Effect of Shear Panels on Elasto-Plastic Behavior of Beam-to-Column Connections of Steel Rigid Flame Piers

Y. MISHIMA\textsuperscript{1a}, K. ONO\textsuperscript{2}, T. MIYOSHI\textsuperscript{3}, and N. NISHIMURA\textsuperscript{4}

\textsuperscript{1} Infrastructure Technology Development Department, Hitachi Zosen Corporation, Japan
\textsuperscript{2} Department of Civil Engineering, Osaka University, Japan
\textsuperscript{3} Bridge Design Department, Hitachi Zosen Corporation, Japan
\textsuperscript{4} Professor Emeritus, Osaka University, Japan

Abstract

This paper describes the seismic performance of the beam-to-column connections of steel rigid frame piers. The purpose of the authors’ research is to ascertain the elasto-plastic behavior of the beam-to-column connections of a steel rigid frame pier, as this has yet to be clarified. To this end, cyclic loading experiments were conducted on test specimens with structural parameters similar to actual structures. This was done with a particular focus on the connections’ web panels, which are shear panels, in order to examine their buckling modes, strength, and ductility. A review of the experiment results revealed that, depending on what parameters were set, there were differences in both the damage process and the buckling mode. This review also clarified that the difference in the mode of buckling had a particularly large impact on the web panels’ ductility.

Keywords: steel rigid frame piers; beam-to-column connections; strength; ductility; buckling mode

1. INTRODUCTION

The beam-to-column connections of a steel rigid frame pier are subject to large stresses resultant from earthquakes, and have significant effects on the seismic performance of the pier. Various studies have been conducted (Miki and Kotoguchi 1991) on beam-to-column connections. However, many of these studies were targeted at the structural parameters that are different from those of actual structures. Therefore, the seismic performance of actual structures has yet to be clarified. In order to obtain basic data on the seismic performance of beam-to-column connections, the elasto-plastic behavior of actual structures is investigated in this paper, including the damage processes, buckling modes, strength, and...
ductility of beam-to-column connections, by taking into consideration the effects of the structural properties of actual structures. To that end, cyclic loading experiments were conducted on test specimens with a stiffened and detailed structure similar to actual structures. Experiments were conducted with a particular focus on the following structural parameters: the aspect ratio and stiffened structure of web panels that transmit shearing force between beams and columns, and fillets that are installed in order to reduce stress concentration and improve fatigue durability. Since we have reported on the effects of fillets in our previous paper (Ono et al. 2006), this paper looks at what effects the aspect ratio and stiffened structure of web panels have on the elasto-plastic behavior of beam-to-column connections.

2. OVERVIEW OF CYCLIC LOADING EXPERIMENTS

2.1. Structural Parameters of Test Specimens

The structural parameters of the test specimens were selected based on a survey on the performance of actual structures that were designed and constructed after the 1995 Kobe Earthquake. Experiments were conducted with three test specimens. Figure 1 shows the geometry of the test specimens and Table 1 shows their structural parameters. For the experiments, as a rule, rectangular cross sections were used with sides each measuring about 750 mm, which is about one-third of the size of the actual structures, and steel plates measuring 10 mm in thickness. Test specimens were made of steel plates with a yield stress of 544–570 MPa and a fillet was installed at the crossing of the beam and the column.

As a stiffened structure that affects the elasto-plastic behavior of beam-to-column connections, three stiffeners were installed at the cross section of the beam and the column. Based on the survey made on the performance of actual structures, it was assumed that the width-thickness ratio parameters \( R_R \) and \( R_F \) of flanges were about 0.5. Furthermore, the installation was based upon the stiffness ratio of longitudinal stiffeners, \( \gamma_l \), being equal to or less than one. \( R_R, R_F \) and \( \gamma_l \) were derived according to the following equations in the Specifications for Highway Bridges (Japan Road Association 2002a, 2002b).

