EXPERIMENTS OF GIRDER JOINT IN TOKYO INTERNATIONAL AIRPORT (HANEDA) GSE BRIDGE USING UFC

Hiroyuki MUSHA
Technical Manager, Technology Center, Taisei Corporation, Tokyo, Japan

Norio WATANABE
Manager, Design Dept. Civil Engineering Div., Taisei Corporation, Tokyo, Japan

Tetsu FUKUHARA
Manager, Tokyo Branch, Taisei Corporation, Tokyo, Japan

Kenichi HARAGUCHI
Manager, Tokyo Branch, Taisei Corporation, Tokyo, Japan

Yasuo TAKEDA
Senior Officer for Construction Management, Tokyo International Airport office, MLIT, Toyko, Japan

Fumihiko MATSUKAWA
Deputy General Manager, Tokyo International Airport office, MLIT, Toyko, Japan

ABSTRACT:
GSE Bridge was constructed at Tokyo International Airport (Haneda), and Ultra-high strength Fiber reinforcement Concrete (hereafter, UFC) has been applied to the girders of this bridge. This bridge has a span of 46m, width of 16.2m, and is the largest bridge using UFC in the world. By using UFC, the reductions of girder height and of the self-weight became possible. Since GSE Bridge is the largest UFC bridge, it was necessary to verify its load performance. Several loading tests on the structures of joint were conducted. In this report, the outline of GSE Bridge is introduced, and the loading tests are described.

Keywords: UFC, ultra-high strength, fiber reinforced concrete, road bridge, loading test, joint, PBL

1. INTRODUCTION

In the Apron Construction Project at Tokyo International Airport, a single-span concrete bridge (hereinafter, GSE Bridge) was constructed over the road connecting the south and north aprons (Figure 1). This GSE Bridge is a span of 46 m, width of 15.2 m (Figure 2 and 3), and is the road bridge for Ground Support Equipments. The main load of this bridge is the large-scale heavy equipment of 50 tons in weight that is called "Towing tractor" for pulling the aircraft. Therefore, it was necessary to verify that the GSE Bridge, which will be constructed by the precast block erection method, has an enough load performance.

Figure 1 GSE Bridge (CG)

