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Effective use of U-Link in Concrete Filled Steel Tubes Beams

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ABSTRACT: *Using a new system of U-links in concrete filled tube, beams to join the steel tube with the concrete core in the compression side of the beam resulted in large confinement of the concrete core. Confinement is achieved by employing the separation expected between the two materials. Two built- up concrete filled tubes were tested experimentally to study the effect of the use of U-link as additional reinforcement in beams subjected to flexural loads. The state of the concrete inside the tubes has shown no crushing of the concrete when those beams were cut open at the location of plastic hinge. Strain measurements revealed that the compressive strain in concrete was 5-6 times the concrete crushing strain. The experimental data showed that the use of U-link as additional reinforcement in the concrete filled tubes improved the beam flexural capacity, stiffness, and ductility.*

Keywords: *Concrete filled tubes, U-Link, confinement, steel beams*

I. INTRODUCTION

Nowadays, more studies are focused on the concrete-filled steel tubes (CFTs) as composite members. One can define a successful composite as a material with a unified structural behavior improved compared to each individual component alone. CFTs have proven superior properties compared to reinforced concrete or steel members (Hajjar, 2000). This can be justified by taking into consideration the confinement that the steel tube offers to the concrete, and the stability offered by the concrete during the buckling of the steel.

As this type of construction is most effective for axial applications, CFTs have been used increasingly as columns and beam–columns. Worldwide, their use has a wide range from compression members in low-rise, open-floor plan construction, using hot-rolled steel circular or rectangular tubes filled with precast or cast-in-place normal or high strength concrete, to large-diameter cast-in-place members used as lateral-resistance columns in braced and un-braced frames. Square or rectangular CFT box columns, fabricated from four welded steel plates, and circular CFT fabricated pipes have been used throughout the world (Roeder, 1998)

The use of mechanical shear connectors in order to improve the interaction between the steel and concrete in CFT beams has not been thoroughly studied. Due to the load limits of the testing apparatuses and the need to run the tests economically, reduced-scale models have been used to perform the experiments on the behavior of CFTs in flexure which do not reflect exactly the actual composite action between the steel tube and concrete core. While it is generally assumed and accepted that the walls of the steel tube provide confinement to the concrete and thus considering the addition of shear connectors are unnecessary (Lu and Kennedy 1994), Probst et al. (2010) showed that without using shear connectors the composite action became ineffective at ultimate level for both rectangular and circular CFT beams. As a result, the slip between the concrete and steel has to be considered in the flexural capacity of CFT beams.

High strength, ductility and stiffness can be obtained in the CFT system because of the properties mentioned above. However, the difficulty in compacting the concrete may create a weak point in the system where bleeding of the concrete may produce a gap between the concrete and the steel. To compensate this deficiency, high quality concrete with a low water-content and superplasticizers to improve the workability are used in construction (Morino and Tsuda, 2003). The placement of the concrete fill can be further enhanced by using self-consolidating concrete (SCC) as vibration is not required (Han et al, 2006).

One of the CFTs application is in the field of foundation engineering, common in large bridges, with piles or pile groups consisting of a large diameter tubes that are filled in-situ with concrete. The steel tube usually used just as a formwork for pouring concrete, and the concrete core reinforced with steel bars are expected to carry the axial loading at the serviceability stage . Considering the existing steel tube in the flexural loads expected during the construction stage would be advantageous. (Wheeler and Bridge, 2011)

Recently, using the CFT members as girders in bridge structures was developed and constructed in Japan. Composite bridge system with CFT girders is compatible with girder height restrictions, relatively easy to build, and resistant to seismic forces by their good durability and deformability. (Kang et al., 2007)

A wide range of design provisions for CFTs exist throughout the world, including the European Community, US, Canada, Japan, China, Australia, and many other countries. Many Authors have outlined the equations and compared the design provisions for both circular and square CFTs from several specifications and international codes, including the American Institute of Steel Construction (AISC), the American Concrete Institute (ACI), the British Standard (BS), Eurocode 4 (EC4), and the Architectural Institute of Japan (AIJ).

Roeder et al. (1999) studied the bond stress capacity of CFTs with dimensions and properties used in actual constructions in the United States. According to this study, the factors affecting bond transfer include the diameter of the steel tube, the shrinkage of the concrete core, and the imperfections of the steel internal walls. These tests also revealed that bond strength is larger in circular than rectangular CFTs, and that it increases with a decrease in the depth to thickness ratio.

