SEISMIC UPGRADING OF STEEL MOMENT CONNECTIONS USING DOUBLE-SHEAR-TAB DETAILS

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Abstract: Steel moment connections with pre-Northridge details usually have difficulties developing a total rotation equal to 4% rad or more. There appears to be a necessity of improving the deformation capacity of the connections. However, the issue still did not receive much attention. To meet the needs of seismic upgrading, a series of full-scale tests have been carried out with SN490B steel beam-to-column connections enhanced with double shear tabs and super high strength bolts F14T. In detail, the connection of double shear tabs bolted to a beam web was tested first. That helps confirm the slip coefficient of high strength bolts. The beam flanges were then welded to the column and the connection were tested. The bolts were confirmed to have an average slip coefficient of 0.43. After correcting for the loss of pretension, the coefficient will get greater than 0.45, the value recommended by AIJ. The connection test stopped for the fracture occurring at the upper beam flange. The connection develops a deformation capacity more than 4%, but fails repeating the 2nd loading cycle in the AISC pre-qualification test. The result has shown a possibility of enhancing the connection deformation capacity using the double-shear-tab detail. That also suggest more attention be paid to connection details.

Keywords: Double shear tabs; Super high strength bolt (F14T); Steel beam-to-column connection; Connection test; seismic upgrading

DOI: 10.18057/ICASS2018.P.142

1 INTRODUCTION

Steel moment connections have been commonly used as moment connections in the areas of seismically active areas such as Northern California, Japan and Taiwan. For the ease of construction, a shear tab is usually welded to the face of a steel column in the shop work. The shear tab connects the column to the web of a steel H-beam with high strength bolts in the field. The beam flanges are then welded to the column face. Even though the connections were thought to be ductile, but many of the connections failed in a brittle fracture mode during the 1994 Northridge earthquake. Such earthquake damage has raised serious concern about the seismic reliability of steel connections and the adopted buildings.

In response to the 1994 Northridge earthquake, for example, the American Institute of Steel Structures (AISC) carried out a connection test program [1]. The removal of existing backing and welding tabs was then recommended as part of connection upgrading programs,
and the workmanship was required to achieve smooth surfaces free of welding defects [2]. Quite recently, the effectiveness of web attachment details were investigated using finite element analysis (FEA) and full-scale connection tests [3]. It is recommended to use one of the two welded beam web attachment details. One detail consists of a complete penetration groove welded web and supplemental fillet welds around the beam shear tab. The other has a heavy shear tab welded to the column with a complete penetration groove weld and to the beam web with fillet welds.

There has been a great amount of experimental and analytical work on the brittle failures of steel moment connections (e.g., [4-6]). While connections to wide flange columns have attracted much attention, connections to box columns have received little attention (Chen et al., 2004). In the past, steel moment connections were usually designed with a plastic rotation capacity equal to 1.5% rad. The connections with pre-Northridge details probably cannot meet the post-earthquake requirement of deformation capacity (i.e. 3% plastic rotation or 4% drift angle). In the present work, the result is presented of experimental work on steel H-beam-to-box-column connections with a double-shear-tab detail. The connection tests were carried out in the National Center for Research on Earthquake Engineering (NCREE), using the AISC 2005 loading protocol.

2 CONNECTION TESTS

2.1 Experimental setup

Figure 1 gives the details of experimental setup. As can be seen, the connection is composed of a steel BOX-750x750x28 column and an H-800x300x14x25 beam. Moreover, continuous plates (or the so-called inner diaphragms) are added inside the column at the same height of the beam flanges. As also can be seen there, there is an actuator imposing a displacement at the end of the beam with a velocity of 1 mm/sec. The distance from the column center to the loading point is 4500 mm. To enhance the structural stability of the setup, 2 pairs of lateral braces have been added at the center point and next to the loading point of the beam.

Figure 1: Experimental setup
2.2 Test program

In the first part of the test program, a steel bolted web connection is tested, as to determine the slip coefficients of the high strength bolts. Figure 2 gives the connection details. As can be seen, the beam web is connected to the column through a pair of shear tabs and 8 high strength bolts. As also can be seen there, the gap between the beam and column is 15 mm. That allows imposing a drift ratio of 2% on the connection. In the second part, a steel moment connection is tested with a double-shear-tab detail. Before the test, the 2 beam flanges were welded to the face of the column. Moreover, the number of high strength bolts increased to 12. The arrangement considers the demands of shear and moment on the beam web. Figure 3 gives the connection details.

