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Authors: Satoshi Yamada, Associate Professor, Tokyo Institute of Technology
Akira Wada, Professor, Tokyo Institute of Technology
Shoichi Kishiki, Graduate Student, Tokyo Institute of Technology
Toru Takeuchi, Associate Professor, Tokyo Institute of Technology
Kazuaki Suzuki, Manager, Nippon Steel Corp

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New Ductile Steel Frame Limiting Damage to the Connection Elements at the Bottom Flange of Beam-Ends

Shoichi Kishiki¹, Satoshi Yamada², Toru Takeuchi³, Kazuaki Suzuki⁴, Akira Wada⁵

¹ Graduate Student, Tokyo Institute of Technology
² Assoc. Professor, Structural Engineering Research Center (SERC), Tokyo Institute of Technology
³ Assoc. Professor, Dept. of Architecture and Building Engineering, Tokyo Institute of Technology
⁴ Manager, Building Construction Division, Nippon Steel Corporation
⁵ Professor, Structural Engineering Research Center (SERC), Tokyo Institute of Technology

Abstract
Many steel structures suffered damage at the welded beam-ends connections, in the Northridge and Kobe Earthquakes more than 9 years ago. Some of them lost structural functions, although many buildings avoided collapse so as to save human life. The loss caused the stop of social and industrial activities, and severe economic loss. In addition to this issue, a feature of the damage at the welded beam-ends connections was the brittle fracture at the bottom flanges. After the disaster, some studies tried to investigate the cause of the fracture, and others proposed new beam-ends connection details to resolve the issue. Nevertheless, in these studies, little attention has been paid to repairing damage caused in an earthquake.

The purpose of this study is to propose new ductile steel frame, which realizes not only structural performances but also easy repairing after an earthquake. In the proposed system, beams are bolted to columns with T-stubs, and plastic deformation, which is damage, is limited to the T-stubs at the bottom flange of beam-ends. And in this paper, the structural performances of the system were verified from quasi-static test on the half scale frame with the hysteretic dampers at the bottom flange of beam-ends.

Keywords: Steel structure, Beam-to-column connection, Bolted connection, Hysteretic damper, Damage-Controlled-Structure

1. Introduction
In the Northridge (1994) and Kobe (1995) Earthquakes, some buildings lost structural functions, although many buildings avoided collapse as to save human life. The loss caused the stop of social and industrial activities, and severe economic loss. At the stage of seismic design, it is important to consider restoring structures soon after an earthquake.

Today, most of high-rise buildings are designed following the Damage-Controlled-Structure ([1] Wada et al., 1992). This system consists of a mainframe and dampers as shown in Fig.1. The mainframe only supports gravity and can remain in the elastic range during an earthquake, because dampers absorb the input energy of the earthquake. Therefore, the structures based on the Damage-Controlled-Structure can be used continuously by repairing or exchanging dampers. However, in urban areas, most of steel structures are low-rise or medium-rise buildings with-out dampers. In the case of these buildings, naturally, energy absorption of an earthquake must be expected with the plastic rotation capacity of beam-ends. In the two earthquakes mentioned above, many steel structures suffered damage at the beam-end connections, and some brittle fractures of bottom flange occurred at the welded beam-ends. After the two earthquakes, a remarkable number of studies have been made on the beam-end connections in the U.S. and Japan. Some of them have tried to investigate the cause of these fractures, and others have proposed the new connection details to ensure adequate plastic deformation capacity at the welded beam-end connections. Those results improved welding practices.
using higher material toughness and geometrical modification of the weld access hole in Japan. The reduced beam section (RBS) and the horizontal-haunch beam have been proposed. However, the improved details and the proposed designs have been based on the plastic deformation capacity at the welded beam-end and near it. Consequently, although many test results have showed that each of them prevents a premature fracture and have a larger plastic deformation capacity, those have not mentioned repairing of the damage part.

2. New Steel Structure

The purpose of this study is to propose the new steel structure, which realize structural performances but also easy repairing after an earthquake. As mentioned above, in the case of low-rise and medium-rise steel structures, it is specially important things now. It is the cause that the conventional dampers, which are wall type, brace type, stud type, and shear link type, cannot be applied to medium-rise and low-rise buildings.

There are two conditions that must be met to make structures easily repairable. One of them is to constitute structures using exchangeable element. In other words, beams have to be bolted to columns. The other is to limit damage part to some of these exchangeable elements as the Damage-Controlled-Structure ([1] Wada et al, 1992) mentioned above.