Side View

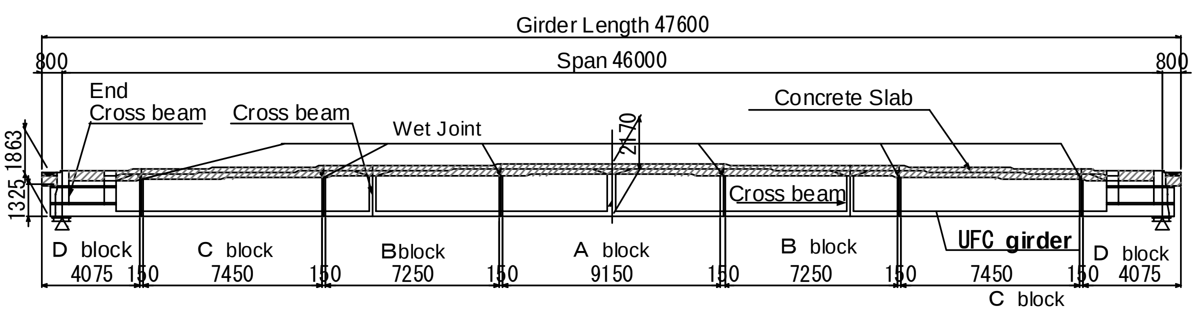


Figure 2 Outline of GSE Bridge structure

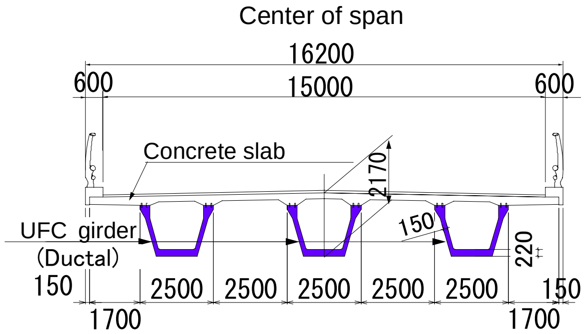


Figure 3 Cross section of GSE Bridge

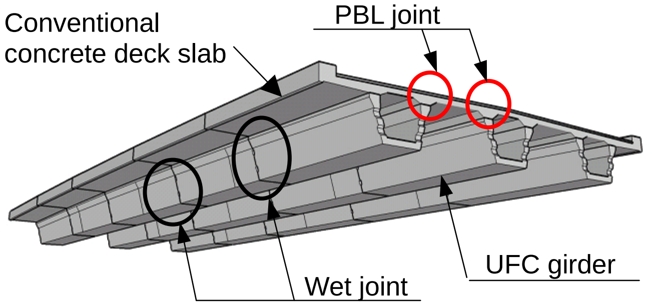


Figure 4 Outline of joint position

Additionally, according to the construction condition, it is requested that the girder height is held low and that the superstructure is light. To satisfy these demand performances, Ultra-high strength Fiber reinforcement Concrete (hereafter, UFC) was adopted to the girders of this bridge.

This UFC has average compressive strength of 200N/mm^2, and has a characteristic compressive strength of 180 N/mm^2, which allows structural members to be designed taking into account the tensile strength of the concrete. UFC also has high ductility, provided by the reinforcing effects of steel fibers. Table 1 shows the mix proportion of UFC. UFC contains high strength steel fibers (2% by volume, with a tensile strength of not less than 2.0 x 10^3 N/mm^2, 0.2mm in diameter and 15mm long). The cross-linked steel fibers help control cracking, which makes UFC highly ductile. As a general rule, UFC structures do not require rebar.

Table 1 Mix proportion of UFC*

*Standard UFC Material described in UFC Guideline

	Pre-mixed UFC	fibers	super-plasticizer	water
Unit Quantity kg/m^3	2,254	157	28 (liquid)	162
			Total Water : 180	

UFC has extremely high durability, ensured by the closely packed microstructure of the matrix with a water-to-binder ratio of W/B=0.14 that lowers the water content per unit volume of the UFC to the hydration limit and minimizes the voids in hardened concrete
.
UFC application to many kinds of structure has been increasing in recent years in Japan. UFC application in the PC bridge field is advanced because of taking advantage of thin members and weight reduction achieved by its ultra-high strength and high durability.

By adopting UFC to GSE Bridge, low girder height, which is 1.86m(girder-height span ratio: 1/25), became possible. Moreover, 40% weight reduction became possible by making structural members thin such as 15cm thickness of the web compared with conventional concrete bridges. Ahead of the adoption of UFC, the element experiment and the beam experiment of the girder joints were conducted, and it was verified that there was an enough load performance to the heavy

Photo 1 UFC precast block

load. In this report, it introduces the outline of the GSE Bridge, and the performance confirmation experiment executed as a verification of the girder joints design concerned is described.

2. OUTLINE OF GSE BRIDGE STRUCTURE

The bridge structure is a single span pre-tensioned composite girder 48 m long (Figure 2). The girder type is determined in 3 box girders because of wide bridge width 16.2 m (Figure 3). The main deck is composite structure between UFC U-shaped girders (f'_{ck}= 180N/mm^2) and conventional concrete deck slab (f'_{ck}= 40 N/mm^2) (Figure 4). Since UFC does not require any reinforcing bars, the web thickness utilizing its ultra high strength is only 150 mm, achieving a very slender form compared to the conventional concrete. UFC girders require heat curing and shop fabrication to ensure quality, therefore, those are inevitably precast structure (Photo2). Taking into account the capacity of the lifting equipment at the shop, it was decided that each segment of the precast girder should weigh less than 25 tons. This meant dividing the 47.6 m long main girder into seven blocks.

After the UFC girders were erected on the supports at the erection sites, the joint between the girders was filled with cast-in-situ UFC to create a wet joint, and prestresses were introduced to integrate the bridge body. The top slab and the U-shaped girder needed to be connected at the erection site. Perfobond Strip shear connector (hereafter, PBL) was used to connect the top slab and the U-shaped girder. This connecting method is originally used to connect a steel girder with a

