

DESIGN AND PERFORMANCE TESTS FOR BRIDGE USING ULTRA HIGH STRENGTH FIBER REINFORCED CONCRETE

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Abstract

In the Apron Construction Project at Tokyo International Airport, a single-span concrete bridge was constructed over the road connecting the south and north aprons (hereinafter referred to as the GSE bridge). A rational and cost-effective bridge design was achieved by the use of ultra high strength fiber reinforced concrete (UFC) despite the height, width, and breadth restrictions of the bridge on the connecting road. This paper presents the features of UFC, outlines the design of the GSE bridge and describes the tests conducted to verify the performance of the bridge.

1. Introduction

The GSE bridge crosses over the road that connects the south and north aprons. The bridge consists of the bridge section and the approach fill as shown in Figure 1.1. GSE is an abbreviation for "ground support equipment," which comprises the vehicles, equipment, and facilities used for supporting aircraft operations. The GSE bridge needed to meet the following requirements ; (1) Minimum volume of approach fill. (2) Secure the required clearance between the girder and the connecting road. That is, the bridge body needed to be lowered to the clearance limit as close as possible and the girder depth needed to be as low as possible. To meet these requirements, UFC was adopted for the superstructure for the following reasons.

- UFC has higher compressive and tensile strengths than ordinary concrete, so the girder depth can be reduced by 80% as compared with ordinary concrete bridges.
- Scaffolding for painting work is not needed for UFC bridges, so the bridge body can be lowered as close as possible to

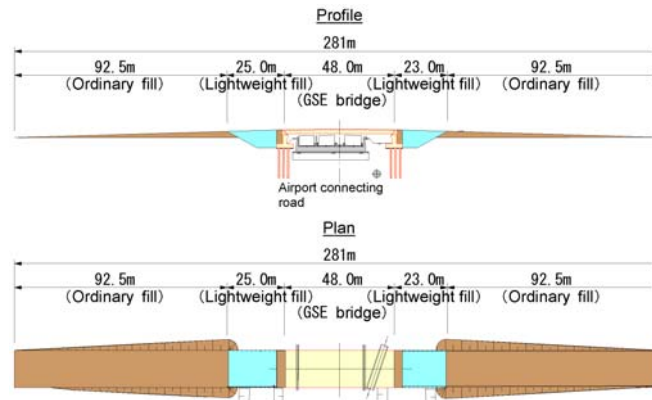


Figure 1.1 Outline drawing of the GSE bridge

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the clearance restriction.

- The dead weight of the superstructure of the GSE bridge can be decreased by 60% as compared with ordinary concrete bridges, reducing the volume of substructure work.

2. Specifications of the GSE bridge

Specifications of the GSE bridge are shown as the following.

Design speed:	40 km/h
Live load:	Towing tractor weight, $W = 500\text{kN}$
Type of structure:	Superstructure: Simply supported UFC box girder bridge Substructure: Inverted T type abutment Steel pipe piles, placed using the earth auger method
Bridge length:	$L_0 = 48.0\text{ m}$
Overall bridge width:	$B = 16.2\text{ m}$
Horizontal alignment:	$R = \infty$
Longitudinal slope:	3%
Cross slope:	2%

3. Basic characteristics of UFC

The UFC used for the GSE bridge is Ductal, an inorganic composite material made from the reactive powders of cement, pozzolan, and other materials. The UFC has the following features. (1) Ultra high strength ; The UFC has a characteristic compressive strength of 180 N/mm^2 . It is possible for the design to take into account the tensile strength of the concrete. Because standard curing (heat curing at 90°C for 48 hours) develops UFC's specified strength and stabilizes its physical properties, it is not necessary to use the curing age method to control the strength, and shrinkage and creep are minimal. Table 3.1 lists the physical properties of the UFC and ordinary high strength concrete. (2) High durability ; The UFC has a densely packed microstructure in which the water-to-cement ratio is lowered to near the hydration limit (0.24 or less) and voids are reduced to the limit. As can be seen from Table 3.1, the UFC is highly resistant to mass transfer. That is, the coefficient of water permeability and the diffusion coefficient of chloride ion in the UFC are about $1/10^6$ and $1/300$ those of ordinary high strength concrete. As specified in the Guidelines for Design and Construction of Ultra High Strength Fiber Reinforced Concrete (Draft) ¹⁾ issued by the Japan Society of Civil Engineers (JSCE) (hereinafter referred to as the Guidelines for UFC), Ductal can be used for more than 100 years without special repairs or reinforcement. Figure 3.1 shows the results of a test calculation of the changes over time in the chloride ion concentration at the interface between a concrete cover (20-mm thick) and a steel test piece. As is clear from the figure, assuming that the splash zone (a chloride ion concentration of 13 kg/m^3 at the surface of the concrete) is a particularly corrosive environment and that steel corrodes at a critical concentration of 1.2 kg/m^3 , the steel will not corrode for more than 300 years after the concrete is brought into