![Figure 2: Details of a bolted-web connection.](image1)

![Figure 3: Details of a moment connection](image2)

2.3 Material properties

Table 1 lists the strength and elongation requirements of steel plates. In detail, the connection is composed of a built-up SN490B steel BOX-750x750x28 column and H-800x300x14x25 beam. A pair of A572 Gr. 50 steel PL15x180x640 plates have been used as
shear tabs to connect the beam web to the face of the column with F14T high strength bolts. The bolts have a diameter of 22 mm, and the size of a standard hole is 23.5 mm (see the details in Figures 2 and 3). The surfaces of shear tabs and beam web have sand blasted surfaces. Such surfaces are expected to provide the bolts with a slip coefficient of 0.45. Table 2 summarizes the mechanical properties of high strength bolts. For comparison, the properties of and F10T high strength bolts are also given in the tables.

Table 1: Mechanical properties of steel plates.

<table>
<thead>
<tr>
<th>Material</th>
<th>Yield strength (MPa)</th>
<th>Tensile strength (MPa)</th>
<th>Elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SN490B plate</td>
<td>≧ 325</td>
<td>490-610</td>
<td>≧ 21</td>
</tr>
<tr>
<td>A572 Gr. 50 plate</td>
<td>≧ 345</td>
<td>≧ 450</td>
<td>≧ 21</td>
</tr>
</tbody>
</table>

Table 2: Mechanical properties of high strength bolts.

<table>
<thead>
<tr>
<th>Material</th>
<th>Yield strength (MPa)</th>
<th>Tensile strength (MPa)</th>
<th>Elongation (%)</th>
<th>Area reduction (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F10T</td>
<td>≧ 900</td>
<td>1000-1200</td>
<td>≧ 14</td>
<td>≧ 45</td>
</tr>
<tr>
<td>F14T</td>
<td>≧ 1260</td>
<td>1400-1490</td>
<td>≧ 14</td>
<td>≧ 45</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Material</th>
<th>Effective area (mm²)</th>
<th>Pretension (kN)</th>
<th>Slip resistance (kN)</th>
<th>Shear strength (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F10T</td>
<td>303</td>
<td>209</td>
<td>171</td>
<td>439</td>
</tr>
<tr>
<td>F14T</td>
<td>316</td>
<td>299</td>
<td>260</td>
<td>639</td>
</tr>
</tbody>
</table>

3 TEST RESULTS

3.1 Test results of a steel bolted web connection

3.1.1 Preload loss

Figure 4: Arrangement of high strength bolts; (a) slip test and (b) connection test

Figure 4(a) gives an example illustrating a typical test of slip resistance [7]. As can be seen there, the total 8 bolts are divided into 2 groups and located in the 2 ends of the joint. As
also can be seen, strain gauges are preinstalled in the head of the bolts. That allows controlling the bolt preload and monitoring the loss. The slip resistance of the 2x2 bolts was tested for 2 shear planes. The preload of high strength bolts reduced by approximately 10%, as expected.

Figure 4(b) shows the arrangement of high strength bolts in the steel web connection. As can be seen, the beam web is connected to the column through a pair of shear tabs and 2 group of 2x2 bolts. The connection was tested using the AISC 2005 loading protocol. The test stopped after finishing the cycle with a max drift ratio of 2%. As also can be seen there, the max loss of bolt preload varies from 2.4% to 14.6%. A 3D optical measurement is set between the shear tab and beam web, and that gives a relative displacement of 0.26 mm. High strength bolts are thought to slip with a displacement varying from 0.15 to 0.5 mm. The measurement of preload loss and slip displacement give a consistent result.

### 3.1.2 Slip coefficient

![Figure 5: Analysis of a slip coefficient; (a) load-rotation relation and (b) acting forces](image)

Figure 5 (a) depicts the load and rotation of the beam. In detail, the max load is 117.4 kN at the drift ratio of 2% in the positive direction, and the load is to 119.9 kN in the negative direction. Fig 5 (b) gives the forces acting on the shear tabs, beam web and high strength bolts. The slip coefficient of high strength bolts can be calculates using the following equations,

\[
M_w = P \times L
\]

\[
V = P
\]

\[
F = M_w / d'
\]

\[
F_{slip} = \sqrt{[(F/4)^2 + (V/8)^2]}
\]

\[
\mu = F_{slip} / T / 2
\]

where
\( M_w \): moment transmitted from the beam web to the shear tabs (kN-mm)