Han (2004) defined the confinement factor (ζ) as an indicator of the composite action of the steel tubes and concrete in CFT beams and proposed the equation:

$$\zeta = \frac{A_s f_y}{A_c.(0.85 f_{c'})}$$

It was found that the higher the confinement factor, the higher the compressive strength of the confined concrete, and that the higher the confinement factor the more ductile is the confined concrete.

Elchalakani et al. (2001) presented an experimental investigation of the flexural behavior of circular CFT subjected to large deformation pure bending where the depth to thickness ratio anged from 12 to 110. A total of 12 specimens (1500mm long) were tested. The steel section used for the construction of the specimens were cold-formed circular hollow sections with nominal yield stress of 350 MPa and the average unconfined compressive strength of concrete cylinder was 23.4 MPa. For comparison, the experiment was also conducted on empty cold-formed circular hollow sections.

Marei (2007) conducted an experiment on six groups with a total of 27 CFT specimens using U-links and/or plated studs that aimed to join the steel shell with the concrete core. 19 square and rectangular CFT beams were tested under bending moment, and 8 circular and square columns were tested under axial compression loads. Rectangular, square and circular HSS were used for the CFT specimens. Two-point loading was used to provide a constant moment region for observations and measurements.

This present study aims to further enhance the bond between steel and concrete in CFT beams in addition to increase the confinement of the concrete core by using U-shaped links. It benefits from the difference in Poisson's ratio of concrete and steel. The beams used in this study were manufactured using L-shaped sections welded together for a more practical way of constructing the CFT specimens with this system of U-links and with an arrangement expected to be more efficient in confining the concrete core.

II. EXPERIMENTAL SET-UP

Two beams were prepared to be tested, one of them was confined for compression purposed, and the other was confined at the compression side using a new system of U-link. Plates were cut from mild steel, and folded to shape L-sections, then welded together to form hollow steel tube. The steel tubes were filled with concrete. The uniaxial compressive strength of concrete was determined using standard cubes tested at 28 days. The yield and ultimate strength of steel was determined using samples from the plates tested under standard tensile tests. Table (1) shows more details of the tested CFT. The thickness of the steel plates used is 6 mm, and the depth to thickness ratio is taken to be 50. The total length of the beam is 3 m, while the clear span length is measured to be 2.8 m.

Table 1: CFT beam specimens' properties

Beams Designation	**General Property**			**Concrete properties**		**U-links**		
	Outer Dimen.	**Area of steel**	**F_y**	**$f_{c'}$**	**Area of concrete**	**Diam.**	**Long. clear spacing**	**f_y**
	(mm)	**(mm^2)**	**MPa**	**MPa**	**(mm^2)**	**(mm)**	**(mm)**	**MPa**
B1(t6)	**150x300**	**5256**	**336.75**	**30.8**	**39744**	**_**	**_**	**_**
B2(t6-UΦ10)	**150x300**	**6084**	**336.75**	**27.6**	**38088**	**10**	**50**	**296.8**

The beams were prepared from 2 m x 3 m mild steel plate, hot rolled, [ASTM A36] with a 6 mm nominal thickness, and a specified yield stress 248 MPa and a modulus of elasticity of 200000 MPa was cut and folded to form the L-shaped sections used to fabricate the two beams B1 (t6) and B2 (t6-UΦ10). The material tensile test using steel samples from the plate measured the average of the actual yield stress to be 336.7 MPa. B1 (t6), is a beam without U-links and B2 (t6-U Φ10), is a beam that has U-links used to join the steel tube with the concrete core at the compression side of the section to improve the confining behavior of CFT beam specimens. Figure (1) and Figure (2) show the beam B1 (t6) and its cross section.