service. (3) High ductility ; The UFC contains high strength steel fibers (0.2 mm in diameter and 15 mm long, 2% by volume), which makes it highly ductile. No rebars are necessary. (4) High flowability ; Mixed mortar has a flow of 200-300 mm. The UFC has high flowability and does not require compaction.

Table 3.1 Physical properties of UFC

Item	Unit	UFC ^(*)	Ordinary high strength concrete (typical)
Compressive strength ^(*)	N/mm ²	180	40
Tensile strength ^(*)	N/mm ²	8.8	2.7
Strength at which cracking occurs ^(*)	N/mm ²	8.0	1.2
Young's modulus	kN/mm ²	50	31
Weight per unit volume	kN/m ³	25.5	24.5
Shrinkage strain		50×10^{-6}	230×10^{-6}
Creep coefficient		0.4	2.6
Coefficient of water permeability	cm/s	4×10^{-17}	10^{-10}
Diffusion coefficient of chloride ion	cm ² /year	0.002	0.700

(*) After standard curing

(*) Characteristic value

Chloride ion concentration at the interface between concrete cover (20-mm thick) and steel test piece

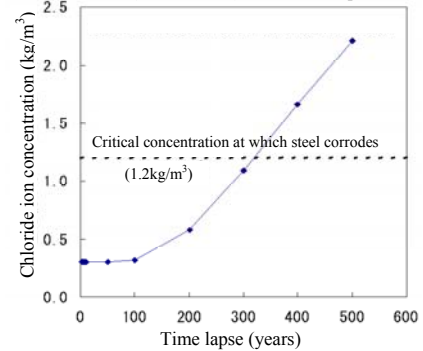


Figure 3.1 Changes over time in chloride ion concentration

4. Design of the GSE bridge using UFC

4.1 Design policy

The superstructure consists of three main box girders. Each box girder has an ordinary PC slab for the top slab and a UFC girder for the U-shaped web and bottom slab (Figure 4.1). The main cables are arranged using an internal cable system to alleviate the stress induced in the cross beam at the end supports due to centralized anchorage of the prestressing steel.