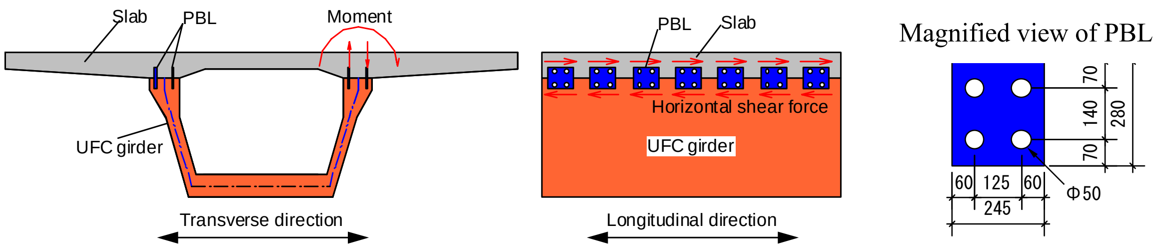


Figure 5 Outline of PBL joint structure

concrete slab, and has been applied to a UFC slab. This PBL connection has been also applied to other UFC bridges such as footbridges [2], [4], road bridge [5], and monorail girder [6].

3. LOADING TEST OF SLAB-GIRDER JOINT (PBL JOINT)

3.1 PBL Joint

Outline of PBL joint adopted for the joint between the UFC girder and the PC slab is shown in Figure 5. Because of the long overhanging slab, the PBL joints of this bridge should also be resistant to pulling forces in the vertical direction caused by the moment in the transverse direction. For this reason, PBLs were arranged in two rows. Half of the steel plate of each PBL was embedded in the slab and the other half in the UFC girder. Rebar was arranged in the holes of the PBL in the concrete slab, but not in the holes of the PBL in the UFC girder.

3.2 Element test of PBL joint

3.2.1 Details of the test

This test was performed to verify that the PBL joint on the UFC side has higher resistance to pulling forces than are assumed in the design and that the PBL joint does not fail as a result of a brittle fracture. The design performances required of the joint between the UFC girder and PC slab are (1) the joint exhibits elastic behaviors in the serviceability limit state and (2) the joint does not fail in the ultimate limit state. The shape of the test specimen is shown in Figure 6. The specimen comprised the joint and surrounding parts of the bridge. Three full-scale test specimens were built to evaluate variations in the test results.

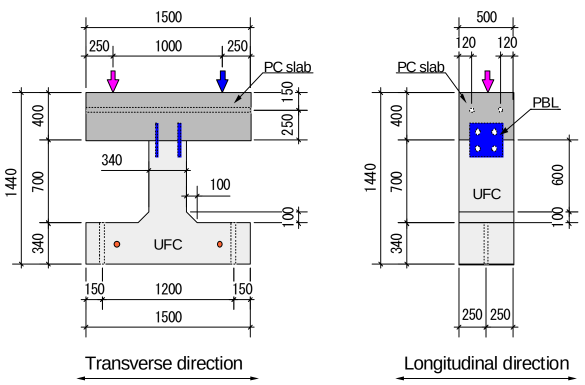


Figure 6 Test specimen

3.2.2 Loading method

As shown in Figure 7, steel frames were installed around the test specimen for loading purposes. Hydraulic jacks placed under the frame beams applied a downward load on both ends of the PC slab test specimen. The load intensities and the steps of the procedure were as follows: first, the design load in the serviceability limit state was applied three times in the alternate cyclic loading mode, the design load in the ultimate limit state was then applied twice, and finally the test specimen was loaded until it fractured. Figure 8 shows loading steps of the test.

