NEW DUCTILE MOMENT-RESISTING CONNECTIONS LIMITING DAMAGE TO SPECIFIC ELEMENTS AT THE BOTTOM FLANGE

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ABSTRACT

After the 1995 Kobe earthquakes, a remarkable number of researches have been conducted on the beam-end connection. Those results have improved welding practices using higher material toughness and geometrical modification in the welded connections. Although the improved connections showed sufficient rotation capacity, little attention has been paid to repairing damage after an earthquake. At the stage of seismic design, it is important to consider restoring structures immediately. The purpose of this study is to propose new ductile MRF connections, which realize not only seismic performance but also easy repairing after an earthquake. In this paper, cyclic loading tests were conducted to evaluate composite effects on new connection detail, and to compare damage to concrete slab with conventional detail. The test results indicated that the plastic rotation capacity of the new connection was hardly affected by the presence of concrete slab, and the new connection was beneficial in reducing damage to concrete slab.

Introduction

In the Northridge and Kobe earthquakes, some buildings lost structural functions, although many buildings avoided collapse as to save human life. The loss caused the termination of social and industrial activities, and severe economic loss. At the stage of seismic design, it is important to consider restoring structures immediately after an earthquake.

Today, most of high-rise buildings are designed according to the Damage-Controlled-Structure (Wada et al., 1992). This system consists of a mainframe and dampers as shown in Fig.1. The main frame 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 designed based on the Damage-Controlled-Structure can be used continuously by repairing or exchanging dampers. However, in urban areas, most of steel structure buildings are low-rise or medium-rise without dampers. In the case of these buildings, naturally, energy absorption of an earthquake must be expected with plastic rotation of beam-ends. In the earthquakes mentioned above, many steel structure buildings suffered damage at the beam-end

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welded connections, and some brittle fractures of beam-bottom-flange occurred at the connections. After the earthquakes, a remarkable number of studies have been made on the beam-end connections in the U.S. and Japan. Those results improved welding practices using higher material toughness and geometrical modification of the weld access hole in Japan. And the reduced beam section (RBS) has been proposed in the U.S. However, the improved details and the proposed designs are based on plastic deformation capacity at the welded beam-end or near there. Consequently, those have not considered the repairing of damage parts. The purpose of this study is to propose new ductile steel beam-end connections, which realize not only structural performance (deformation capacity) but also easy repairing after an earthquake. Moreover the structural performance and the easy repairing were verified from quasi-static cyclic tests.
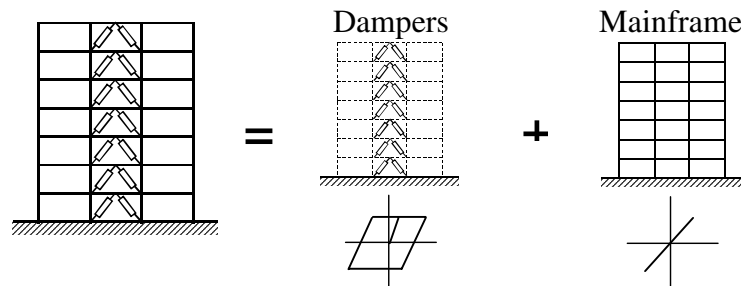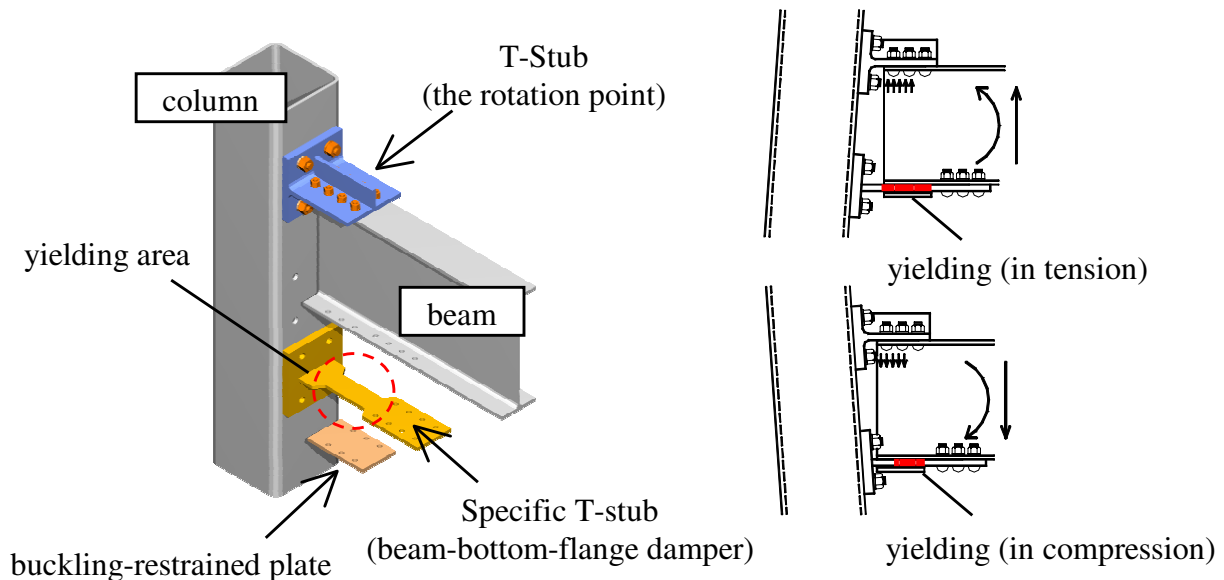
Fig.1 Damage-Controlled-Structure (Wada et al., 1992)