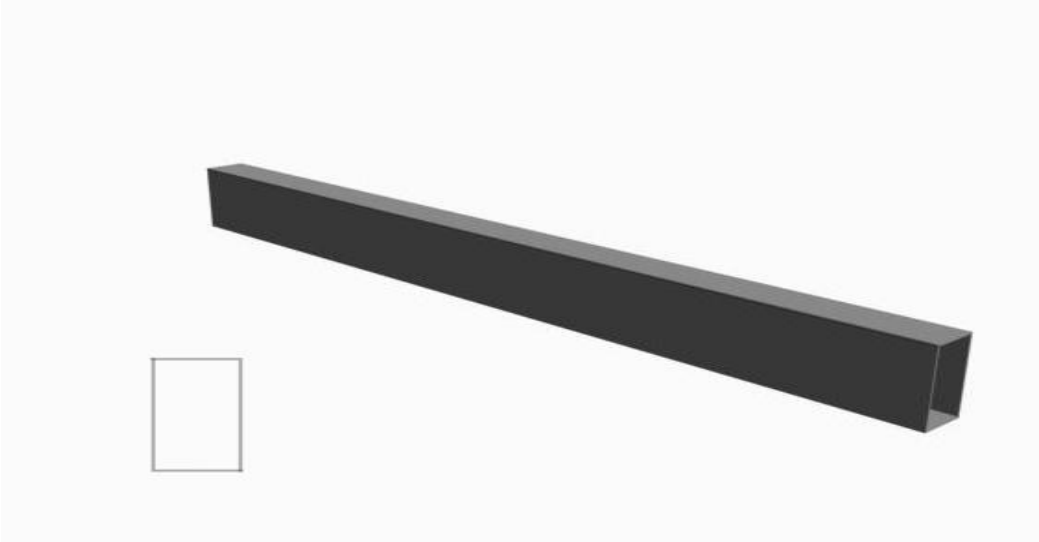

Figure 1: B1 (t6) and its cross section

Figure 2: the beam B1(t6) is ready for concrete pouring.

The second beam, B2(t6-Uφ10), consisted of three L-shaped steel sections, having the following cross sectional dimensions: 100×144×6 mm, 150×200×6 mm and 150×300×6 mm, all with a length of 3 m. Four lines of groove welding are used to form the rectangular steel section with a horizontal plate at the expected neutral axis level. Figure (3) and Figure (4) show the process of building the beam B2 (t6, U10).

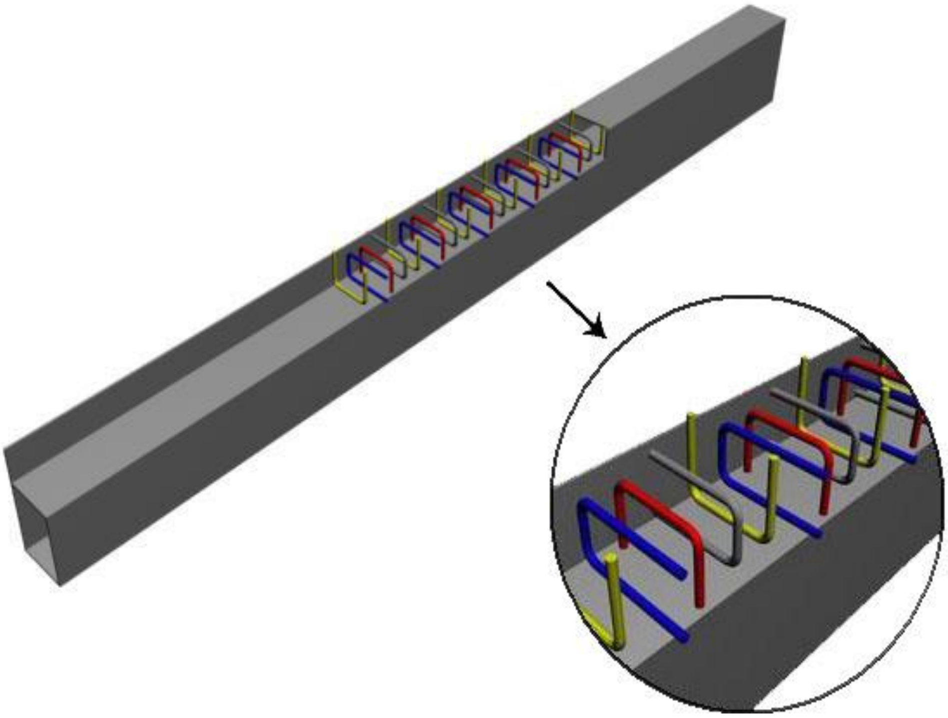

Figure 3: The arrangement of the U-links in B2 (t6, U10).

Figure 4: Beam B2 (t6, U10) ready to be assembled.