The new system, which realizes the two conditions mentioned above, is shown in Fig.2. At the proposed system, the damage is limited to the exchangeable elements, which are T-stub at the bottom flange of beam-ends. The T-stub at the bottom flange has the weak section, which yields earlier than beam flanges and columns. Consequently, plastic deformations are limited to T-stub at the bottom flange of beam-ends under earthquakes. After an earthquake, by repairing or exchanging the T-stubs at the bottom flange of beam-ends, buildings can be used continuously. However, when the new T-stubs are applied to beam-end connections, the T-stubs have to show the same stable behavior in tension and compression. In other words, the buckling of the weak section has to be solved. In this paper, the buckling-restrained method is applied to the weak section in order to obtain the stable plastic deformation in compression. The buckling-restrained method, which is applied to the T-stub at the bottom flange, will be described later.

In this study, the term "the weak-web T-stub" is defined as the T-stub at the bottom flange, which has the weak section on web plate. And in order to use the weak-web T-stubs as the hysteretic dampers in the Damage-Controlled-Structure ([1] Wada et al. 1992), it must be noticed that the beams with the weak-web T-stubs should have the larger depth section than the conventional beam section in the medium-rise or low-rise steel structures.

3. Test Planning

3.1. Test Specimen and Set-up

The test specimen was the 1/2 scale model of medium-rise steel structures. It was one-story and one-span plane-frame. The test specimen consisted of beams, columns, connections and concrete slab. A main feature of this specimen was that beams were connected to columns by high strength bolts, and hysteretic dampers were installed at the bottom flange of beam-ends. The detail of beam and column, beam-ends connections, and concrete slab will be described later.

The test specimen and test set-up was shown in Fig.3. Assuming a height of inflection point was in the half of the height of a story, the columns of the specimen were cut and connected to the pin supports in the position. The lower pin supports were connected to reaction devices, and the upper pin supports were connected to a loading beam. And an actuator was mounted to a loading beam on the upper pin supports.

3.2. Details of Beams and Columns

A main beam was made of section (depth x flange width x web thickness x flange thickness) of 440x140x6x9. The other beams, which were in the rectangular position to the main beam and back, were made of sections of 194x150x6x9. Steel grades JIS SS400 were chosen for flange and web of both beams. Columns were made of cold-formed square-tubes, and had section (width x width x thickness) of 250x250x12. As shown in Table1, mechanical properties were obtained from tensile coupon tests according to JIS-1A.
Fig. 3. Test Specimen and Set-up (Unit: mm)

Fig. 4. Detail of Beam-End Connection (Unit: mm) and Strength of Connections
3.3. Details of Connections

The main beam was connected to columns by high strength bolts with T-stubs, connection elements. As mentioned above (2. and Fig.2), the connection elements were not conventional T-stubs. The detail of beam-ends connections is shown in Fig.4.

In order to keep beams and columns the elastic range, the yield strength of the weak-web T-stubs was designed lower than those of beams, columns and the T-stubs at the top flange. The yield strength of the weak-web T-stubs can be obtained by multiplying the sectional area of the weak-web by yield stress ((1.1.) in Fig.1). And the ultimate strength of the weak-web T-stubs, which was obtained from coupon tests before the main test, can be obtained by multiplying the sectional area of the weak-web by ultimate stress ((1.2.) in Fig.4). In the conventional seismic design in Japan, it is required that connections do not break until the connected members reach to full plastic condition sufficiently. In order to ensure this condition, connection factor \( D \) is usually introduced to the design formula. In the proposed system, significant plastic deformations are expected only at the weak-web T-stubs, the hysteretic dampers. In other words, it is required that connections do not yield until the weak-web T-stubs shows significant plastic deformation sufficiently. Consequently, in this test, the connections were designed considering ultimate strength of the weak-web T-stubs as a standard strength of design instead of yield strength of those.

The yield strength of the T-stubs at the top flange was obtained multiplying a sectional area by yield stress ((1.3.) in Fig.4). The slip strength of friction joints, which was obtained following the design standard ([3] AIJ, 2001), was designed to be larger than the standard strength of design. The yield strengths of tension joints at T-stub and skin plate of column were also designed to be larger than the standard strength of design ((2), (3) in Fig.4). The yield strengths of tension joints were obtained following in Fig.5 and Fig.6 ([3] AIJ, 2001, [4] Masuda et al., 2001). From the design of connections, plastic deformations are limited to the weak-web T-stubs, and the rotation point of beam-ends is kept at the top flange mentioned above (2. and Fig.2).