Fig.2 New Ductile MRF Connection (Damage-Controlled-MRF Connection)

New Ductile Moment-Resisting Beam-End Connection

It is important to realize structural performance and easy repairing, because conventional dampers, which are wall type, brace type, and shear link type, cannot be applied to low-rise and medium-rise steel structure buildings. Owners and designers like large windows (spaces).

There are two conditions that must be met to make structures easy repairable. One of them is to constitute structures using exchangeable members. The other is limiting damage to

some of these exchangeable elements as the Damage-Controlled-Structure mention above. New beam-end connection, which satisfies the two conditions, is shown in Fig.2. In the proposed connections, seismic damage (plastic deformation) is limited to specific T-stubs at the beam-bottom-flange of beam-ends. The specific T-stubs yield earlier than beams, columns, and T-stubs at the beam-top-flange. Consequently, plastic deformations are limited to the specific T-stubs at the beam-bottom-flange during an earthquake, and by repairing or exchanging the specific T-stubs, the steel structure buildings can be used continuously. Naturally, the specific T-stubs are not conventional because they have to show same stable behavior in tension and compression. In other words, they have to work as the hysteretic dampers (the beam-bottom-flange dampers). The specific T-stubs have buckling-restrained weak-section on the weak flat-plate in Fig.2, and detail of the buckling-restrained method will be described later.

Test Planning

Test Specimen and Test Set-up

A consist comparison was used for all specimen to evaluate composite effects. The tests were cantilever beam, cyclic-load tests with a stiff, strong column as seen in Fig.3. Two pairs of specimen were tested. Each pair consisted of a bare steel specimen and a similar specimen with a concrete slab attached. In the first pair, a conventional welded connection was applied. In the second pair, the proposed connection was investigated. Overall details are complied in Table 1.
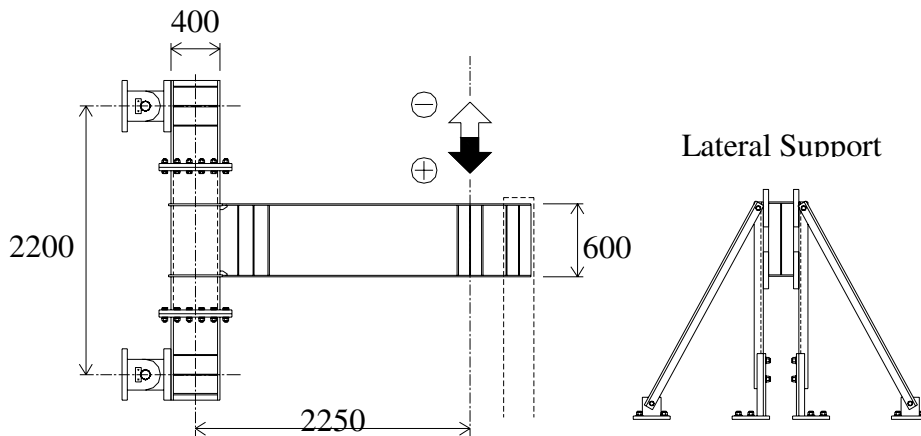