\[
R_R = \frac{b}{t} \sqrt{\frac{\sigma_y \cdot 12(1 - \nu^2)}{E \cdot 4n^{2}\pi^{2}}} \\
R_F = \frac{b}{t} \sqrt{\frac{\sigma_y \cdot 12(1 - \nu^2)}{k_F \pi^{2}}} \\
\gamma_l = \begin{cases} 
4\alpha^2 n \left(1 + n \delta_1\right) - \frac{\left(1 + \alpha^2\right)^2}{n} & \left(\alpha < \sqrt[4]{1 + n \gamma_l}\right) \\
\frac{1}{n} \left[2n^2 \left(1 + n \delta_1\right) - 1\right]^2 - 1 & \left(\alpha \geq \sqrt[4]{1 + n \gamma_l}\right)
\end{cases}
\]

where \( b \) = plate width; \( t \) = plate thickness; \( E \) = Young’s modulus of steel; \( \sigma_y \) = yield stress of steel; \( \nu \) = Poisson’s ratio (=0.3); \( n \) = number of panels; \( k_F \) = buckling coefficient; \( \alpha \) = aspect ratio of stiffened plate; \( \delta_1 \) = cross-section ratio of one longitudinal stiffener; and \( \gamma_l \) = relative stiffness of longitudinal stiffener.
Table 1: Major parameters of test specimens

| Specimen | Beam (mm) | Column (mm) | Stiffener plate (mm) | Web panel | Aspect ratio \(\alpha\) | Yield stress \(\sigma_y\) (MPa) |
|----------|-----------|-------------|----------------------|-----------|------------------------|-------------------|
|          | \(B_b\)  | \(t_b\)    | \(H_b\)  | \(B_c\)  | \(t_c\)  | \(H_c\)  | \(h_s\)  | \(t_s\)  | \(N_p\)  |             |
| C-1      | 722   | 10       | 754    | 722   | 10       | 754    | 74     | 10       | 3        | 1.0         | 563             |
| C-2      | 722   | 10       | 1000   | 722   | 10       | 754    | 74     | 10       | 3        | 1.3         | 570             |
| C-3      | 722   | 10       | 754    | 722   | 10       | 754    | 74     | 10       | 1        | 1.0         | 544             |

2.2. Structural Parameters

The experiments focused on the following structural parameters: the aspect ratio and stiffened structure of web panels, which are considered to affect the strength and ductility of beam-to-column connections. The aspect ratio of web panels \(\alpha = H_b/H_c\), where \(H_b\) is the web height of the beam and \(H_c\) is the web height of the column) was as a rule set at 1.0, and set at 1.3 only for test specimen C-2 in order to examine the effects of the aspect ratio. Based on the characteristics of actual structures, as a rule three stiffeners of the same size as the general part of the columns were installed. However, the number of stiffeners were reduced to one for test specimen C-3 in order to examine the effects of the degree of stiffness.

2.3. Experiment Method

Figure 2 shows the loading device used in our cyclic loading experiments. As shown in the figure, experiments were conducted using an experiment device with a test specimen and a jack pin-supported on each end built into a part of the loading frame. The jack was installed so as to allow it to be tilted for loading at an angle of 35° from the vertical axis in order to reproduce the bending moment distribution generated when a lateral force is applied to a rigid frame pier.

The displacement was measured on the line connecting the beam and column diaphragms that are one panel away from the crossing of the beam and the column, as shown in Figure 2. The displacement measured on this line, which is called corner displacement \(\delta\), was used to control load and to organize the results of the experiments. The cyclic loading pattern shown in Figure 3 was used for the experiments. The displacement obtained when the average value of the strain in inner flanges measured at a distance of
50 mm from the crossing of the beam and the column, as shown in Figure 4, reached yield strain was used as the basic yield displacement, $1 \delta_y$. The strain of a web panel was measured at locations shown in Figure 5.

