Deformation Capacity of Composite Beam Determined by Rupture

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Abstract
In the Hyogoken-Nanbu Earthquake (Kobe Earthquake), damage of rupture in the beam-to-column connection was mainly observed in steel building structures. After the earthquake, a lot of experimental research works on the deformation capacity of steel beam, which is determined by rupture, were carried out. From those research works, the cause of the rupture, such as stress concentration at connection part and the lack of the quality of steel material, was pointed out. However, steel beams without slab were used in most of those research works. When the slab exists, large tensile strain is generated in bottom flange of beam under positive bending moment. So, it can be estimated that the existence of the slab greatly influences the deformation capacity of beam determined by rupture.

In this study, cyclic loading test on composite beam and steel beam was carried out. By comparing both experimental results, the effect of the slab on the deformation capacity was clarified. The conclusions were summarized as follows. The deformation capacity of composite beam is nearly half of that of steel beams without slabs. The reason why the deformation capacity of composite beam drastically decreases is the concentration of tensile strain at bottom flange. Though strain of bottom flange greatly develops in the positive bending in tension side, it does not develop much in the negative bending for compression side.

Keywords: Composite Beam; Rupture; Deformation Capacity

1. Introduction
In Hyogo-ken Nanbu Earthquake, many fractures of bottom flanges occurred at beam-to-column connections of steel framed structures. After the earthquake, many researchers tried to resolve the issues about fractures and ductility capacity of steel members. However, most of them ignored effects of slabs even though composite beams, which consist of steel beams and RC slabs strongly connected each other by stud connectors, are generally used for actual buildings.

A schematic diagram of beam-to-column connections subjected to seismic force is shown in Fig. 1. In positive bending, because of the RC slab resisting compression, the full section of the beam is subjected to tensile force and the tensile strain in the bottom flange becomes excessive, so that the ductility capacity of the composite beams is considered to reduce. However, in current structural design, ductility capacity of composite beams is supposed to be equal to that of steel beams without slabs.

In this study, ductility capacity of composite beams is investigated experimentally, focusing on flange fractures, which mainly determine ductility capacity of these beams.

2. Experiment
A list of the specimens is shown in Table 1 and the standard specimen (No.1) is shown in Fig. 2. The scale and the shape of the specimens were designed based on those of beam-to-column connections of medium-rise steel buildings. The columns and the panels are composed of thick plates over 22mm so that they have sufficient strength to remain elastic range during the experiment. A list of mechanical properties of the materials used for the specimens is given in Table 2.

Specimens consist of four composite beams (No.1, No.3~No.5) and one steel beam without slab (No.2). They have conventional type of weld access holes in their beam-to-column connections. Object of this experiment are: (a) to demonstrate rupture of composite beams in Hyogo-ken Nanbu Earthquake; (b) to investigate ductility capacity of composite beams.
for rupture; and (c) to clarify the effects of slabs on ductility capacity of composite beams. Parameters of specimens are as follows: (1) existence of a slab; (2) section properties; and (3) loading patterns.

The testing setup is shown in Fig.3. During the tests, lateral deformation is restricted by frames positioned at the ends of the lateral beams and the free end of the main beam.

The deformation of the specimens is defined in Fig. 4 as the rotation angle \( \theta \). The actuator is controlled so that \( \theta \) follows the target deformation shown in Fig.5. Pattern 1 is applied to all specimens except No.5, and pattern 2 is applied to No.5 to investigate effects of the difference in the loading patterns.

