Innovation and application of UFC bridges in Japan

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Summary
The design and construction technology of UFC bridges in Japan has been advancing in recent 5 years. This paper presents representative 13 projects of UFC structures that have been designed and constructed in Japan as well as the innovative technologies that have been applied step by step through the verification due to the fundamental experiments or accumulated construction know-how. Recent 4 projects among those 13 projects are introduced in more detail. First one is Toyota Footbridge that was applied for the first time the dry joint technology. Second one is a 40 m span monorail girder bridge that may be the longest span girder made of concrete. Third project is a just completed GSE (Ground Support Equipments) bridge located in Haneda international airport. The last project is the mass production of UFC slabs in Haneda Runway D that must be the largest single project in the world in application of UFC.

Keywords: Pre-cast segments; wet joint; dry joint; pre-stressed structures; long span bridge; perfobond strip (PBL); hybrid structure; materials combinations; tension softening of UFC; Japanese design recommendation for UFC

1. Introduction
A 50 m span Sakata-Mirai Footbridge has been completed for the first time in Sakata City applying Ultra high strength Fibre reinforced Concrete (hereafter referred to as UFC), the brand name of Ductal® in October 2002. Based on this achieved design, construction technology, material test data and structural experiments, "Recommendation for Design and Construction of Ultra High Strength Fibre Reinforced Concrete Structures, -Draft" (hereafter referred to as "UFC recommendation") was published in 2004 by Japan Society of Civil Engineers [1].

Because UFC is new generation concrete material possessing high strength, high ductility, high fluidity and high durability, it is possible to drastically reduce the self weight of the structures made of UFC compared with those made of ordinary concrete. The authors have been challenging the technical development of new design method, joint technology, efficient production methods of pre-cast segments and construction methods on site for various types of UFC structures.

In this paper, the representative 13 projects of UFC structures are presented and the recent 4 projects among those projects are introduced in more detail featuring the innovated technologies. The material performance and accumulated field data of Ductal® are also introduced. Furthermore, the analyses of the advantages and disadvantages of UFC structures in Japan based on our experiences of the past projects are discussed.
2. Material performance of UFC

2.1. Mechanical property

2.1.1. Compressive strength

The formulation of Ductal® produced by Taiheiyo Cement Corporation is shown in Table 1. In the Sakata-Mirai Footbridge project, 6 test cylindrical specimens with the size of Φ10x20cm were sampled for each pre-cast segment. The mean value of compressive strength for sample data N=48 was 196 N/mm² and the coefficient of variation was only 3.6%. In the mass production of slabs for Haneda Runway D, 3 test specimens were sampled for every 40 m³ in the batcher plant. The mean value of compressive strength for sample data N=378 was 210N/mm² and the coefficient of variation was only 5.7%. It should be noted that the coefficient of variation for compressive strength for Ductal® is very low compared to that of ordinary concrete that may be from 10% to 15% according to the information of Japan Ready Mixed Concrete Institute. According to the field data of Haneda Runway D project, the characteristic value that signifies the compressive strength resulted in 190N/mm² that is surly cleared the characteristic value fck=180N/mm² recommended by UFC recommendation.

<table>
<thead>
<tr>
<th>Table 1 Mixed proportion of Ductal</th>
<th>unit quantity (kg/m³)</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>pre-mixed powder</td>
<td>1308</td>
<td></td>
</tr>
<tr>
<td>fine aggregate</td>
<td>932</td>
<td></td>
</tr>
<tr>
<td>fibre</td>
<td>157</td>
<td></td>
</tr>
<tr>
<td>super-plasticizer</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td>water</td>
<td>180</td>
<td>22</td>
</tr>
<tr>
<td>W/C</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

2.1.2. Tensile strength

Unlike conventional reinforced concrete, it is a special characteristics of the UFC that tensile strength itself develops in the concrete. According to UFC recommendation, two important characteristic values of tensile strength are defined; i.e. one is the first cracking strength fck and another is the tensile strength ft. For the design of service limit state, the tensile stress generated in the UFC members should not reach the predetermined first cracking strength fck. For the design of ultimate limit state, the tensile softening diagram (abbreviated to TSD) as shown in Fig.1 is used to ensure the safety verification for the UFC structures. The direct tensile test would be essentially the best way to obtain the tensile strength and the TSD. The UFC recommendation adopted the inverse analysis for the flexural test results. Fig.1 demonstrates the recommended TSD derived from inverse analysis applied to the results of three-point bending tests using notched and un-notched prismatic specimens with 10x10x40cm. The characteristic values, the tensile strength fck=8.8N/mm², end of plateau w1=0.5mm, and zero stress point w2=4.3mm for the design TSD are determined so as to fit the inverse analysis.