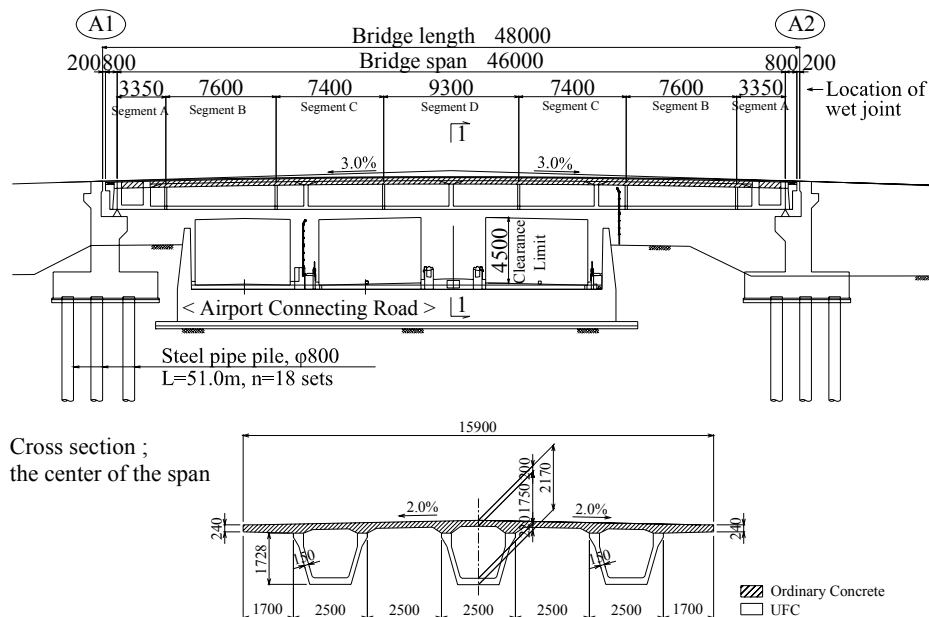


Figure 4.1 General structure drawing

The bridge was designed by the limit state design method in accordance with the Guidelines for UFC. In other words, the stress and sectional force induced in the structure in the serviceability limit state and ultimate limit state under external forces were confirmed to be below the limit stress and the limit capacity of section, respectively.

4.2 Division of the main girder into segments

UFC girders generally require heat curing and shop fabrication to ensure quality, and, therefore, inevitably of precast structure. Taking into account the capacity of the lifting equipment at the shop, it was decided that each segment of the precast girder should weigh about 10 tons. This meant dividing the 47.6 m long main girder into seven segments (Figure 4.1).

4.3 Design of the UFC girder

This paper describes the results of a study of the serviceability limit state. The structural safety of the bridge under flexural, axial, and shear forces in the serviceability limit state was studied in accordance with the Guidelines for UFC. That is, the stress calculated from the sectional force and the rigidity of a member was confirmed to be below the limit stress. Figure 4.2 shows the distribution of maximum and minimum flexural stresses at the uppermost and lowermost ends of the UFC girder in the serviceability limit state. The limit tensile stress for the general section was -8.0 N/mm^2 , whereas the joint between the segments needed to be fully prestressed. Therefore, the cross section and the required quantity of prestressing steel were calculated for the joint nearest to the center of the span.

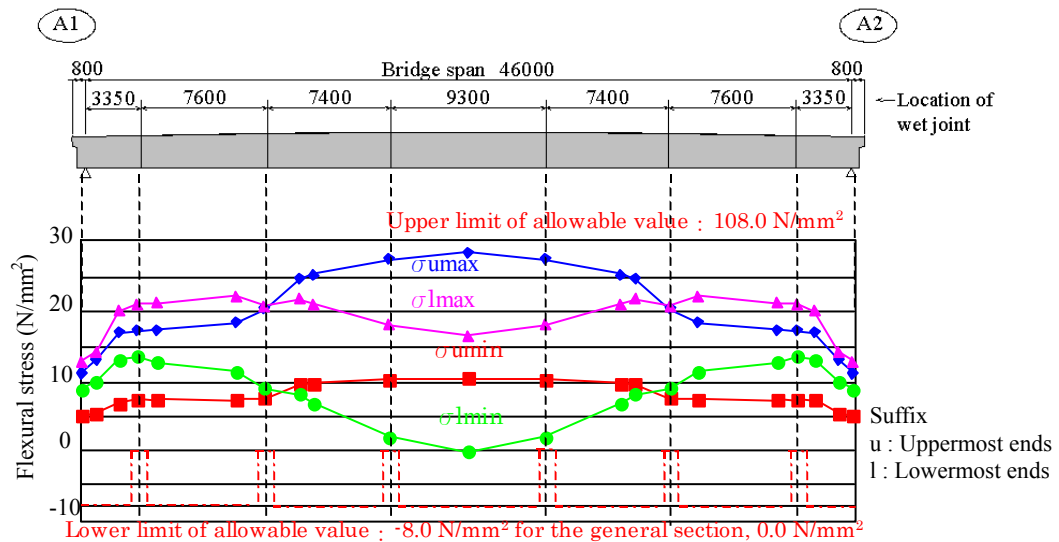


Figure 4.2 Maximum and minimum flexural stresses of the UFC girder

4.4 Design of shear connector between PC slab and UFC girder

Perfobond Strip shear connector (PBL) was adopted for the joint between the UFC girder and the PC slab (shown in Figure4.3). Because of the long overhanging slab, the PBL 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.