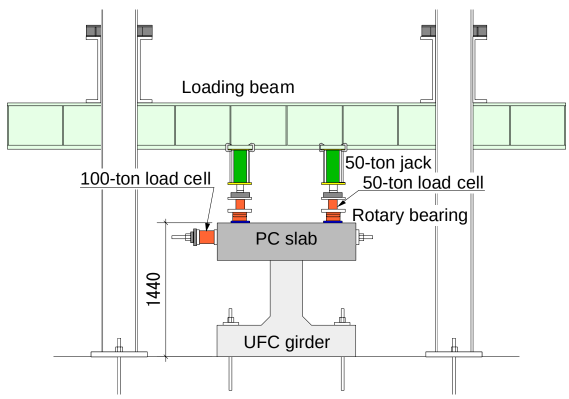


Figure 7 Loading test equipment

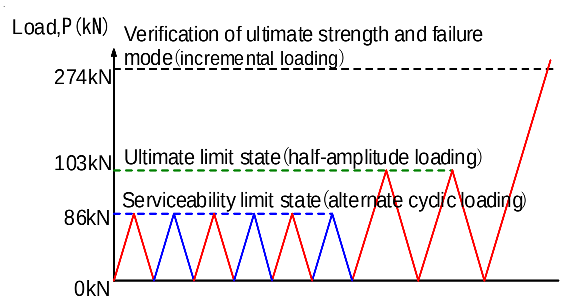


Figure 8 Loading steps

3.2.3 Test results

Figure 9 shows the load vs. displacement curve. The displacement was measured at the end of the slab. The test specimen did not crack under the design load (in the serviceability limit state) of 86 kN. The test specimen exhibited elastic behaviors with little variation after the design load in the serviceability limit state was applied three times in the alternate cyclic loading mode. Subsequently, the test specimen did not fail under the design load (in the ultimate limit state) of 103 kN. The slab was slightly detached from the web under a load of about 120 kN and the test specimen inclined gradually. Under a load of 250 kN, or more than twice the design load (in the ultimate limit state), cracks of 0.06 mm width developed, the rigidity of the test specimen dropped, and the load reached its peak.

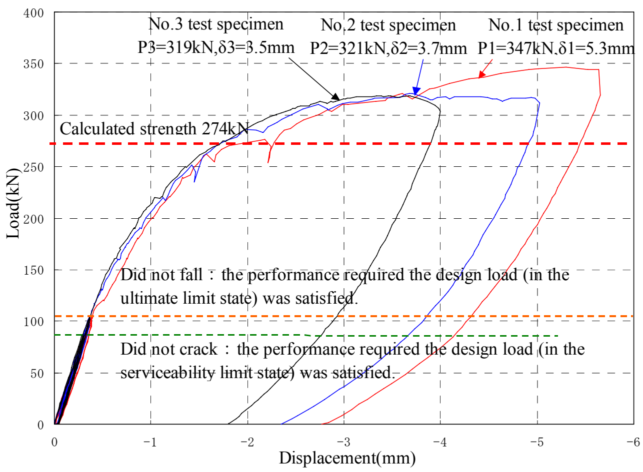


Figure 9　Load vs. displacement curve

Figure 10 shows cracks on the test specimen after completion of the test. As can be seen from the figure, the cracks developed diagonally from the center of the holes for the PBL. The width of the cracks is, however, under 0.08mm at maximum load, it remains small due to the bridging effect of the steel fibers.

This test verified that the PBL joint had higher resistance to pulling forces than calculated in the design and did not fail as a result of a brittle fracture under loads exceeding the peak load.

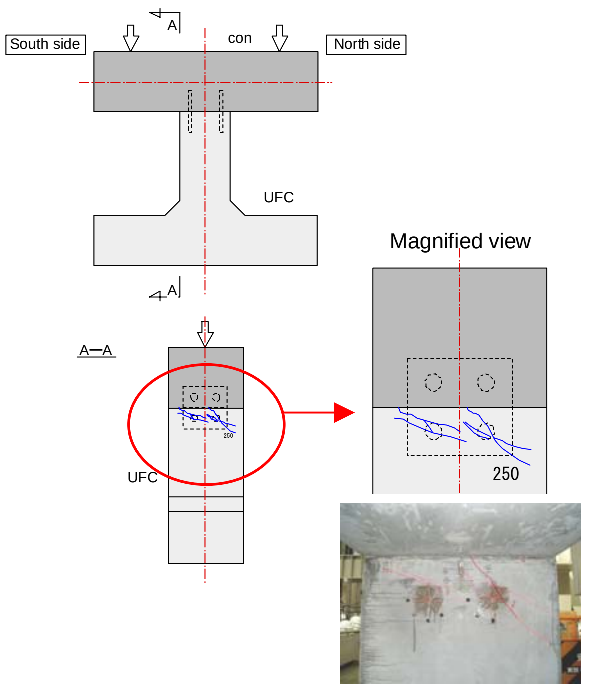


Photo Test specimen after the test

Figure 10 Cracks in the test specimen after the test

4. TEST OF GIRDER JOINT (UFC WET JOINT)

4.1 UFC wet joint

In a construction method unique to UFC bridges, a hollow (dent) is created in each joint surface of the precast girders. This hollow is then filled with cast –in-situ UFC to join the girders. It is called “UFC wet joint,” and Figure 11 shows outline of that. This method was developed for the construction of the Sakata Mirai Bridge and the shear transmission performance was verified experimentally. The results of this experiment are described in “Shear Transmission Capacity of Joints” in the reference material for the Guidelines for the UFC.”[1] This UFC wet joint was applied to other bridges [2], [4], [6].