The test started with the placement of the rectangular CFT beam specimen (loaded about its major axis) on two reinforced concrete blocks. The specimens were placed on a pinned support on one end and a roller support on the other. These supports were located 100 mm from the end of the CFT beam, resulting in a center-to-center beam span length of 2.8 m. As the beam deflects under the load the rollers allow the beam to move freely, the simple support ends allow rotation and longitudinal translation without resistant. The load is applied at the mid-span of the beam. To transfer loading from a hydraulic ram, a plate of 3 mm thick was used underneath, creating a single point loading system, as can be seen in Figure (5).

Figure 5: Test set up

III. PLASTIC STRESS DISTRIBUTION FOR THE SECTION WITH HORIZONTAL PLATE

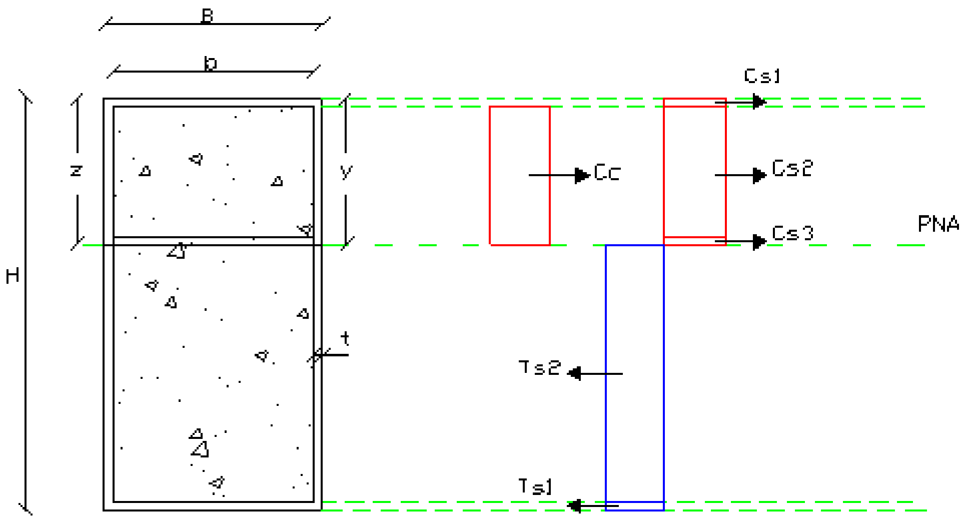

Figure 6: Stress distribution for the cross section of the CFT beam specimen with horizontal plate.

To calculate the plastic neutral axis we use the equilibrium of resultant compressive and tensile stresses:

$$C_{s1} = T_{s1}$$

$$C_c + C_{s1} + C_{s2} + C_{s3} = T_{s2} + T_{s1}$$

Then,

$$0.85bf_{c'}(y-t) + 2tF_y(y-t) + F_ybt = 2tF_y(H-y-t)$$

$$PNA = y = \frac{2tF_yH + 2(0.85)btf_{c'} + F_ybt}{0.85f_{c'}b + 4tF_y}$$

Where,

C_{s3}: resultant compressive stresses on the steel wall (kN)

To calculate the nominal flexural strength, moment of the resultant compressive and tensile forces about the plastic neutral axis (PNA) were determined as:

$$M_p = C_{s2}\frac{(y-t)}{2} + C_c\frac{(y-t)}{2} + T_{s1}(H-t) + T_{s2}\frac{(H-y-t)}{2} + C_{s3}(z-y)$$

$$= 2tF_y\frac{(y-t)^2}{2} + 0.85f_{c'}b\frac{(y-t)^2}{2} + BtF_y(H-t) + 2tF_y\frac{(H-y-t)^2}{2} + F_ybt(z-y)$$

Where,

z: distance from the wall inside the cross section to the top flange (mm)

IV. BEAM EXPERIMENTAL RESULTS

Figure (7) shows the relationship between the loads versus deflection at the mid-span of the CFT beam specimens. This is an important measurement to illustrate the behavior of a beam. These curves show the maximum load carried by the beam and the maximum measured deflection.