Fig.3 Test Specimen and Test Set-up (unit : mm)

Table 1 Specimen List

Specimen	Connection Detail	Composite or Bare Steel
W_B	conventional welding	bare steel
W_C	conventional welding	composite
T_B	with beam-bottom-flange damper	bare Steel
T_C	with beam-bottom-flange damper	composite

Beam, Column, and Connection

Beam with both connections was made of section (depth × flange width × web thickness

× flange thickness) of 600×200×9×12. Column with the beam-bottom-flange dampers was H-shaped, and made of section of 400×400×16×19. Steel grades JIS SS400 were chosen for flange and web of beam and column. Column with conventional welded connections was made of cold-formed square-tubes, and had section (width × width × thickness) of 400×400×19. Both columns were used to provide strong column, weak beam action and to provide for a strong panel zone. As shown in Table2, mechanical properties were obtained from tensile coupon tests according to JIS-1A.

Details of beam-end connection are shown in Fig.4. The conventional welded connection referred to as a "through-diaphragm connection" is used (Nakashima et al., 1998). A beam was shop-welded by complete joint penetration (CJP) groove welds. Welds in the conventional connection were made using the gas metal arc welding (GMAW) with CO_2 shielding was used to fabricate the welded joints of test specimens. A solid electrode designated as YGW-11 with a diameter of 1.2mm was used for all welding. Backing bars and weld tabs were left in place for the conventional connection.
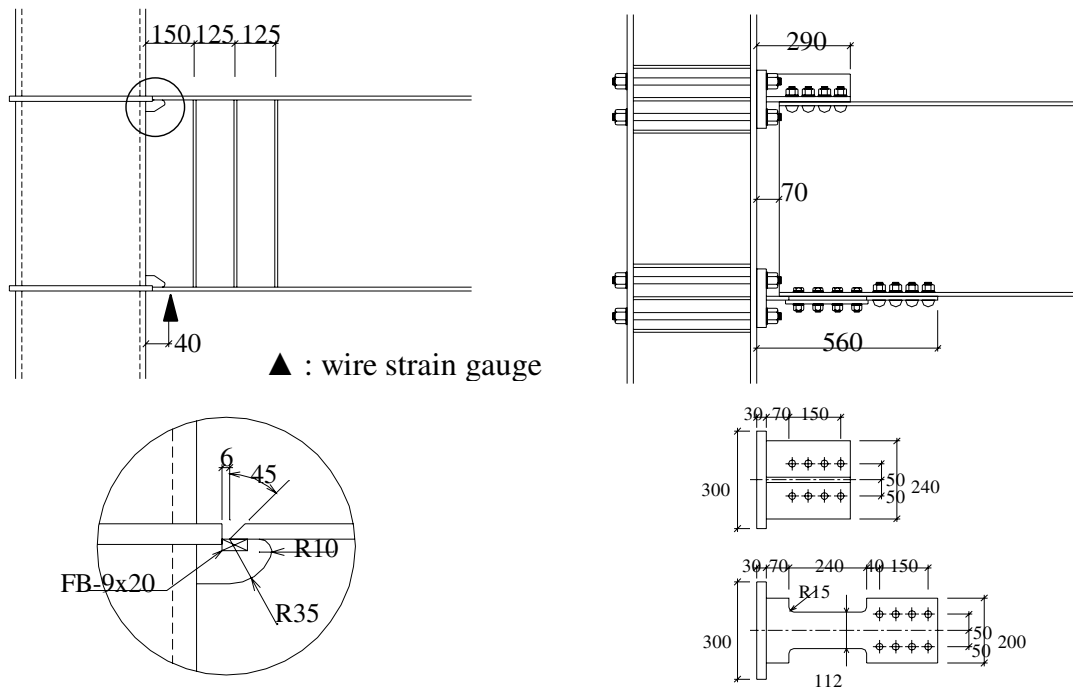
(a) Conventional Welded Connection   (b) New Connection
Fig.4 Details of Beam-End Connection (unit : mm)