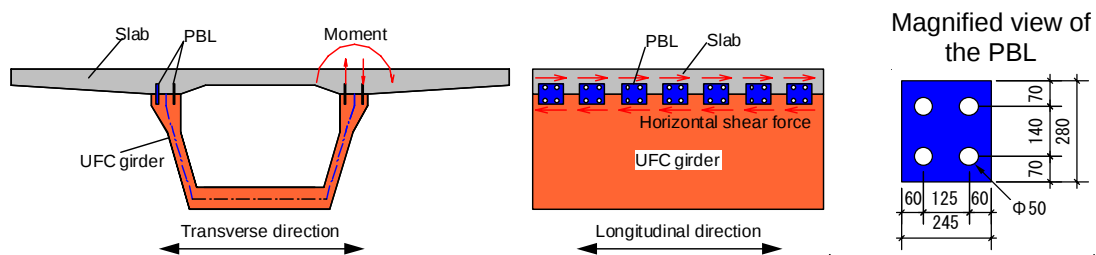


Figure 4.3 Outline drawing of Perfobond Leisten (PBL)

The shear capacity of the PBL on the PC slab side (ordinary concrete) was calculated in accordance with the Design Manual of Perfobond Strip Shear Connector (Draft) issued by the JSCE. The shear capacity on the UFC side was set to one-third the shear capacity determined from the formula for calculating the shear capacity²⁾ without reinforcing bars based on historical usage. Tables 4.1 and 4.2 and Figure 4.4 show the specifications and verification results of the PBL. The horizontal shear force between the PC slab and UFC girder at the girder ends is larger than that in the center of the span due to differences in temperature, creep and drying shrinkage. For this reason, the method of laying out the PBL was changed. The specifications of the PBL were determined by the horizontal shear force on the UFC side in the serviceability limit state.

Table4.1 Specifications of the PBL (Material: SS400)

		Range 1	Range 2
Material	-	SS400	SS400
Thickness	mm	22	22
Hole diameter	mm	50	50
Spacing between holes	mm	125	150
Spacing between steel plates	mm	250	500
Through reinforcing bar (slab)	mm	D22	D22

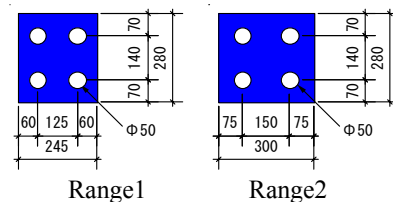


Figure 4.4 Shape of the PBL

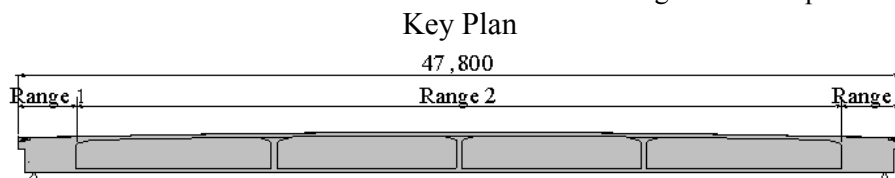


Table 4.2 Results of design of the PBL
(1) Results of checks on the PBL for horizontal shear capacity

				Range 1			Range 2		
				Design value	Capacity	Decision	Design value	Capacity	Decision
Serviceability limit state	UFC girder	Shear capacity	kN	89	98	O.K.	80	98	O.K.
	PC slab	Shear capacity	kN		111	O.K.		111	O.K.
		Splitting capacity	kN		109	O.K.		109	O.K.
	Steel plate	Shear capacity	kN		123	O.K.		164	O.K.
Ultimate limit state	UFC girder	Shear capacity	kN	98	296	O.K.	89	296	O.K.
	PC slab	Shear capacity	kN		338	O.K.		338	O.K.
		Splitting capacity	kN		330	O.K.		330	O.K.
	Steel plate	Shear capacity	kN		373	O.K.		498	O.K.

(2) Results of checks on the PBL under the moment in the transverse direction

				Range 1			Range 2		
				Design value	Capacity	Decision	Design value	Capacity	Decision
Serviceability limit state	UFC girder	Shear capacity	kN	35	98	O.K.	70	98	O.K.
	PC slab	Shear capacity	kN		111	O.K.		111	O.K.
		Splitting capacity	kN		109	O.K.		109	O.K.
	Steel plate	Shear capacity	kN		74	O.K.		74	O.K.
Ultimate limit state	UFC girder	Shear capacity	kN	41	296	O.K.	83	296	O.K.
	PC slab	Shear capacity	kN		338	O.K.		338	O.K.
		Splitting capacity	kN		330	O.K.		330	O.K.
	Steel plate	Shear capacity	kN		224	O.K.		224	O.K.