3. Results of experiments and considerations

3.1. Damage Process and Buckling Modes

Figure 6 shows the relationship between the load ($P$) and displacement ($\delta$) of the corner hysteretic curves ($P-\delta$) obtained from the experiments. The observed states of test specimens are also indicated in the figure. The initial yield of a web panel is defined as the state where the von Mises stress calculated from the strain measured by any of the strain gauges installed at the locations shown in Figure 5 has reached the yield stress shown in Table 1. The yield of a flange is defined as the state where the average value of the strain measured by strain gauges installed in the locations shown in Figure 4 has reached the yield strain. With all test specimens, the fillets buckled first as a result of load at an early stage as web panels deformed due to an increase in load, as shown in Figure 7. At the next stage, web panels yielded as
they continued to deform, with the stiffness of the beam-to-column connections transitioning from linear to nonlinear. Then, as load continued to increase, beam or column flanges yielded, after which the maximum load was reached.

Figure 6: Load – displacement of corner hysteretic curves

Figure 7: Out-of-plane buckling at fillet; Figure 8: Local buckling at inner flange in beam of C-1

Figure 9: Shear buckling at web panel of C-3 of C-3 (-2 δy); Figure 10: Local buckling at inner flange in column of C-2in column of C-2 (+5 δy)
Two modes of buckling occurred after the maximum load was reached, depending on the degree of stiffness of the web panels of test specimens: one is a buckling mode predominantly characterized by the local buckling of flanges, as shown in Figure 8 and the other is a buckling mode predominantly characterized by the shear buckling of web panels, as shown in Figure 9. The mode predominantly characterized by the local buckling of flanges was observed with test specimens C-1 and C-2. These specimens were sufficiently stiffened against shearing force by the use of three stiffeners. However, the locations of the local buckling of flanges differed between the two test specimens. Local buckling occurred on the inner flange of the beam with test specimen C-1, as shown in Figure 8, while buckling occurred on the inner flange of the column with specimen C-2, as shown in Figure 10. This difference was due to the fact that the bending stiffness of the beam of test specimen C-2 was greater than the bending stiffness of its column. On the other hand, the mode predominantly characterized by shear buckling was observed with test specimen C-3, which had only one stiffener and was less sufficiently stiffened compared with other test specimens. These results reveal that the degree of stiffness of web panels has significant effects on the mode of buckling after the maximum load was reached.

3.2. Effects of Structural Parameters on Strength and Ductility

Table 2 shows the yield load ($P_{YE}$), corner displacement ($\delta_{YE}$) observed at the time of yield load, maximum load ($P_{max}$), and displacement ($\delta_m$) observed at the time of maximum load obtained from the experiments. The yield of beam-to-column connections is defined by the transition of the stiffness of beam-to-column connections from being linear to nonlinear in the $P$-$\delta$ relationship. Specifically, $P_{YE}$ is calculated using the relationship of $P$ and the square of $\delta$ as can be seen in Figure 11, with reference to the past research (Hwang et al. 1994). Figure 12 shows the results of the $P$-$\delta$ envelope curves, which was normalized by $P_{YE}$ and $\delta_{YE}$, in order to compare the strength and ductility of the test specimens. In the following section, the effects of the structural parameters based on these results are examined.

| Specimen | $P_{YE}$ (kN) | $\delta_{YE}$ (mm) | $P_{max}$ (kN) | $\delta_m$ (mm) | $P_{max}/P_{YE}$ | $\delta_m/\delta_{YE}$ |
|----------|--------------|-------------------|----------------|----------------|-----------------|-------------------|
| C-1      | 737          | 5.6               | 955            | 30.0           | 1.30            | 5.38              |
| C-2      | 953          | 5.4               | 1205 (1156)    | 14.6 (25.0)    | 1.26            | 2.69 (4.63)       |
| C-3      | 739          | 4.7               | 966            | 17.0           | 1.31            | 3.65              |

Note: Figures in parentheses are recalculated values.