\( P \): concentrated load applied at the free end of the beam (kN)

\( L \): length from the column face to the loading point (mm)

\( V \): shear of shear tabs (kN)

\( F \): horizontal force acting on one set of 2x2 high strength bolts (kN)

\( d’ \): average distance between the 2 sets of high strength bolts (mm)

\( F_{slip} \): slip strength of a high strength bolt (kN)

\( \mu \): slip coefficient

\( T \): pretension load (= 330 kN per bolt)

The calculation gives a slip coefficient of 0.43. The AIJ recommendation (2012) suggests that the slip coefficient be 0.45 for sand sprayed surfaces [8]. The coefficient becomes greater than 0.45 after correcting for the preload loss.

### 3.2 Test results of the steel moment connection

#### 3.2.1 Test preparation

Before the test, the welds between shear tabs and column face were repaired, because some cracks occurred during the preparation. The 2 beam flanges were then welded to the face of the column. As illustrated by Figure 6 (a), the technique of ultrasonic testing has been applied to detect the surface cracks and subsurface defects of the welds. Moreover, strain gauges are installed outside the shear tabs in the beam web and flanges.

The number of high strength bolts increases to 12, as shown in Figure 6 (b). As mentioned, strain gauges are preinstalled in the heads of the bolts. That allows controlling the preload before the test. That also makes it easy to monitor the loss during the test. A pair of tilt meters and diagauges are also set there, as to measure the shear deformation of the panel zone as well as the out-of-plane deformations of column faces. The connection was painted as to tell the yielding of steel plates during the test.

![Figure 6: Test preparation; (a) UT of welds, and (b) experimental measurement](image)

#### 3.2.2 Connection behavior

A cyclic loading test has been conducted with the steel moment connection with a double-shear-tab detail. As the drift ratio increases to 0.5\%, the paint starts to flake off from the beam flanges and tells
the yielding. After finished the 2%-drift-ratio cycles, crack initiates from the access hole, as shown in Figure. 7 (a). The cracks don’t make any difference in the load-rotation relation, as shown in Figure. 8 (a). The connection continues testing until the drift ratio increases to 4%. Right before the max strength in the 2nd cycle, fracture occurs at the upper beam flange, as shown in Figure. 7 (b), and brings the end to the test. The flaked-off paint also tells the yielding of the beam web, as shown in Figure. 7 (c).

As illustrated by Figure. 8 (b), the max loss of bolt preload varies from 11.5% to 43.8%. By comparing to the result of the web-bolted connection, as shown in Figure. 4 (b), the max loss of bolt preload increases to a greater extent, especially for the bolts located near the beam flanges and far away from the column face. As mentioned, a pair of tilt meters and diagauges have been set in the test. That allows analyzing the behavior of the column and panel zone. As depicted in Figure. 9(a), the panel zone has remained in elasticity during the test. As illustrated by Figure. 9 (b), the column does not have any out-of-plane deformation.

Figure 7: Failure mechanism; (a) crack initiation at access hole (at the drift ratio of 2%), (b) fracture of upper beam flange (at the drift ratio of 4%), and (c) yielding of beam web.

Figure 8: Test results; (a) load-rotation relation, and (b) max loss of bolt preload (- no data).

Figure 9: moment and rotation; (a) column and (b) panel zone (PZ).
4 CONCLUSIONS

A series of full-scale connection tests have been conducted to investigate the role of shear tabs and high strength bolts in steel moment connections. In the first part of the test program, a steel bolted web connection was tested. In the second part, the two flanges of the steel H-beam were welded to the face of the column, and then the steel moment connection with a double-shear-tab detail were tested. The bolts were confirmed to have an average slip coefficient of 0.43. After correcting for the loss of pretension, the coefficient becomes greater than 0.45, the value recommended by AIJ. The test stopped for fracture at the upper flange of the beam. The max drift ratio is 4%, but the connection fails repeating the second loading cycle in the AISC pre-qualified test. The result indicates that the double-shear-tab detail can eliminate stress concentration near the access hole, increasing the plastic rotation capacity of steel connections. The example has shown a possibility of enhancing the connection deformation capacity using the double-shear-tab detail. That also suggests more attention be paid to weld details.

REFERENCES