4.5 Design of the wet joint

A wet joint was adopted for the joint between the precast girders. 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/mm². 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 shear capacity between the wet joints was set to either the design shear capacity of a rod member specified in the Guidelines for UFC or the shear transfer capacity of a joint specified in the Standard Specifications for Concrete Structures: Structural Performance Verification (Equation 1), whichever was smaller. In the design, factor b, representing the shape of the plane, was set to 0.4 based on past performance records. The validity of this value was verified by performance tests to be described later in this paper.

$$V_{yd} = V_{c wd} + V_{p ed} \quad (1)$$

where:

$V_{c wd}$; design shear transfer capacity

$$V_{c wd} = (\tau_c \cdot A_{cc} + V_k) / \gamma_b$$

$$\tau_c = \mu \cdot f_{cd}^b \cdot \sigma_{nd}^{1-b}$$

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

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

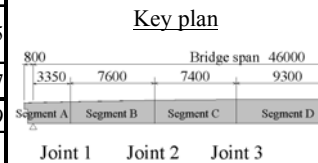
f_{cd} ; design compressive strength of concrete

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

Table 4.3 lists the test results for the shear capacity of the wet joint. The wet joint was confirmed to be sufficiently resistant to shear forces.

Table 4.3 Results of checks on the wet joint for shear capacity

			Unit	Joint 1	Joint 2	Joint 3
G1 (Outside girder)	Design shear force	V_d	kN	3,040.1	1,808.3	1,229.9
	Design shear transfer capacity	V_{yd}	kN	6,203.5	4,730.2	3,135.5
	Test results	$\gamma_i \cdot V_d / V_{yd} \leq 1.0$		0.59	0.46	0.47
G2 (Inside girder)	Design shear force	V_d	kN	2,899.7	1,760.9	1,097.9
	Design shear transfer capacity	V_{yd}	kN	5,851.1	4,800.5	3,457.1
	Test results	$\gamma_i \cdot V_d / V_{yd} \leq 1.0$		0.60	0.44	0.38



5. Performance tests of the UFC girder

5.1 Objective of the tests

To verify the following performances of the bridge, tests were conducted.

- (1) The pulling force resistance of the PBL in the structure of the joint between the UFC girder and PC slab
- (2) The shear transfer capacity of the wet joint

5.2 Elemental test on the PBL

5.2.1 Details of the test

This test was performed to verify that the PBL on the UFC side has higher resistance to pulling forces than are assumed in the design and that the PBL 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 5.1. 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.

5.2.2 Loading method

As shown in Figure 5.2, 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 5.3 shows loading steps of the test.