Due to the UFC recommendation, the split-cylinder test with Φ10x20cm cylinder attached with strain gauges on the surfaces of both ends to observe the load and strain relationship is a preferable method for determination of the characteristic value of the first cracking strength fck. The characteristic value of the tensile strength fck may be determined from the flexural strength obtained...
from the three-point bending tests for the prismatic specimens with 10x10x40cm. In the Haneda Runway D project, the characteristic values of the compressive strength $f_{ck}$, the first cracking tensile strength $f_{ck}$ and the tensile strength $f_t$ are obtained as shown in Table 2 in order to manage the quality control of the UFC material. Those field data were acquired from the samples of the batcher mixing plant operating from the end of 2007 to the end of 2008.

<table>
<thead>
<tr>
<th>statistics</th>
<th>unit</th>
<th>compressive strength</th>
<th>first cracking tensile strength</th>
<th>tensile strength</th>
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<tr>
<td>sample number</td>
<td>sample</td>
<td>378</td>
<td>378</td>
<td>378</td>
</tr>
<tr>
<td>mean value</td>
<td>N/mm²</td>
<td>210</td>
<td>10.2</td>
<td>14.4</td>
</tr>
<tr>
<td>coeff. of variation</td>
<td>%</td>
<td>5.7</td>
<td>7.8</td>
<td>13.9</td>
</tr>
<tr>
<td>characteristic value</td>
<td>N/mm²</td>
<td>190</td>
<td>8.9</td>
<td>11.0</td>
</tr>
</tbody>
</table>

**2.2. Durability**

2.2.1. Pore volume ratio

The matrix of the UFC forms a dense structure due to the densest packing and microstructures resulting from pozzolanic reaction of silica fume. The pore size distribution of the UFC is therefore quite different from the ordinary concrete. Fig. 2 shows the comparison of pore size distribution between the UFC and the ordinary high strength concrete with W/C=30%. The relationship between the pore volume ratio and the pore diameter was predicted by applying a mercury intrusion porosimetry apparatus. The pore structure is designated depending on the pore size; i.e. it is defined as ‘Gel Pore’ for the size up to 10nm on the other hand it is defined as ‘Capillary Pore’ for the size from 10nm to 1μm. The capillary pore is a continuous pore like ameba and this is related to the water permeability; i.e. the lesser the capillary pore the better durability. It is obvious that the UFC has extremely denser structure than the ordinary concrete. Fig. 2 shows that the pore volume ratio of the UFC for either heat curing or curing in water is 4.3-4.5%, where that for high strength concrete with W/C=30% is 9.0%. In addition, the pore volume ratio of capillary pore of the UFC is only 0.5%, where that for high strength concrete is

**Fig.2 Comparison of pore volume ratio**

**Fig.3 Chloride ions profile**
5.8%.

2.2.2. Diffusion coefficient of chloride ions obtained by field observation

Because Sakata-Mirai Footbridge is located in Sakata City facing to the Japan Sea, the footbridge has been attacked by severe corrosive winds in winter. In order to investigate the durability time history, Ductal specimens were exposed inside of the box girder of the footbridge. The specimens of 4x4x16cm regular prism that were exposed for about 5 years inside the box girder were sliced off 10mm thickness. EPMA (Electron Probe Micro Analyzer) was used to predict the apparent chloride diffusion coefficient through the regression curves. The chloride ions concentration profile and the regression curve are illustrated in Fig. 3. From the regression function, the apparent chloride ion diffusion coefficient was predicted to be 0.000148 cm²/year that is smaller than that of the UFC recommendation.

3. Application of UFC bridges

3.1. Footbridge

3.1.1. Sakata-Mirai Footbridge (completion in 2002, Photo 1 and 2)

This footbridge is a pre-tensioned box girder segments made monolithic with external tendons to a 49.2m single-span and was designed and constructed for the first time in Japan as a UFC bridge. The pre-cast segments are composed of two types of segments; i.e. one is closed form cross-section and another as shown in Photo 2 is composed of two symmetrical segments being divided into half at the centre of the cross section. The total dead weight is only 540kN because the members of girder are very thin; i.e. the top slab is 5cm thick and the web is 8cm thick. The dead weight of the bridge is about one-fifth of that of the bridge made of the ordinary concrete, therefore the cost of the foundation was drastically reduced. This bridge was planned to replace four span old pre-stressed concrete bridge and has a restriction that the bridge bottom line should not be less than 0.6m from the high water level, therefore this bridge has low girder height (55cm at girder ends) and long span. The ratio of girder height to span is 1/90 at the girder ends and 1/32 at the centre of the girder. Furthermore the distinctive design such as large circular openings in the webs was achieved.

