurred during the $4\Theta_p$ cycles. From the flaking pattern of the whitewash, Fig. 8 showed that yielding of the bottom flange occurred through the horizontal stiffener region. Specimen RSS04 achieved a maximum plastic rotation of 0.033 rad. The final failure was due to a crack at the toe of weld access hole of beam top flange, and the crack penetrated into the beam flange, during the cycles of $6\Theta_p$ under negative bending.

Specimen RSS05 delivered the best performance of all specimens tested. Fig. 9 showed the significant yielding and buckling of the beam during the cycles of $8\Theta_p$ under negative bending. The maximum plastic rotation was 0.05 rad. As demonstrated in the test, the plastic hinge mechanism of the beam of the RSS05 specimen formed away from the face of the column, and the yielding zone of the beam ensured the development of good hysteresis behavior and the dissipation even in composite specimens with slab.

Fig. 4. Moment-rotation angle relationships
3.2 Ultimate strength

The ratio of the ultimate strength to the calculated composite beam plastic strength (2480kN.m) under positive bending (when top flange is under compression) for specimen CON01 was about 1.02 and the ratio was 0.73 under negative bending (when top flange is under tension). The plastic capacity of the sections of these composite specimens was computed assuming an effective concrete stress of $1.3f'_c$ (AIJ, 1998). The ratio of the ultimate strength to the calculated bare steel beam plastic strength (1692kN.m) under positive for CON01 was about 1.08.

The results showed that the computed plastic capacity of the beam section estimated effectively the actual plastic capacity. These results indicated that the contribution of concrete floor slabs to the strength and stiffness of the connection can be ignored when the slab is subjected to tension, while the top flange is in compression, it creates the strongest composite action on the connection. For specimen RBS02, the ratio of the ultimate strength to the calculated composite beam plastic strength under positive was about 0.99 and it was due to the reduced beam top flange is under tension).
section. The ultimate strength of specimen RRS03 was about 1.21 times higher than the calculated composite plastic strength by the reinforcement of the horizontal stiffener. In RSS series specimens (RSS04 and RSS05), the ratio of the average ultimate strength to the calculated composite beam plastic strength under positive bending was about 1.45 and the ratio of average ultimate strength to the calculated bare steel beam plastic strength under positive was about 1.45. The average ultimate strength is 1.45 times larger than the strength when slab is under tension. This showed that the contribution of horizontal stiffener as well as the floor slab added considerable extra strength to the ultimate strength of the beam.

3.3 Strain behavior
Fig. 10 plotted strain distributions of bottom flange at critical section, which is 75 mm distance from the column face. From Fig. 10, it is found that in the same cumulative rotation angle, the strain on the bottom flange of CON01 was slightly larger than that of RBS02, while it was much larger than those of RRS03, RSS04 and RSS05. For example, at the same cumulative rotation angle, 0.0828rad, strain of CON01 was 3-32 times higher than those of retrofit specimens, except for specimen RBS02. The composite action from concrete slabs would result in an unsymmetrical section with larger deformation demand at the bottom flange. At the same cumulative rotation angle, 0.0828, strain of CON01 was 1.3 times slightly larger than that of RBS02. As above mentioned, this result clearly demonstrated that simply adding an RBS cutout to the beam flanges may not, by itself, be adequate to assure significantly improved connection performance. Fig. 11 showed the strain distributions in the beam’s longitudinal direction at completion of the peak moment. In CON01 and RBS02, the strain distribution changed exceedingly. But, RSS series specimens had strain distribution that changed gradually. Maximum strain value of specimen RRS03 occurred in RBS cutout region. These results illustrated that moment connections retrofitted using RRS and RSS method moved the plastic hinge away from the column face and, thus, achieved a more reliable connection performance even in composite connections.

4. Conclusions
Based on the experimental results, the following conclusions can be drawn concerning the retrofitted moment connections with floor slabs.

1. The conventional composite specimen, CON01 was very poor deformation capacity. This is due to floor slabs, the effects of which are considered as the strain concentration to the bottom flange.
2. The RBS connection, RBS02 was also not sufficiently improved for the deformation capacity as compared with conventional composite connection. However, RRS03 developed the excellent plastic deformation. It is speculated that in design of the RBS connection, additional connection reinforcement may be needed.
3. The RBS shape horizontal stiffener (RSS series) mitigated the stress concentration and developed the average plastic rotation of 0.036 rad. This rotation exceeded plastic rotation angle, 0.03 rad, required in special moment frame, even though this test was not based on a specified loading protocol (AISC, 1997). Further study is needed to elucidate the effects of horizontal stiffener and determine its dimensions since the conclusions presented here are based on a very limited experimental study.

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