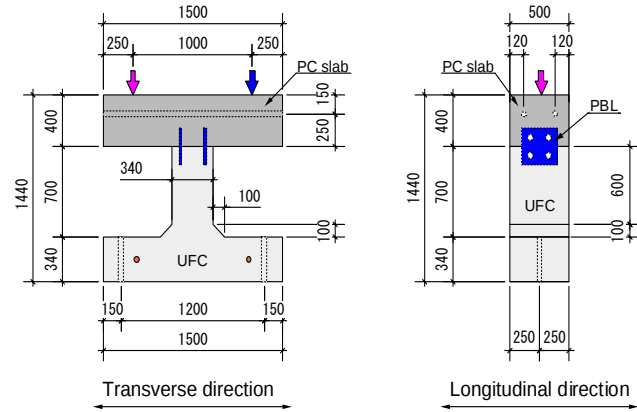


Figure 5.1 Test specimen

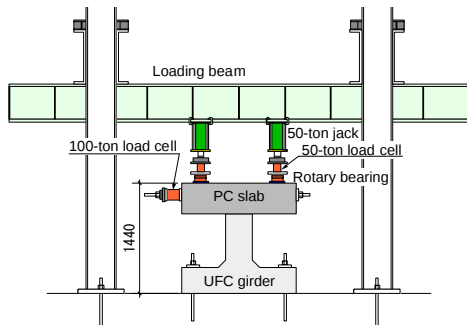


Figure 5.2 Loading test equipment

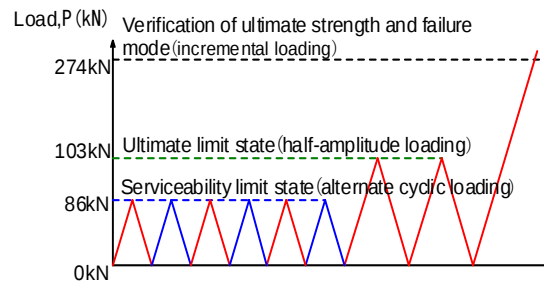


Figure 5.3 Loading steps

5.2.3 Test results

Figure 5.4 shows the load vs. displacement curve. The test specimen did not crack under the design load (in the serviceability limit state) of 86 kN, and the test specimen exhibited elastic behaviors with little variation after the design load in the serviceability limit state was applied three times

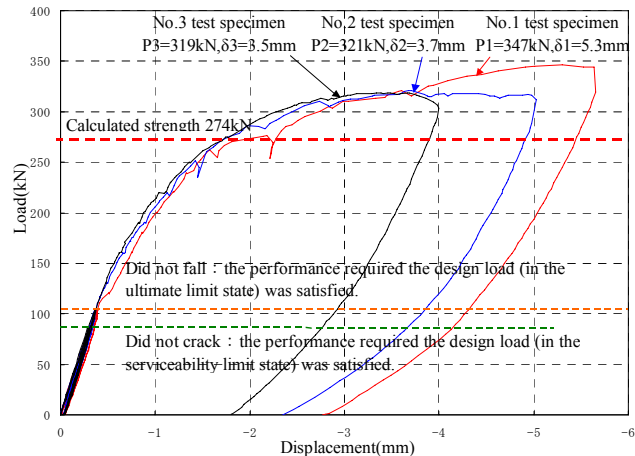


Figure 5.4 Load vs. displacement curve

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 edge between the PC slab and the web was displaced 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 developed, the rigidity of the test specimen dropped, and the load reached its peak. Figure 5.5 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.

This test verified that the PBL 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.¹