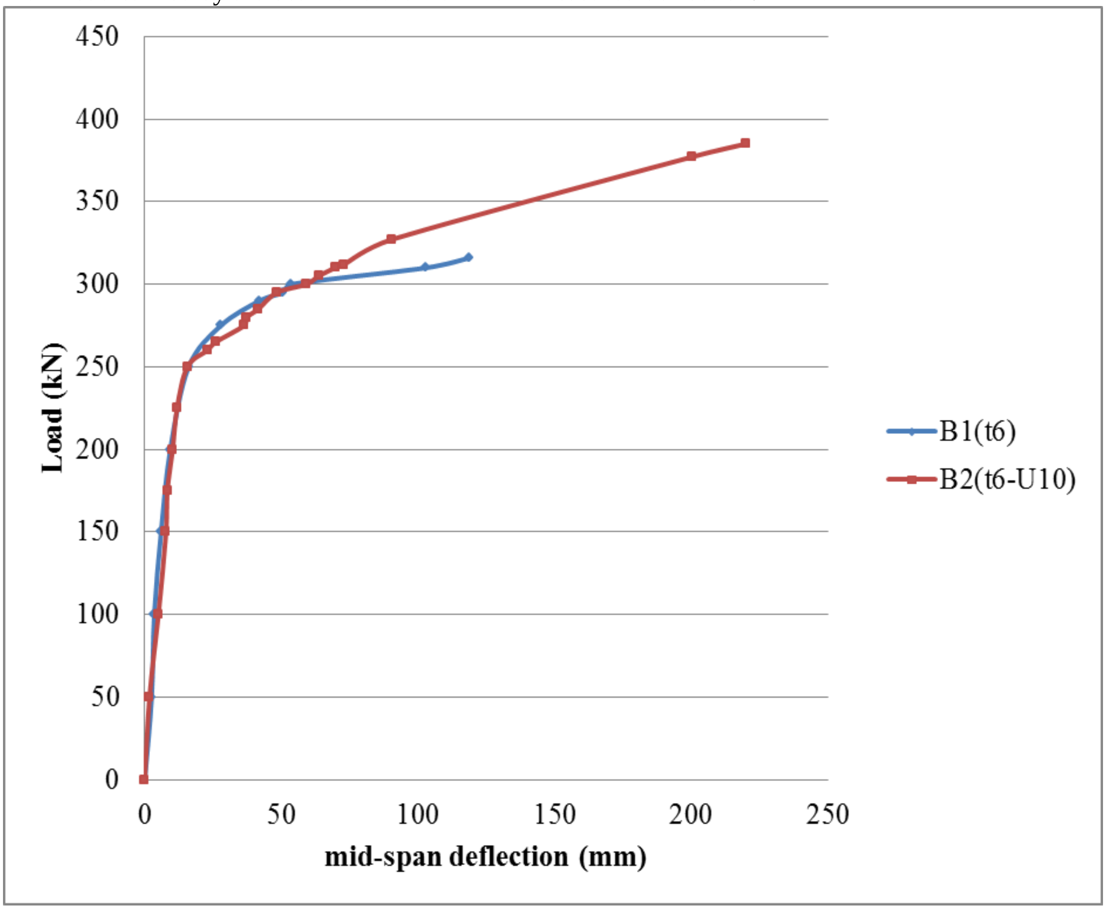

Figure 7: Load versus mid-span deflection for beam specimens B1 and B2

Figures (8) and **(9)** show the relationship for moment versus extreme strain at the mid-span for beam specimens. These curves show the maximum strains at the extreme top and bottom fibers in the mid-span cross section.