The steel material used for the weak-web T-stubs, the hysteretic dampers, was the low yield point (LYP) steel. Mechanical properties of the LYP steel are shown in Table 1. Note that Young’s modulus and Poisson’s ratio of the LYP steel are identical to those of conventional structural steels.
3.4. Detail of Buckling-Restrained Method

A detail of buckling restrained method at the bottom flange of beam-ends is shown in Fig. 7, and installing the buckling-restrained plate is shown in Photo. 1. A yield zone of the hysteretic dampers at the bottom flange was put between the bottom beam flange and the buckling-restrained. And unbounded materials for removing friction have to be stuck on the yield zone. The buckling-restrained plate and the bottom flange of beams were connected through the loose holes of the yield zone by high strength bolts. Note that high strength bolts had to be bound loosely so that the unbounded materials may not be crushed.

3.5. Detail of Concrete Slab

Metal deck sheets were placed on beams. Studs, which had a diameter of 13mm and a height of 80mm, welded in 170mm pitches through the metal deck sheets. Wire-meshes were placed 30mm above the metal deck sheets. Concrete with a thickness of 100mm was placed on the metal deck sheets. Material property of concrete as shown in Table 2 was obtained from concrete cylinder tests on the same day of the main test.

3.6. Loading Program

Quasi-static loading was carried out following to the loading program shown in Fig. 8. The loading program was based on the story drift angles, which were 1/200, 1/100, 1/50, 1/33, and 1/25 radian. The relative story displacement is calculated as a difference of the displacement in upper pin supports and lower pin supports. And the story drift angle can be found out by dividing the relative story displacement by the height of the story, which is a distance between pin supports.

3.7. Measurement

During the test, the shear force of specimen was measured by load cell, which was attached to the head of the actuator. A displacement of dampers and a out-of-plane deformation of beams were measured by displacement transducers. Strain gauges were glued on the beam flange of both ends, in order to observe the effect of concrete slab and damage of beam flanges. And the cracks of concrete slab were observed and written down, in order to grasp damage on concrete slab, when the deformation of the specimen was reached to each target story drift angle.

---

Table 1. Mechanical Properties of Steel Plate

<table>
<thead>
<tr>
<th>Sampled Plate &amp; Thickness</th>
<th>Grade</th>
<th>Yield Strength [N/mm²]</th>
<th>Ultimate Strength [N/mm²]</th>
<th>Elongation [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>the weak-web T-stub</td>
<td>t = 16</td>
<td>LYP225</td>
<td>219</td>
<td>293</td>
</tr>
<tr>
<td>Column</td>
<td>t = 12</td>
<td>BCR295</td>
<td>388*</td>
<td>464</td>
</tr>
<tr>
<td>(Rbox-250x250x12)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam (BH-440x140x6x9)</td>
<td>Web (t = 6)</td>
<td>SS400</td>
<td>354</td>
<td>475</td>
</tr>
<tr>
<td></td>
<td>Flange (t = 9)</td>
<td>SS400</td>
<td>311</td>
<td>452</td>
</tr>
<tr>
<td>Beam (RH-194x150x6x9)</td>
<td>Web (t = 6)</td>
<td>SS400</td>
<td>382</td>
<td>484</td>
</tr>
<tr>
<td></td>
<td>Flange (t = 9)</td>
<td>SS400</td>
<td>316</td>
<td>456</td>
</tr>
</tbody>
</table>

* 0.2% offset strength

Table 2. Mix Proportion and Compression Test (Concrete Slab)

<table>
<thead>
<tr>
<th>W / C [%]</th>
<th>sand percentage [%]</th>
<th>weight per unit volume [kg/m³]</th>
<th>water</th>
<th>cement</th>
<th>fine aggregate</th>
<th>coarse aggregate</th>
<th>admixture</th>
</tr>
</thead>
<tbody>
<tr>
<td>55.5</td>
<td>47.5</td>
<td>162</td>
<td>292</td>
<td>863</td>
<td>968</td>
<td>0.73</td>
<td></td>
</tr>
</tbody>
</table>

Compression Test (age : 25 [days])
- Young's modulus : 2.82 [× 10⁹ N/mm²]
- slump : 15.5 [mm]
- compressive strength : 19.69 [N/mm²]
- air content : 5.2 [%]

(specified design strength : Fc=21 [N/mm²])

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Fig. 8. Loading Program
4. Test Results and Consideration

[Behavior of Frame Specimen]

Shear force $Q$ versus the story drift angle $\gamma$ relationship obtained from the tests is shown in Fig.9. Although the depth-to-thickness ratio of beam web plate was larger than the conventional details to which plastic deformation was expected, the specimen showed the sufficient plastic deformation and stable behavior. Finally, the shear force degradation at the point to which it points, caused the ultimate state. Not the fracture of a hysteretic damper but the out-of-plane lateral deformation at the bottom flange of the compression side beam was the cause of the degradation. The state of the lateral deformation will be described later.