Table 2 Mechanical Properties of Steel Plate

Sample plate & Thickness	Grade	Yield Strength [MPa]	Tensile Strength [MPa]	Elongation [%]
Beam-flange	SS400	279	429	31
Beam-web		296	435	33
Box Column	BCR295	388	439	27
Dampers	LYP225	244	298	39

In the new connection, the yield strength of the beam-bottom-flange damper was designed weaker than those of beam, column, and the T-stub at the beam-top-flange to keep the other member the elastic range. The steel material used for the beam-bottom-flange dampers was the low yield point (LYP) steel. Mechanical properties of the LYP steel are shown in Table2. Note that Young's modulus and Poisson's ratio of the LYP steel are identical to those of conventional structural steels.

Buckling-Restrained Method for the Beam-Bottom-Flange Damper
Detail of buckling-restrained method for the beam-bottom-flange dampers is shown in Fig.5. Yield area of the dampers was put between the beam-bottom-flange and the buckling-restrained plate. Unbounded materials for removing friction have to be stuck on the yield area of the dampers. The buckling-restrained plate and beam-bottom-flange were connected through rings by high strength bolts. A 2mm out-of-plane clearance was obtained by setting up the ring, because the yield area of the damper had a thickness of 12mm and the rings had a thickness of 14mm.
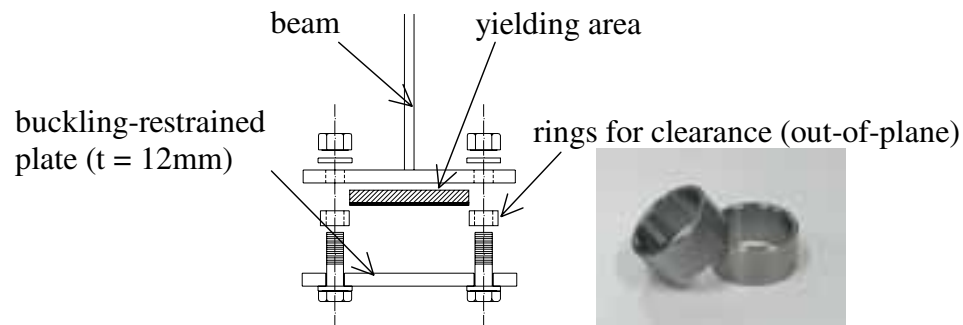

Fig.5 Detail of Buckling-Restrained for the Beam-Bottom-Flange Damper (cross section)

Concrete Slab
Shear studs, which had a diameter of 19mm and a height of 80mm, welded in 140mm pitches. Wire-meshes were placed 25mm and 115mm above the beam-top-flange. Concrete, which had a thickness of 130mm, was placed on the beam-top-flange without metal deck sheets. Material properties of concrete shown in Table3 were obtained from concrete cylinder tests on the same day of the cyclic loading tests.