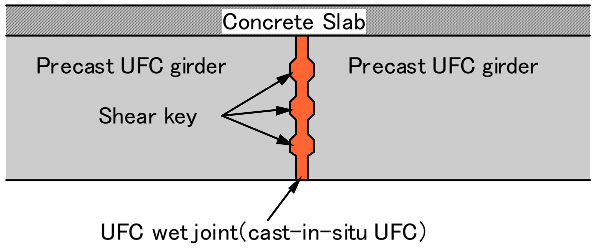


Figure 11　Outline of UFC wet joint

4.1.1 Background of UFC wet joint development

The most common method of jointing the girders erected by the precast segment method is to use match-cast joints to ensure the accuracy of girder end faces. An epoxy adhesive is in most cases used for the girder end faces, not only to keep the segment joints watertight but also to reduce the stress concentration on the end faces by adjusting uneven end faces. For this reason, adhesives having compressive strength comparable to or higher than the girders are often used.

In contrast, the UFC contains large quantities of reactive powders, such as cement, and the autogeneous shrinkage of the UFC in manufacturing girders is large, about 800 μ, and therefore it is difficult to ensure the accuracy of girder end faces of UFC bridges. Further, adhesives having a very high compressive strength of the UFC, or 180 N/mm^2, were unavailable and there was a concern about the stress concentration on the end faces of the UFC bridge girders where high compressive stresses would be induced in the axial direction. For this reason, the UFC wet joint was developed as an alternative method of jointing the UFC bridge girders to the conventional one.

Incidentally, with the development of UFC members manufacturing methods, it is now possible to manufacture the members with the accuracy of girder end faces ensured. The applications of adhesives to the UFC girders have been found in the cases where the cross section is not large or the compressive stress in the axial direction is not high, such as the footbridge [4] and monorail girder [6].

4.1.2 Features of UFC wet joint

Figure 12 shows the dimensional drawing of the UFC wet joint for the GSE Bridge.

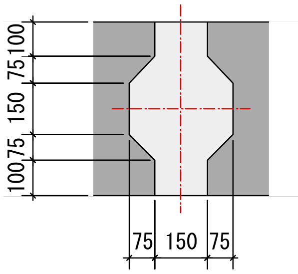


Figure 12 Detail of shear key (Unit: mm)

Because the UFC is cast-in-situ in the joint between precast girders that are placed apart to some extent, the girder ends facing each other do not require accuracy. That is, measures to keep end faces accurate, such as match-cast joints, are not required in manufacturing the UFC girders. This eliminates the need of manufacturing the girders in the order of erection and allows the girders to be manufactured in no particular sequence, improving production efficiency. Further, the high-flowability UFC, which does not contain coarse aggregate, has superior filling property and the stress concentration on the joints is considered very small. In addition, the wet joint makes it easy to adjust the bridge alignment. The UFC wet joint has these advantages, whereas it has disadvantages, such as the need to cure the UFC cast in the wet joint on site and it takes a long time to cure the UFC in the winter, resulting in increased work volume at the erection site.

4.1.3 Design of UFC wet joint

The strength of the UFC at the wet joint was calculated assuming that the safety factor of the shear capacity was not less than the flexural capacity. This assured the required strength when prestressing and prevented brittle failure in the completed structural system. The specified design strength of the UFC at the wet joint was calculated as 120 N/mm^2. Taking into account the constructability of the joint, where the inner cable sheaths needed to be joined between the precast girders, the width of the wet joint was set to 15 cm.