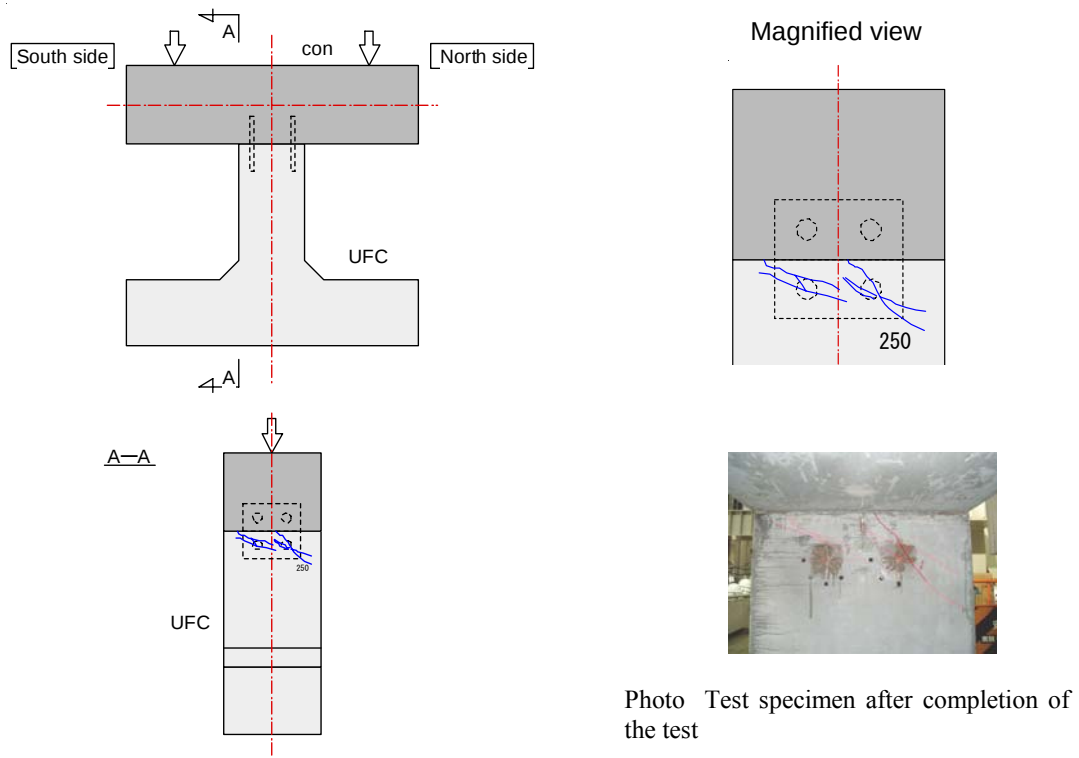


Figure 5.5 Cracks in the test specimen after completion of the test

5.3 Elemental test on a wet joint

5.3.1 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 5.6. The specimen comprised the wet joint and surrounding parts of the bridge. Two test specimens were prepared: one with a shear key and one without (Types 1 and 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.

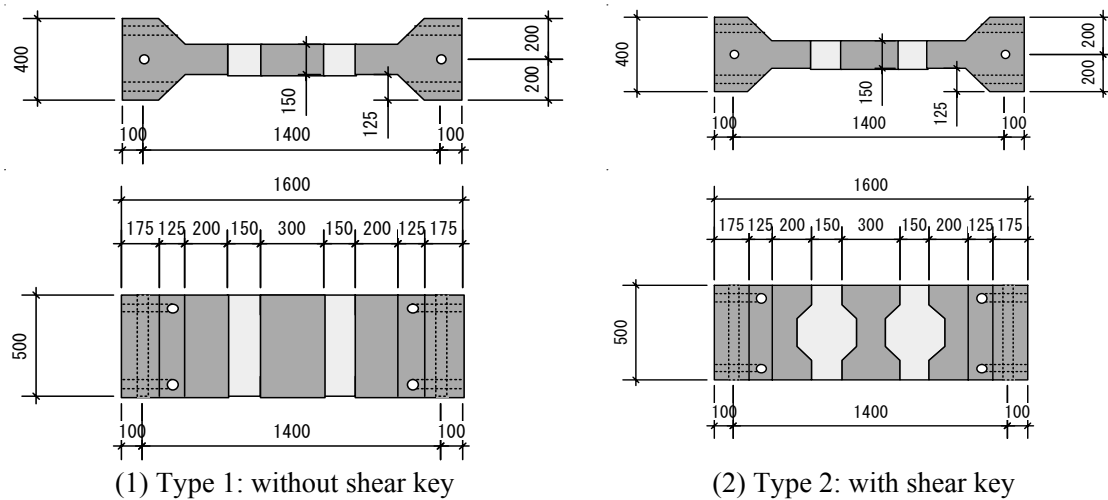


Figure 5.6 Test specimens

5.3.2 Loading method

As shown in Figure 5.7, 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 specimen. An average design compressive stress of 10 N/mm² 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.