3.1.2. Akakura Onsen Yukemuri Bridge

(completion in 2004, Photo 3)

This pedestrian bridge is a 36.4m long, single-span PC box girder-bridge with an effective width of 3.0m. The thickness of the top and bottom slab and web are 7cm. The girder height is 95cm and the ratio of girder height to span is 1/40. The top slabs and the U-shaped girders were separately manufactured as pre-cast segments in the factory. The top slabs and the U-shaped girders were
segments were connected together by perfobond strips (abbreviated to PBL) on construction site. This PBL joint method can be compact in size compared with the ordinary shear studs because the shear transfer stiffness of PBL is stiffer than the ordinal studs. Furthermore the erection of the top slabs is easy and quick. The connection procedure is only to cast-in-situ the UFC into the PBL joints.

3.1.3. Keio University Pedestrian Bridge (completion in 2005, Photo 4)

The pedestrian bridges were designed to connect from the first floor to the fifth floor in an atrium. This bridge is a single-span slab type pre-tensioned girder. The length is 11.3m, the width is 2m and the height at the centre is only 22.5cm. Due to the architectural design sense, the atrium covered with glasses was required to be rich in light, therefore the thinner the height of bridge the better design. The edge height of the bridge is 4cm. The pre-stressing PC tendons were 18 mono-strands with the diameter of φ15.2mm. After the primary curing, the compressive strength of the UFC was 78N/mm² and the effective bond length at each end seemed to be 50cm (35 times of strand diameter) from the observation of the strain data of the strands.

3.1.4. Hikita Footbridge (completion in 2007, Photo 5)

This footbridge is a single-span trough-girder PC bridge with 63.3m span length. This span length may be the world's longest span bridge of trough-girder type made of the UFC. The bridge length of the bridge is 64.5m, the width is 2m, the web thickness is 12cm and the bottom slab thickness is 25cm. For the design purpose many circular openings in the webs were equipped. The U-shaped pre-cast segments are manufactured in the factory and they were conveyed to the construction site. Those pre-cast segments were jointed by wet joints and were stressed by internal PC tendons.

3.1.5. Mikaneike Footbridge (completion in 2007, Photo 6 and 7)

This 81.2m in length and 3.5m in width PC bridge has two-span (2x39.3m) continuous PC box girder. The continuous type PC girder made of the UFC may be constructed for the first time. The girder height is 1.0m so the ratio of girder height to span is 1/40. The top slab is 7cm thick, the bottom slab is 8cm thick and the web varies from 8 to 20cm thick. The cross section of the box girder was divided into top slabs and lower U-shaped girders. The U-shaped pre-cast segments were connected together by wet joints and the top
slabs and U-shaped girder were jointed together by PBL joints.

3.1.6. Kobe Sanda Premium Outlet Footbridge
(completion in 2008, Photo 8)
This footbridge is a single-span trough-girder PC bridge with a 26.3m span length. Because the bridge width is 4.1m, the U-shaped pre-cast segments were divided into 9 standard pre-cast segments with about 2.5m in length and 2 end pre-cast segments with about 2.45m in length to be conveyed by trucks. The thickness of web and bottom slab for the standard segments is 8cm and 13cm, respectively, on the other hand for the end segments is 22cm and 28cm. The pre-cast segments were connected by wet joint with longitudinal curvature next to the construction site. After completion the joint, the bridge was installed on the shoes in the midnight by crane.

3.1.7. Akasaka Yagenzaka Footbridge (completion in 2009. Photo 9)
This footbridge is a single-span two boxes PC bridge with a 20.2m span length. As the width of the bridge was 6.0m, two boxes composed of U-shaped girders pre-cast segments 2.0m wide and 11.0m long were manufactured. The bridge height is 0.6m and the ratio of the bridge height to the span is 1/34. The top slabs were divided into 9 pre-cast segments. The U-shaped girders were connected together by wet joints and the girder and the top slabs were jointed together by PBL joints.