The method of checking the wet joint structure is specified in Reference 8 "Example Design of Structures Using the UFC" of the Guidelines for the UFC [1]. The equation for calculating the design shear transfer capacity of a block joint, V_{yd}, is given below.

$$V_{yd} = V_{cwd} + V_{ped} \qquad (1)$$

where:

V_{cwd} ; design shear transfer capacity

$$V_{cwd} = (t_c \cdot A_{cc} + V_k) / \gamma_b$$

$$t_c = \mu \cdot f'^{\beta}_{cd} \cdot \sigma_{nd}^{1-\beta}$$

$$\sigma_{nd} = -(1/2) P'_d / A_{cc}$$

σ_{nd} ;average compressive stress acting perpendicular to the shear plane

A_{cc} ; area of the shear plane on the compression side

B ; factor representing the shape of the plane (0.4)

μ ; average coefficient of friction due to contact with the solid body (0.45)

V_k ; shear capacity of the shear key

: $V_k = 0.1 \cdot A_k \cdot f'_{cd}$

A_k ; cross-sectional area of the shear key on the shear plane on the compression side

f'cd ; design compressive strength of concrete

V_{ped} ; component of the effective tensile force of the axial tendon parallel to the shear force

In calculating the shear transfer capacity to be borne by friction ($\tau_c A_{cc}$), in this equation, the factor representing the shape of the plane, β, needs to be selected appropriately depending on the state of the shear plane. Particularly for the UFC bridge where the average compressive stress acting perpendicular to the shear plane (σ_{nd}) is very high at 10-30 N/mm^2 because the members are very thick, the shear transfer capacity to be borne by friction is large. For this reason, the selection of the value of β becomes very important in calculating the shear transfer capacity.

In the example design described in Reference 8 of the Guidelines for the UFC, the factor representing the shape of the plane, β, is set to 0.4 based on the results of the element test (Reference 5) [1] that was conducted using the model of Sakata Mirai Bridge [2],[3]. However, the Sakata Mirai Bridge is a footbridge and differs much in the loading level from the GSE Bridge. Therefore, it was necessary to verify whether it was appropriate to set the factor β to 0.4.

4.2 Element test of UFC wet joint

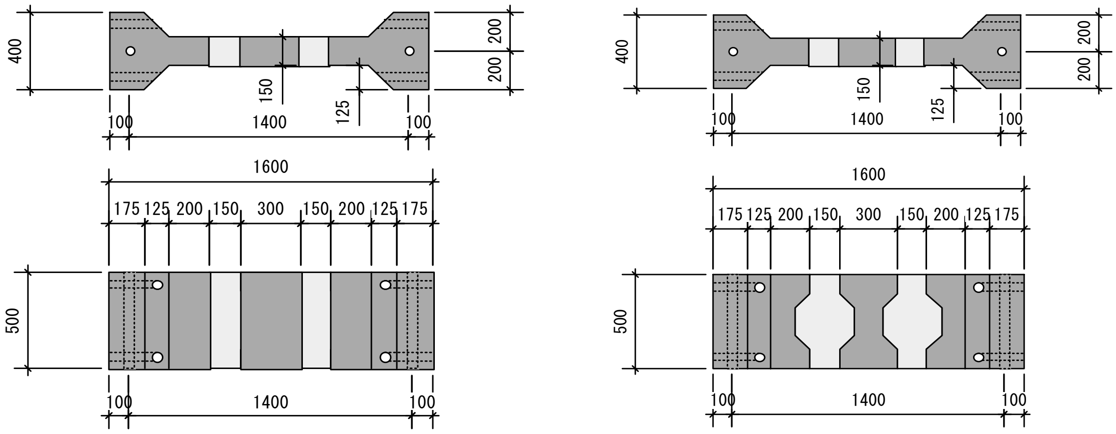
4.2.1 Details of the test

This test measured the shear resistance of the shear key and verified that the wet joint has higher shear resistance than was assumed in the design.

The shape of the test specimen is shown in Figure 13. The specimen comprised the wet joint and surrounding parts of the bridge. Two types of test specimen were prepared: one without a shear key and one with a shear key (Types 1and 2, respectively). The validity of the equation for calculating the design shear transfer capacity (Equation 1) was first verified using the Type 1 test specimen. The strength of the shear key was verified by comparing the shear strength of the Type 1 test specimen with that of the Type 2 test specimen. Three full-scale test specimens were prepared to evaluate variations in the test results.

4.2.2 Loading method

As shown in Figure 14, the wet joint was placed on blocks and supported at both ends. A load was then applied downward from the center block onto the test



(1) Type 1: without shear key (2) Type 2: with shear key

Figure 13 Test specimens

specimen. An average design compressive stress of 10 N/mm^2 acting on the wet joint was recreated in the test. The load was applied in increments using the 10 MN loading test equipment. Loading was stopped at certain times to check for cracking.