5.3.3 Test results and discussions

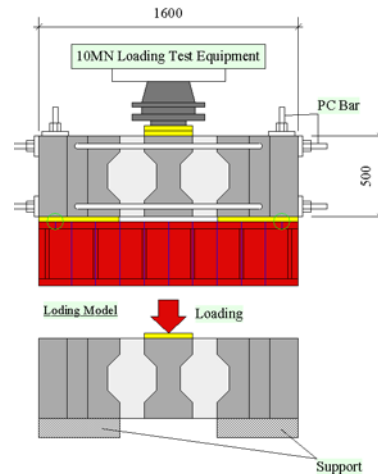


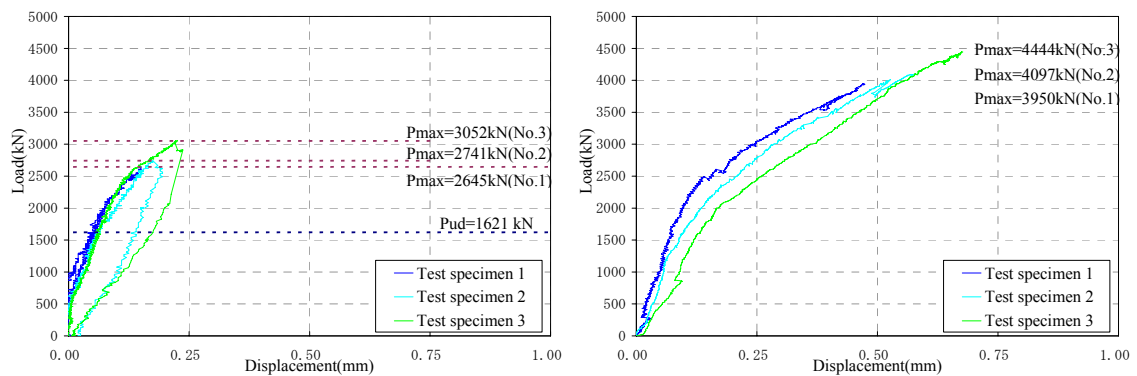
Figure 5.7 Loading test equipment

Figure 5.8 shows the load vs. displacement curve (relative displacement between the end of a test specimen and the wet joint). Figure 5.9 shows the cracking distribution under the tests.

With the Type 1 test specimen, as shown in Figure 5.9, 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 ended when the joint became displaced under a 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. The test verified that the wet joint had higher shear resistance than had been assumed in the design.

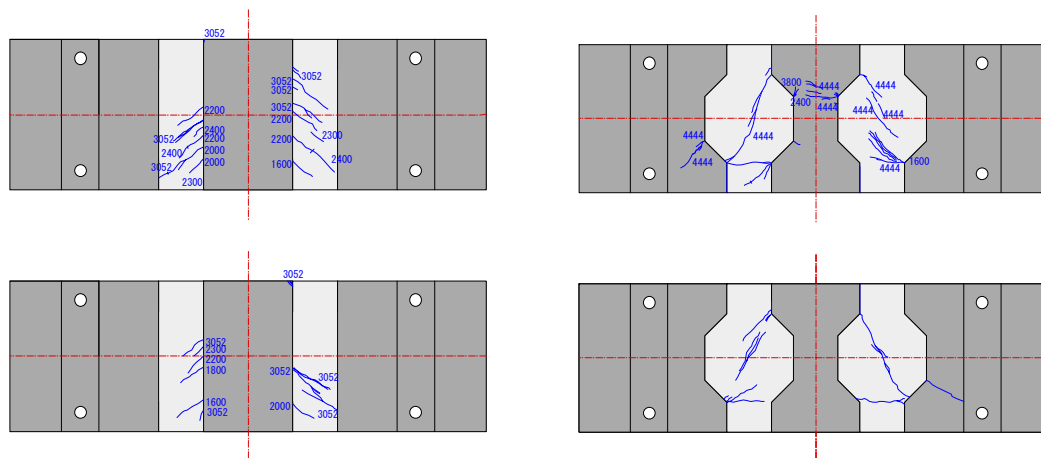
With the Type 2 test specimen, as shown in Figure 5.9, cracks developed from the corner of the shear key under loads of 1,600-1700 kN. The number of cracks increased sharply under loads near the maximum load, at which point diagonal cracks developed between the shear keys, and the load-carrying capacity dropped. Table 5.1 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 test proved that the shear key increased the shear capacity.



(1) Type 1 : without shear key (2) Type 2 : with shear key
Figure 5.8 Load vs. displacement curve

Table 5.1 Maximum load of test specimen Types 1 and 2 (load across two joints)
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



(1) Type 1 : without shear key

(2) Type 2 : with shear key

Figure 5.9 Cracks in the test specimen after completion of the test

Conclusions

This paper describes design and performance tests for the GSE bridge. The bridge incorporates a new technology, UFC, which permitted a reduction in the girder depth and minimized the volume of fill needed for the approaches, thereby ensuring the necessary clearance between the girder and the connecting road. Moreover, the use of USC reduced the volume of substructure work, which helped to reduce construction costs. UFC also provides the bridge with greater durability. Tests verified that the joint between the PC slab, the UFC girder, and the wet joint between the UFC girders exceeded the design performance specifications.

The authors would like to thank the members of the expert committee and others who were part of this study for their valuable opinions on the design of the bridge.

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- 2) T Hosaka, K Mitsuki, H Hiragi, Y Ushijima, Y Tachibana, H Watanabe, “An Experimental Study on Shear Characteristics of Perfobond Strip and it’s Rational Strength Equations”, Proceedings of Structural Engineering, pp.1593~1604, Vol.46A, March 2000