### Table 1. List of Specimens

<table>
<thead>
<tr>
<th>No.</th>
<th>Parameter</th>
<th>Beam Parameter</th>
<th>( \Delta M_p ) [kN m]</th>
<th>( \Delta M_p ) [kN m]</th>
<th>( \delta_p ) [rad.]</th>
<th>( t_c ) [mm]</th>
<th>Weld Access Holes</th>
<th>Loading Pattern</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Standard</td>
<td>R-H-612x202x13x23</td>
<td>2.24x10^3</td>
<td>1.41x10^3</td>
<td>0.0082</td>
<td>200</td>
<td>Conventional</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>(1) Existence of a slab without Slab</td>
<td>R-H-612x202x13x23</td>
<td>---</td>
<td>1.41x10^3</td>
<td>0.0082</td>
<td>0</td>
<td>Conventional</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>(2) Section properties Small Section Beam</td>
<td>R-H-596x199x10x15</td>
<td>1.69x10^3</td>
<td>0.99x10^3</td>
<td>0.0087</td>
<td>200</td>
<td>Conventional</td>
<td>1</td>
</tr>
<tr>
<td>4</td>
<td>(2) Section properties Thinner Slab</td>
<td>R-H-612x202x13x23</td>
<td>2.07x10^3</td>
<td>1.41x10^3</td>
<td>0.0082</td>
<td>140</td>
<td>Conventional</td>
<td>1</td>
</tr>
<tr>
<td>5</td>
<td>(3) Loading pattern Pattern 2</td>
<td>R-H-612x202x13x23</td>
<td>2.24x10^3</td>
<td>1.41x10^3</td>
<td>0.0082</td>
<td>200</td>
<td>Conventional</td>
<td>2</td>
</tr>
</tbody>
</table>

\( \Delta M_p \): full plastic moment calculation for composite beams [AIJ], \( \Delta M_p \): full plastic moment calculation for bare steel beams, \( \delta_p \): elastic rotation angle of bare steel beams subjected to \( \Delta M_p \), \( t_c \): thickness of slabs

### Table 2. Material Property

<table>
<thead>
<tr>
<th>Material</th>
<th>( \sigma_y ) [N/mm²]</th>
<th>( \sigma_u ) [N/mm²]</th>
<th>Y.R. [%]</th>
<th>( \varepsilon_u ) [%]</th>
<th>Flange</th>
<th>Web</th>
<th>Web</th>
<th>Flange</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel (SM400)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H-612x202x13x23 (No.1,2,4,5)</td>
<td>386</td>
<td>555</td>
<td>72.2</td>
<td>14.4</td>
<td>444</td>
<td>555</td>
<td>414</td>
<td>556</td>
</tr>
<tr>
<td>H-596x199x10x15 (No.3)</td>
<td>414</td>
<td>556</td>
<td>74.5</td>
<td>14.4</td>
<td>445</td>
<td>565</td>
<td>45</td>
<td>565</td>
</tr>
<tr>
<td>Concrete [( f_c = 23.5N/mm² )]</td>
<td>28 days</td>
<td>---</td>
<td>26.0</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
</tbody>
</table>

\( \sigma_y \): yield stress, \( \sigma_u \): maximum stress
Y.R.: yield ratio, \( \varepsilon_u \): elongation

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Fig. 2. Configuration of Specimen (No.1)

Fig. 3 Testing Setup

Fig. 4. Rotation Angle \( \theta \)
Fig. 5 Loading Pattern

Experimental results

Moment versus rotation angle relationships are shown in Fig. 6, where ▼ shows the point where the flange ruptured. Full plastic moment calculations for composite beams and steel beams are shown by broken lines as \( M_p \) and \( M_p^c \). Experimental results are listed in Table 3.

The failure mode of all the specimens is flange rupture initiated by a ductile crack which generated at the tip of a weld access hole. Rupture of the composite beam specimens occurred in positive bending state. The ductility capacity of specimens is compared by \( \theta_{\text{max}} \) and \( s_{\theta_{\text{max}}} \), where \( \theta_{\text{max}} \) is the maximum rotation angle in positive bending state in M-\( \theta \) relationships, and \( s_{\theta_{\text{max}}} \) is the maximum rotation angle in positive bending state in skeleton curves. The skeleton curves are converted from M-\( \theta \) relationships in the way shown in Fig. 7, and they are shown in Fig. 8. \( \theta_{\text{max}} \) and \( s_{\theta_{\text{max}}} \) of specimens are compared in Fig. 9. \( \theta_{\text{max}} \) of the composite beams are 50–60% and \( s_{\theta_{\text{max}}} \) of them are 40–50% when comparing to steel beam without slabs (No. 2). Differences in section properties and loading patterns have little effect on their deformation capacity.
### Table 3 Experimental Results