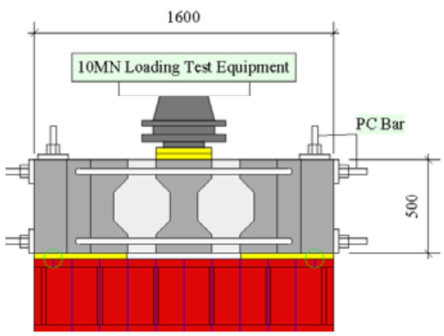


Figure 14 Loading test equipment

Photo 2 Loading test of UFC wet joint

Table 2 UFC strength at loading test

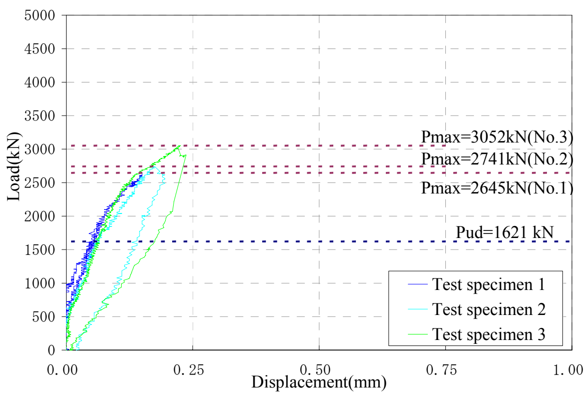
Specimen	Part	Compressive strength N/mm^2	First cracking strength N/mm^2	Young's modulus 10^4kN/mm^2
Type 1 Without shear key	Girder	207	10.0	5.3
	WJ^{*1}	145	5.2^{*2}	5.1
Type 2 With shear key	Girder	200	9.2	5.3
	WJ^{*1}	141	7.1^{*2}	5.0

＊1 : UFC wet joint
＊2 : after heat curing

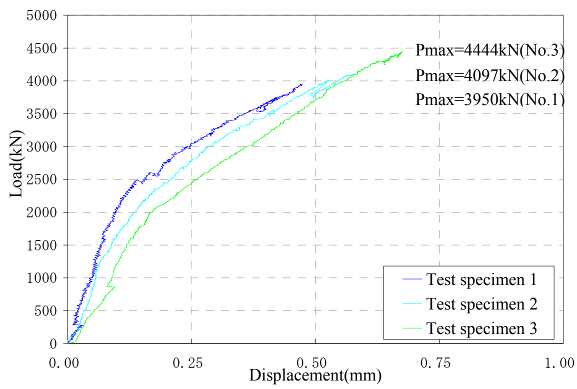
Table 2 shows the strength and the young's modulus of the specimens. Since it was difficult to make steam curing to the UFC wet joint at job site, the target strength of the wet joint was 120 N/mm^2 as determined by the design. At the end of heat curing for the wet joint, its strength was about 120 N/mm^2. Because UFC has a potential of 150 N/mm^2 with normal temperature, the strength had increased to 141 N/mm^2 during 10 days to the loading test

4.2.3 Test results

Figure 15 shows the load vs. displacement curve (relative displacement between the end of a test specimen and the wet joint). Photo 3 - 4 and Figure 16 –17 shows the cracking distribution under the tests.



(1) Type 1 : without shear key

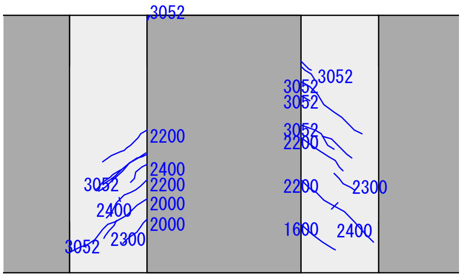


(2) Type 2 : with shear key

Figure 15 Load vs. displacement curve

without shear key (specimen 3)

Photo 3 Clack distribution of type 1

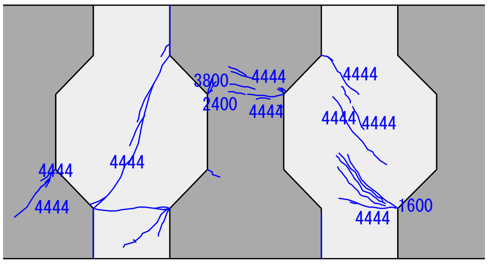


without shear key (specimen 3)