3.2.1. Effects of the Aspect Ratio of Web Panels

A comparison of test specimen C-1 ($\alpha = 1.0$) and test specimen C-2 ($\alpha = 1.3$) is shown in Table 2, and as can be seen there was no significant difference between these two test specimens in the ratio $P_{max}/P_{YE}$, which represents increase in their strength after yield. On the other hand, a considerable difference of nearly two-fold was observed between the two test specimens in the ratio $\delta_m/\delta_{YE}$, which represents increase in their ductility. However, the envelope curve of test specimen C-2 in Figure 12 indicates that the applied load decreased around the point where $\delta/\delta_{YE}$ was 2.7, increasing again thereafter. This decrease in load is likely to have been caused by a crack in the weld connecting the stiffener and the column flange. The crack was discovered when the inside of the test specimen was checked to identify the cause of the decrease. For this reason, we recalculated the results by regarding the displacement obtained at the time of maximum load ($P_{max} = 1,156$ kN) after load increased again as the maximum displacement ($\delta_m = 25.0$ mm). Our recalculation showed that increase in the specimen’s ductility,
represented by $\delta_m/\delta_{YE}$, was 4.63. Therefore, the difference in the increase of ductility of specimen C-2 compared with that of test specimen C-1 was only about 10%. These results lead us to conclude that the aspect ratio of web panels has no significant effect on ductility.

3.2.2. Effects of the Stiffened Structure of Web Panels

A comparison of test specimen C-1 ($N_p = 3$) and test specimen C-3 ($N_p = 1$) is shown in Table 2 and as can be seen there was no significant difference between these test specimens in terms of the increase in their strength, $P_{max}/P_{YE}$. On the other hand, test specimen C-3 showed an increase in its ductility, $\delta_m/\delta_{YE}$, which was smaller by more than 30% than the increase in ductility for test specimen C-1. This is considered to be a result of the difference between these two test specimens in buckling mode. Namely, with test specimen C-1, which had a web panel that was sufficiently stiffened, the buckling of the flange preceded the shear buckling of the web panel, whereas with test specimen C-3, which had a web panel that was insufficiently stiffened, the shear buckling of the web panel preceded the buckling of the flange, before the ductility of the web panel could serve its purpose. These results lead us to conclude that the post-yield ductility of beam-to-column connections is significantly affected by the degree of stiffness of web panels.

4. CONCLUSIONS

- 1) Damage to beam-to-column connections occurs in the following order: buckling of fillets; initial yield of web panels and yield of beam-to-column connections, which occur almost simultaneously; and yield of flanges. The damage process is terminated by the local buckling of flanges or the shear buckling of web panels.
- 2) Two modes of the buckling of beam-to-column connections were observed after the maximum load was reached. If web panels are sufficiently stiffened, the mode is predominantly characterized by the local buckling of flanges. Otherwise, the damage process is terminated by the shear buckling of web panels.
- 3) The aspect ratio of web panels has almost no effect on their strength and ductility within the range of variability of actual structures. On the other hand, the degree of stiffness of web panels does not have any significant effect on their strength but has a considerable effect on their ductility; specifically, their ductility decreases with the lowering of the degree of stiffness, which has a large

Figure 11: Transition point definition for stiffness of beam-to-column connection corner envelope curves

Figure 12: Load-displacement of corner envelope curves

Figure 13: Exemplary load-displacement of plastic range and elastic range envelopes
effect on buckling modes. Therefore, in order to improve web panels’ ductility, it is necessary to prevent shear buckling by sufficiently stiffening them.

REFERENCES

[1] Miki T and Kotoguchi H (1991). Experimental studies on collapse behaviors and deformation capacity of steel beam-to-column connections, Journal of Structural Engineering, JSCE, Vol. 37A, pp. 121–134.

[2] Ono K, Tahara J, Tanaka K, and Takahashi M (2006). Elasto-plastic behavior of beam-to-column connections with fillets of steel bridge frame piers, Proceedings of the 22nd U.S.-Japan Bridge Engineering Workshop.

[3] Japan Road Association (2002a). Specifications for Highway Bridges, Part II, Steel Bridges.

[4] Japan Road Association (2002b). Specifications for Highway Bridges, Part V, Seismic Design.

[5] Hwang W, Nishimura N, and Nishino Y (1994). Strength and ductility of panel zone in steel frame connections, Journal of Structural Engineering, JSCE, Vol. 40A, pp. 215–226.