[Yield Strength and Elastic Lateral Stiffness]

In the figure, the horizontal broken lines are the prediction value of the yield shear force, which can be calculated following a model in Fig.10. The bending yield strength of beam-ends can be obtained by multiplying the yield strength of the weak-web T-stub by the beam depth. Furthermore, assuming a plastic hinge occurs to the beam both ends in the model in Fig.10, the yield shear force can be calculated. Here, note that the effect of concrete slab was disregarded on calculation of bending strength. The prediction value of the yield shear force is in good agreement with the bending point of the skeleton curve extracted from the $Q$ versus $\gamma$ relationship.

And the diagonal broken line is the prediction value of the elastic lateral stiffness, which can be calculated following a model in Fig.11. Bending and shear deformations are considered on beam and columns. And panels of beam-to-column connection are considered as the shear panel. The connection parts of T-stubs are considered as the rotation spring, which set the top flange as the rotation point. A detail of the calculated model is described in [1] Kishiki et al, 2004. Here, note that the effect of concrete slab was disregarded again on calculation. The prediction value of the elastic lateral stiffness is in agreement with the $Q$ versus $\gamma$ relationship.

[Out-of-Plane Deformation]

In order to evaluate the out-of-plane deformations, a rotation angle was evaluated by dividing the deformation by the length from a measurement point to beam-end. The out-of-plane rotation angle $\Theta$ versus the story drift angle $\gamma$ relationship is shown in Fig.12. The out-of-plane rotation of left side began to increase when the story drift angle reached to $-1/33$ radian. When a negative deformation of specimen occurred, the hysteretic damper at the bottom flange of left side suffered compression axial force. At the ultimate state of specimen, $-1/25$ radian, the bottom flange of the left side deform in the direction of out-of-plane as in Photo.2.
[Composite Action and Hysteretic Dampers]
A time history of beam flange of both sides is obtained from the strain gauges and is shown in Fig.13. And a time history of axial deformation of the hysteretic dampers at the bottom flange of beam-ends is obtained from the test results and is shown in Fig.14. The time history of strain at the bottom flange was expressed with the solid line. When concrete slab resists compression force, at the bottom flange, two times as large strain as the top flange occurs. It is the composite action by concrete slab and beam. And it is the cause that the brittle fractures at the bottom flange in the Northridge and Kobe Earthquakes. However, when attention is paid to the strain at the bottom flange of both sides, the symmetrical change of the strain is observed. The same of strain at the bottom flange can be said of the hysteretic dampers at the bottom flange. The symmetrical change of deformation of the hysteretic dampers of both sides is observed. And it meant damage concentrated on the dampers at the bottom flange, and having formed the state as in Fig.2. In other words, it meant that the state of frame specimen was in the state assumed in Fig.1. Addition to this, deformation capacity of the frame cannot receive influence in composite action by concrete slab, but can be decided by the plastic deformation capacity of the hysteretic dampers at the bottom flange of beam-ends.

At the ultimate state, the hysteretic dampers had shown the limit of the plastic deformation capacity. That is to say, the dampers had absorbed 100% accumulation plastic strain and the maximum strain was 10%. Consequently, it was natural that the out-of-plane deformation occurred on the basis of the hysteretic damper of compression side.
5. Conclusions

Many steel structures suffered damage at the beam-end connections in the Northridge and Kobe Earthquakes. After the earthquakes, although a lot of researchers proposed the new connection details, most of them paid attention to repairing.

In this paper, the new ductile steel frames, which realize not only structural performances but also easy repairing after an earthquake, were proposed. By conducting the quasi-static tests on half scale frame with the hysteretic dampers at the bottom flange of beam-ends, the following results were obtained.

(1) The frame specimen showed the sufficient the plastic deformation capacity and stable behavior.
(2) Although the composite action by concrete slab was observed, the symmetrical change of deformation of the hysteretic dampers of both sides was observed.
(3) Consequently, plastic deformation capacity of the steel structure frame with the hysteretic dampers is decided by plastic deformation capacity of the hysteretic dampers.

In addition to the summary, it is easy to repair the damage parts in the proposed steel structure with the hysteretic dampers at the bottom flange, because the damage was limited to the hysteretic at the bottom flange until the story drift angle 1/50 radian.

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