3.2. Highway bridge

3.2.1. Horikoshi C-Ramp Bridge (completion in 2005, Photo 10 and 11)
This 16.6m long highway bridge has a span of 16m. It is located at the Kita Kyushu Junction. The bridge was originally designed as a simply supported pre-tensioned conventional concrete hollow slab girder, but the design was later changed to a composite girder bridge with UFC main girders and a cast-in-situ PC slab. This was the first time in Japan that the UFC had been applied to the main girder of a road bridge. The composite girder bridge is composed of four UFC pre-tensioned I-shaped girders as shown in Photo 10 and the cast-in-situ conventional pre-stressed concrete slab. The I-shaped girder and the slab was connected by PBL joints. The number of girders was decreased from 11 to 4 and the total dead weight was more than 30% lighter than the original design. The weight of a single girder was reduced from 120kN to 50kN; therefore a smaller crane could be used for erection of the girders.
3.2.2. Torisalogawa Bridge (completion in 2006, Photo 12 and 13)

This 554m long, 11-span, continuous PC box-girder bridge has a maximum span of 56m and is composed of corrugated steel webs and conventional concrete slabs. This bridge was constructed applying incremental launching method. Instead of using a temporary launching steel nose, the front section of the main girder was constructed of corrugated steel webs, a steel upper chord, and the lower chords were constructed of the UFC to save weight. After launching was completed, these components became permanent members. The launching nose was 44.8m long and divided into 5 blocks. The UFC lower chord joint was a cast-in-situ wet joint. By adopting the UFC instead of conventional concrete for the lower chord, the cross section of the launching nose was smaller and the launching nose was lighter.

4. Toyota City Gymnasium Footbridge (completion in 2007)

4.1. Structural features

This footbridge is two-span continuous two-box girder with the dimension of 27.96m in length (span length is 22.5m), 4.72m in width and 0.55m in height (Photo 14). The bottom and the top slab thickness is 6cm and the web thickness is 7cm (Fig. 4). A pre-cast segment method has been adopted accounting for newly developed dry joint method for the preparation of future large projects. The whole structure was divided into 12 pre-cast segments with the length of 1.9~2.5m. 1% parabolic curvature was required in the longitudinal direction. This means the match cast for dry joint should take this curvature into account when manufacturing. The pre-stressing cables were 4 sets of 19s15.2mm external tendons.

4.2. Match cast method for dry joint

The fundamental concept of our dry joint is same as the conventional match cast method usually applied for PC bridges so called pre-cast segment method; i.e. the match cast face is glued by epoxy resin and each segment is connected together by pre-stressing. The different aspect from the conventional one is how to fabricate the match cast segment.

The shrinkage of the UFC is primarily 400~500μm/m autogenous shrinkage that is much larger than the conventional concrete. The match cast face of the new segment is...
usually fabricated by arranging the match cast concrete face of the old segment as a mould.

However, the match cast face of the old segment must already have had some autogenous shrinkage in case of the UFC and this sequentially causes unmatched segments. Our new fabrication method for the segments is that a steel end plate reference mould is set on the old segment to keep constant the sectional dimensions. The mould of the new segments is set on the old segment remaining the steel end plate reference mould on the old segment. As the steel end reference plate is constant thickness, the match cast faces result in the mirror image (Fig. 5).

Another unique point is that all shear keys on both sides of the matching face (Fig. 4), are couples of concave shapes. These UFC shear keys were manufactured in advance and were installed while dry-joint was proceeding. It is noted cracking risks usually generated around the shear keys for the case of the conventional match cast faces, will not happen for this case.

### 4.3. Manufacturing and erection of pre-cast segments

The height of two-box girder is too low to handle a conventional interior mould. New interior mould material was developed to easily relief the interior mould. The material for the interior mould acts as shrinking behaviour due to heat, therefore after the secondary heat curing the interior mould can be easily deleted (Fig. 6). The erection of the pre-cast segments was carried out by crane (Photo 15). After giving a coat of epoxy resin on the matching surface, each pre-cast segment was pressed against the adjacent segments by traction equipment. The temporary contact pressure on the matching surface was 0.3N/mm² (Photo 16).

### 5. Tokyo Monorail Girder (completion in 2007)

#### 5.1. Structural features of long span monorail girder

Tokyo Monorail and Taisei corporations have been carrying out the corporative technical development of a 40m long monorail girder applying the UFC that must be the longest span in the world made of concrete material (Photo17). Because of the length restriction for the case of 40m long monorail girder, three reversed U-shaped girder (hereafter referred to as "rU girder") segments and three bottom slab segments were separately manufactured in the factory, conveyed to the construction site and jointed together by wet joint and dry joint. Three rU girder segments were connected by dry joint, on the other hand three bottom slab segments and rU girders were connected by wet joint. It should be noted that the dry joints for
rU girders are located in inner side of the span but the wet-joints for bottom slabs are located outside of the dry joint as shown in Fig. 7. The tensile stress for the design load of service limit state (SLS) should be less than the first cracking strength (\( f_{c} = 8 \text{N/mm}^2 \)) except the joint section, however the tensile stress for SLS can not be allowed on both types of joint; i.e. it should be full pre-stressing for those joints. It is therefore possible to reduce the pre-stressing cables compared with the case that the locations of both joints coincide. The section of the dry joint and the wet joint is illustrated in Fig.8 and Fig.9, respectively.