Table 3 Mix Proportion and Compression Test (Concrete Slab)

W / C	Sand	weight per unit volume [kg/m^2]				
[%]	[%]	water	cement	fine	coarse	admixture
55.5	47.5	162	292	863	968	0.73

Compression Test (average)
Young's modulus : 2.5 [$\times 10^4$ MPa]
Compressive Strength : 16.5 [MPa]

Loading Program
Quasi-static loading was carried out following to the loading program shown in Fig.6. The loading program was based on the rotation angle of beam, which were 1/100, 1/50, 1/33,

and 1/25 radian. The cantilever beam had a length from the face of the column to the center of the load, L. The total tip deflection was due to elastic and plastic flexural deflection of the beam, δ. The rotation angle of beam can be found out by dividing the total tip deflection by L.
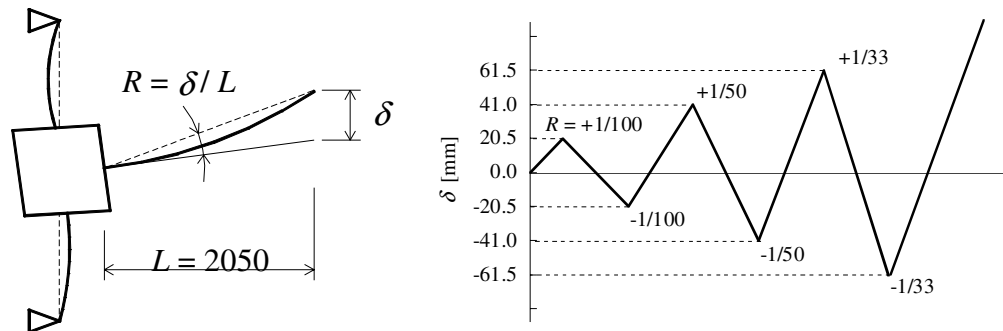
Fig.6 Loading Program

Test Results and Consideration

Specimen Performance

The shear force versus the rotation angle of the beam is plotted for all specimens in Fig.7. Although specimen W_B and W_C ultimately failed during the cycles of 0.03 radian amplitude, W_C failed earlier than W_B. Failure mode of W_B was lateral buckling of beam. On the other hand, that of W_C was fracture of beam-bottom-flange near the toe of the weld access hole. It meant that the sudden change in geometry at the toe of the weld access hole caused a stress (and strain) concentration. The reason why the failure mode changed from lateral buckling to beam-bottom-flange fracture was that concrete slab resisted compression and the tensile strain at the beam-bottom-flange increased.

Although cracks were observed at the yielding area of the beam-bottom-flange damper, specimen T_B and T_C did not ultimately fail during the cycles of 0.05 radian amplitude. Composite action led to a roughly 98% increase in initial stiffness and 20% increase in yield strength. Nevertheless, T_B results, especially plastic rotation capacity, were very similar to T_C.
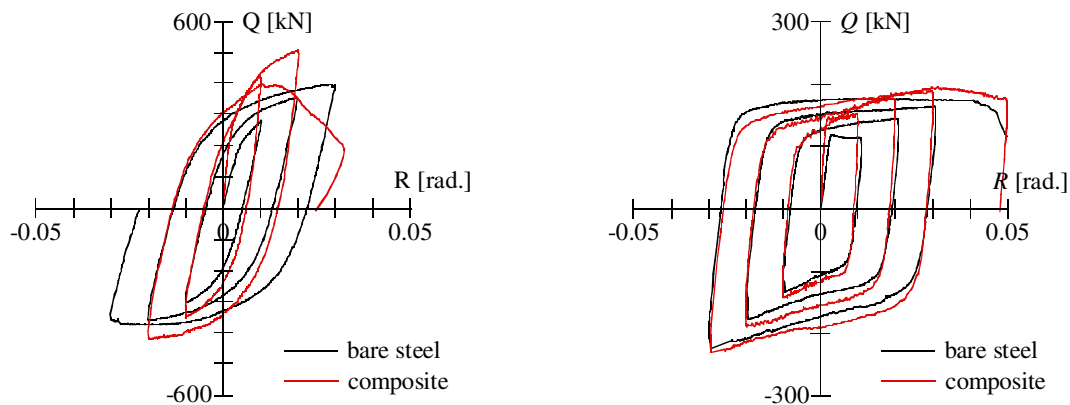
(a) Conventional Welded Connection   (b) New Connection
Fig. 7 Specimen Performance (Shear Force(: Q) versus Rotation Angle(: R))