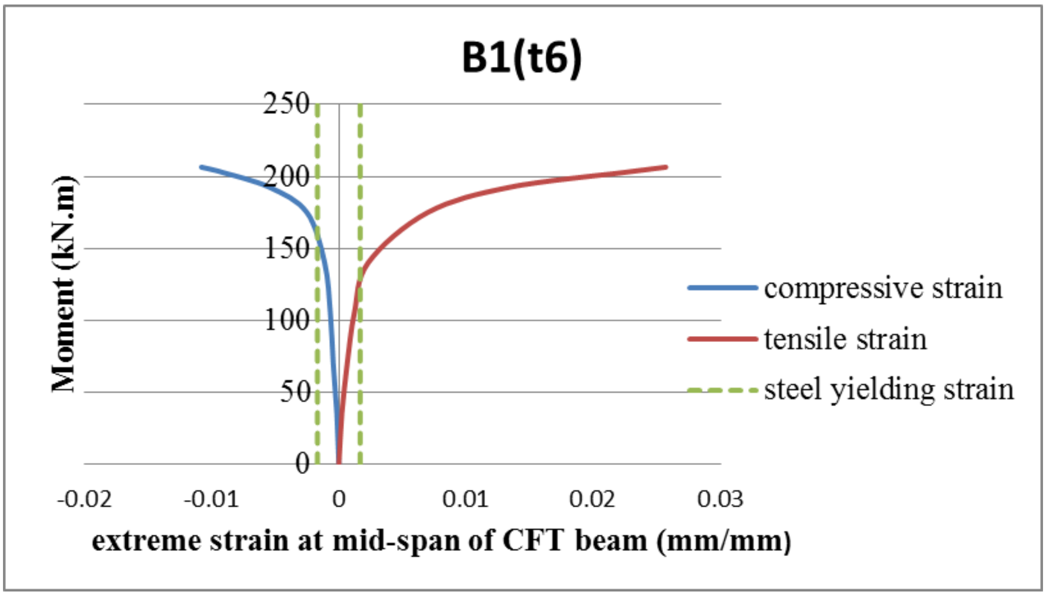

Figure 8: Moment versus extreme strains at mid-span of beam specimen B1

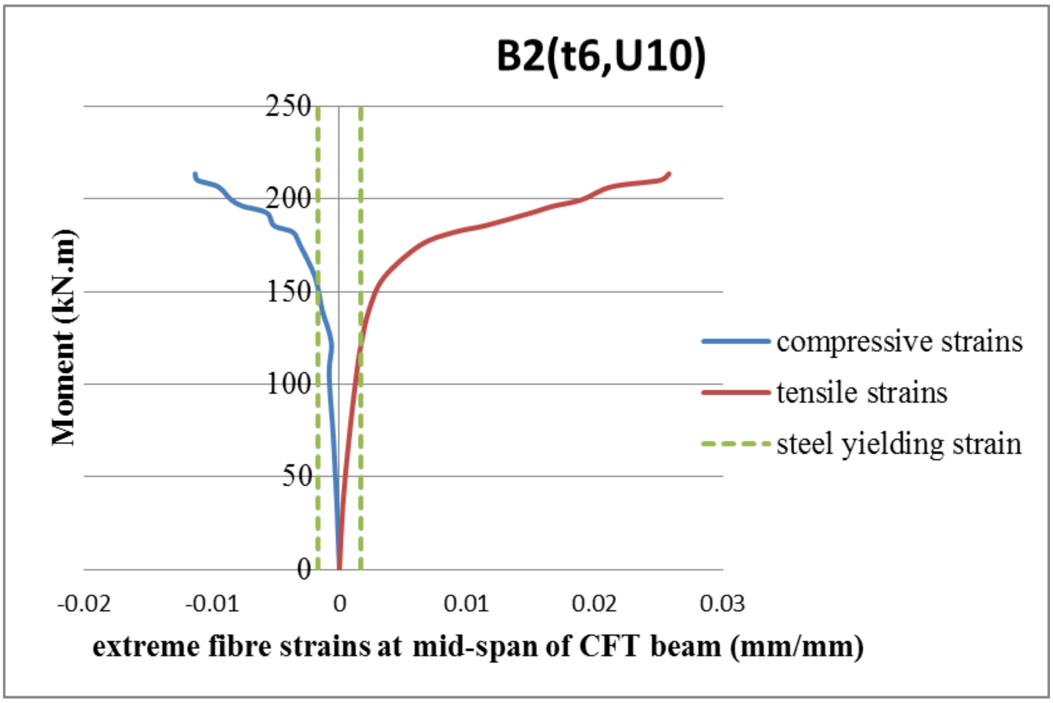

Figure 9: Moment versus extreme strains at mid-span of beam specimen B2

V. CONCLUSION

When analyzing the flexural capacity, the presence of the U-links modified the flexural behavior of the composite member, not only because they contribute to the compressive strength of concrete by confining it, but also because they delay, and even prevent, the local buckling of the steel in compression by supporting it laterally along with the concrete core. In general, in beams without U-links, failure occurred when an upward buckle of the top flange and side walls of the steel cross section developed in the mid-span of the beam specimens. The failure mechanism of beams with U-link shows two bulges on the top flange of the steel cross section beside the applied load.

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