Figure 16 Cracks in the test specimen : Type 1

without shear key (specimen 3)

Photo 4 Clack distribution of type 2



with shear key (specimen 3)

Figure 17 Cracks in the test specimen : Type 2

With the Type 1 test specimen, as shown in Figure 16, cracks developed in the wet joint under loads of 1,600-1700 kN. The number of cracks increased under loads exceeding 2,000 kN, and a minor displacement developed between the wet joint and the concrete under loads of 2,600-2,700 kN. With test specimen 3, cracks eventually developed between the original cracks, and the load-carrying capacity of the test specimen dropped. The load-carrying capacities of test specimens 1 and 2 were lower than that of test specimen 3 because the tests on test specimens 1 and 2 were stopped when the joint was slightly slided under loads of 2,600-2,700 kN. The test results show that the stress transfer mechanism of the wet joint was as follows: at the interface between the wet joint and the concrete, forces were transferred with frictional, bonding, and other forces; and a compression strut formed in the wet joint to transfer the forces. The failure mode was not slippage at the interface but diagonal compression failure of the compression strut. The maximum load was more than the design shear capacity of 1621 kN, and calculated factor β was more than 0.7. The test verified that the wet joint had higher shear resistance than had been assumed in the design.

With the Type 2 test specimen, as shown in Figure 17, cracks developed from the corner of the shear key under loads of 2,400 kN. The number of cracks increased sharply under loads near the maximum load, at which point diagonal cracks developed between the shear keys, and the load-carrying capacity dropped.

Table 3 Maximum load of test

unit:kN

	TYPE1	TYPE2	TYPE2-TYPE1
No. 1	3, 052	3, 950	－
No. 2	2, 741	4, 097	－
No. 3	2, 645	4, 444	－
Ave.	2, 813	4, 164	1, 351

Table 3 shows Maximum load of test specimen Types 1 and 2. The Type 2 test specimens had a higher average maximum load than the Type 1 test specimens: 2,813 kN vs. 4,164 kN. The shear capacity had increased more than 30% with the shear key. The test proved that the shear key increased the shear capacity.

At type 2, the stress transfer mechanism of the wet joint was also the friction and the compression strut, which was formed in the wet joint. The number of wet joint cracks was apparently less than that of type 1 under the maximum load. The reason was presumed that the compression strut was clearly formed at type 2 due to the shear key. Figure 18 describes the stress transfer mechanism of compression strut.

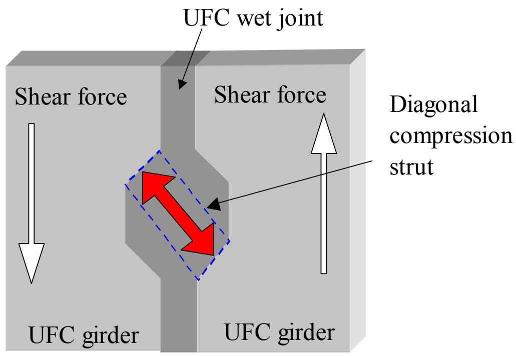


Figure 18 Stress transfer mechanism

5. CONCLUSIONS

The results of the tests verified that the PBL joint between the concrete slab and the UFC girder of the GSE bridge had sufficient load-bearing capacity. Further, it was verified that the UFC wet joint had sufficient shear transfer capacity even under large loads, such as Towing tractor loads, and the design on the sufficiently safe side would be made possible by conforming to the Guidelines for the UFC.

The new material UFC was applied to this bridge and the technology of jointing the members was also introduced to the bridge. The substantial reductions in girder height and self-weight was realized, which contributed to reducing costs of the approach section and substructure, by the effective use of the advanced technology. In addition, the use of the UFC contributed to improving the durability of the GSE Bridge.

The attempt made this time for the GSE Bridge is considered an example of the UFC road bridge that could be presented as a result of not merely applying the new material but also introducing many new structural and construction technologies and verifying the validity of the technologies. It is hoped that the various technologies used for the bridge can help develop the UFC and concrete technologies.

The authors would like to thank Prof. Kouichi Maekawa, the University of Tokyo, for his guidance and advice in the tests.

Photo 5　Erection of UFC precast blocks

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