Composite Effects (especially, an increase in strain of beam-bottom-flange)

      In the conventional welded connection (W_B and W_C), strain gauges were glued on the beam-bottom-flange as shown in Fig.4, in order to observe composite effects. In the new connection, an axial deformation of the beam-bottom-flange damper was measured by displacement transducers, and axial strain can be obtained by dividing the axial deformation by a length of the yielding area. Strain hysteresis of the beam-bottom-flange and the damper are shown in Fig.8. In the conventional welded connection, composite action led to a roughly 50% increase in the tensile strain by the presence of concrete slab. In other words, the plastic deformation capacity of the conventional connection was affected by the presence of concrete slab. To opposite side of that, the symmetrical strain hysteresis was observed in the beam-bottom-flange damper. Further, the plastic strain at the beam-bottom-flange damper was increase by only 10%, indicating that the new connection was hardly affected by the presence of concrete slab.
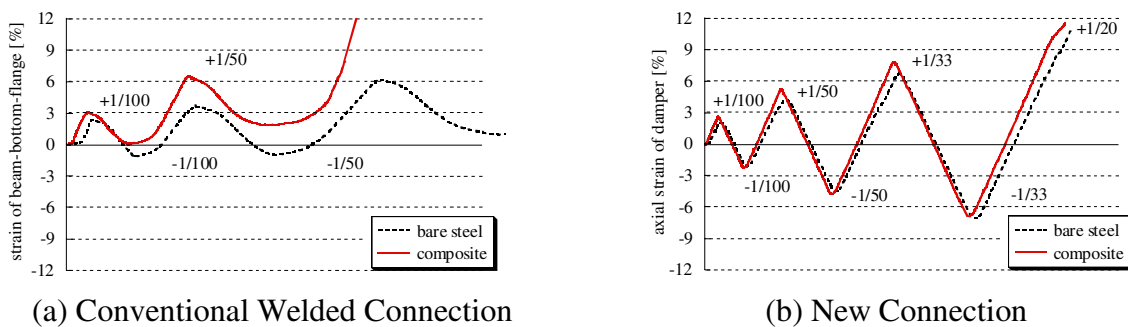
(a) Conventional Welded Connection   (b) New Connection

Fig. 8   Strain of Beam-Bottom-Flange and Damper

Damage to Concrete Slab (crack pattern and maximum crack width)

      Crack pattern and crack width were observed and written down, in order to grasp damage to concrete slab, when the rotation angle of the beam was reached to 0, –1/200, –1/100, and –1/50 radian. Crack pattern and crack width obtained from the tests are shown in Fig.9. In the conventional welded connection specimen, cracks occurred on the entire slab. To the contrary, in the new connection, cracks were concentrated near the face of column where the beam-bottom-flange damper attached. The reason why cracks were concentrated near there was that plastic deformation was limited to the beam-bottom-flange damper.

      When an attention is paid to the maximum crack width of both connections, it can be found out that the maximum crack width of the conventional connection specimen is gradually growing by subjected to cyclic loadings. To the contrary, the maximum crack width of the new connection specimen is hardly growing at the same rotation angle of beam (for example, 1/100 radian). Further, when the rotation angle of beam changes from 1/100 to 1/50 radian, the axial strain of the damper becomes twice. It indicates that there is a correlation between the maximum crack width and the axial strain of the damper (the axial deformation of the damper). Therefore, in this paper, the maximum crack width and the axial deformation of the damper are related by a calculation model shown in Fig.10. In Fig.9 (b), the red broken line is evaluation value, which is calculated by using the model and an experimental data of the axial deformation of the damper. The evaluation value of the maximum crack width is in good agreement with the experimental value.