<table>
<thead>
<tr>
<th>No.</th>
<th>$K_e^+$ [kN.m/rad]</th>
<th>$K_e^-$ [kN.m/rad]</th>
<th>$M_{max}^+$ [kN.m]</th>
<th>$M_{max}^-$ [kN.m]</th>
<th>$\theta_{max}^+$ [rad.]</th>
<th>$\theta_{max}^-$ [rad.]</th>
<th>Fracture Part</th>
<th>Failure Mode</th>
<th>Temp. [°C]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>412000</td>
<td>262000</td>
<td>2500</td>
<td>-1730</td>
<td>0.0231</td>
<td>-0.0129</td>
<td>Bottom Flange</td>
<td>Brittle Fracture</td>
<td>29.0</td>
</tr>
<tr>
<td>2</td>
<td>157000</td>
<td>-----</td>
<td>2020</td>
<td>-1710</td>
<td>0.0416</td>
<td>-0.0268</td>
<td>Top Flange</td>
<td>Brittle Fracture</td>
<td>29.0</td>
</tr>
<tr>
<td>3</td>
<td>307000</td>
<td>185000</td>
<td>1860</td>
<td>-1150</td>
<td>0.0206</td>
<td>-0.0138</td>
<td>Bottom Flange</td>
<td>Ductile Fracture</td>
<td>29.0</td>
</tr>
<tr>
<td>4</td>
<td>314000</td>
<td>233000</td>
<td>2250</td>
<td>-1710</td>
<td>0.0214</td>
<td>-0.0131</td>
<td>Bottom Flange</td>
<td>Brittle Fracture</td>
<td>28.5</td>
</tr>
<tr>
<td>5</td>
<td>377000</td>
<td>262000</td>
<td>2440</td>
<td>-1950</td>
<td>0.0187</td>
<td>-0.0203</td>
<td>Bottom Flange</td>
<td>Brittle Fracture</td>
<td>28.5</td>
</tr>
</tbody>
</table>

$K_e^+$: initial stiffness under positive bending, $K_e^-$: initial stiffness under negative bending, $M_{max}^+$: maximum moment under positive bending, $M_{max}^-$: maximum moment under negative bending, $\theta_{max}^+$: maximum rotation angle under positive bending, $\theta_{max}^-$: maximum rotation angle under negative bending, Temp.: temperature

### 3. Effects of a slab on ductility capacity of composite beams

Focusing on the existence of slabs, the effect of the slab on ductility capacity is investigated from the result of No.1 and No.2. The strain data are measured in two sections, Section A and Section B, shown in Fig. 10. The examples of strain distributions for Section A under positive bending are also shown in the figure. The strain in the bottom flange of No.1 is larger than that of No.2, so that the neutral axis of No.1 seems to have moved to the upper flange side. This is the main reason for reduction of ductility capacity of composite beams.

Moment (M) versus curvature ($\phi$) relationships can be obtained from the strain data in each section. The positive bending part of the skeleton curves of M-\(\phi\) relationships are shown in Fig. 11. The ductility capacity around Section A (near the fracture point) is compared by the maximum curvature. The ductility capacity around Section A of No.1 is 65% of that of No.2.

As shown in Fig. 11, a restoring force of Section A is higher than that of Section B at same curvature. The difference in restoring forces between Section A and Section B of No.1 is about 35%, and that of No.2 is about 20%. This indicates that Section A of No.1 is relatively weaker than that of No.2, and that the deformation around the beam-to-column connection tends to be larger in No.1 than No.2, so that No.1 has reduced the ductility capacity. The curvature distribution diagrams for No.1 and No.2 specimens are shown in Fig. 12, and these are obtained at the maximum rotation angle of Fig. 8. It is shown that the plastic zone of No.1 was narrow and concentrated to the beam-to-column connection when flange fracture occurred.
4. Conclusion

In this study, ductility capacity of composite beams is investigated by cyclic loading tests. The results indicate that ductility capacity of composite beam is nearly half of that of steel beams without slabs. This is due to slabs, the effects of which are considered as the strain concentration to the bottom flange and the deformation concentration to the beam-to-column connection.

References