Conclusions

This paper presents an experimental study on new MRF connections designed to restrict damage to specific connection elements (T-stubs) placed at the beam-bottom-flange. Especially, composite effects and damage to the concrete slab were addressed in these tests. Composite action led to a roughly 98% increase in initial elastic stiffness and 20% increase in yield strength. Meanwhile, the plastic strain at the specific connection elements was increased by only 10%, indicating that the plastic rotation capacity of the connections was hardly affected by the presence of the concrete slab. And the feature of the design forcing the connection to rotate about the beam-top-flange was beneficial in reducing damage to the concrete slab (cracks).
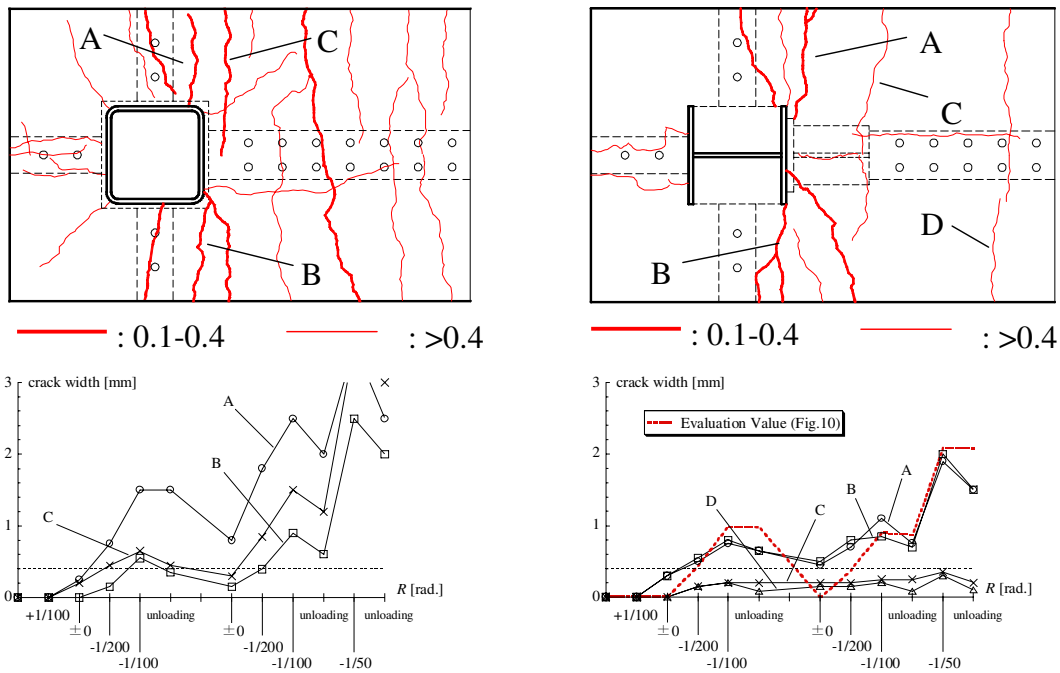
(a) Conventional Welded Connection   (b) New Connection

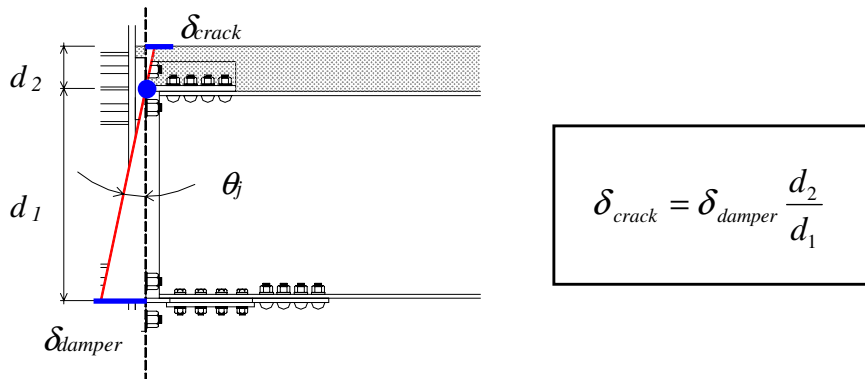Fig. 9 Crack Pattern and Maximum Crack Width

$$\delta_{crack} = \delta_{damper} \frac{d_2}{d_1}$$

Fig. 10 Calculation Model for Evaluating Maximum Crack Width of Concrete Slab

Acknowledgment

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