5.2. Experiment for proto-type 10m long girder

Most of the modelling parameters of the proto-type 10m long monorail girder such as girder width, combination of joints and pre-cast segments, surface finishing of top slab and sizes of PBL are identical with the 40m long girder except the total length and the height (Fig. 10). There are two main purposes for the proto-type monorail girder; i.e. one is to confirm the erection and fabrication method for the complex composition of segments, and another is to verify the structural safety including two kinds of joints. The sequential fabrication steps such as production of segments, match casting, dry joint and wet joint were implemented and evaluated how those effect on the final structural performance. The rU girder segment is lifted to assemble with the rest of the segments as illustrated in Photo 18 where the match cast face with concave shear keys and PBL inserted in the segment for wet joint with bottom slab are observed.

![Fig. 7 Structural composition of pre-cast segments](image)

![Fig. 8 Dry joint](image)

![Fig. 9 Wet joint](image)

![Fig. 10 Structural composition and loading set up for proto-type girder](image)
The loading set up for the completed proto-type girder was arranged so that both joints could have both bending moment and shear force (Fig. 10). The experimental result and a 3D-FEM analysis considered the modelling of material nonlinearity is indicated in Fig. 11. Because a 10m long monorail girder instead of a 40m girder was to be tested to prove the structural safety, the equivalent loading values were calculated so as to have equivalent forces at joints for SLS and ULS. The loading value $P$ for SLS and ULS became $830\text{kN}$ and $1748\text{kN}$, respectively. It was proven that no initial cracking was observed for SLS and no serious damage for ULS. The first cracking was observed at the bottom slab of mid-span for the loading value $P=1200~1300\text{kN}$. At the same time, the first cracking at wet joint of bottom slab was found. For the loading value $P=1700\text{kN}$, the diagonal cracks were observed on web.

5.3. Fabrication of a 40m long monorail girder

Three rU girder segments and three bottom slab segments were manufactured and conveyed to the construction site. Three bottom segments with PBL were settled on the levelled supporting beams (Photo 19). Three rU girder segments with four adjustable legs were erected on the bottom segments at 10cm above the final level (Photo 20). After giving a coat of epoxy resin on the matching surface, each rU girder segment was pressed against the adjacent segments by traction equipment. The temporary contact pressure on the matching surface was $0.3\text{N/mm}^2$ and those dry jointed segments were set down about 10cm ready to the subsequent wet joint step. As a wet joint step, 1) cast-in-situ UFC was placed into the space between the web and the bottom slab as well as into the space between bottom slabs, 2) heat curing was carried out keeping the atmosphere temperature $60^\circ\text{C}$ for 48 hours. After the compressive strength of wet joint cast-in-situ UFC reaching up to at least $160\text{ N/mm}^2$, the pre-stressing step has been progressed to supply the design stresses not only on the dry joint but also on the wet joint. It should be noted that the top slab surface and the side web surface of the dry joint were finished extremely smooth.

6. GSE (Ground Support Equipments) Bridge (completion in 2008)

6.1. Structural features of GSE Bridge

A single-span GSE Bridge was constructed over the road connecting the south and north apron in the extension of Tokyo International Airport project as shown in Photo 21. The GSE Bridge has a span of 46.0m and width of 16.2m and this road bridge must be the largest bridge made of UFC in the world. The main live load of the bridge is the heavy towing tractor of $500\text{kN}$ in total weight for pulling the aircraft. The wheel load is $125\text{kN}/\text{wheel}$ that is heavier compared with the ordinary truck wheel load. Answering for the request of low bridge height and light dead weight, the end girder height of the bridge resulted in $1.86\text{m}$ (girder height to span ratio=1/25) and the dead weight
reduced 40% when compared with ordinary concrete bridge. The major structural features are 1) three-box girders were composed of U-shaped UFC pre-cast segments, 2) the main deck slab was the cast-in-situ conventional concrete (fck=40N/mm²), 3) one box U-shaped girder was divided into 7 pre-cast segments and those were connected together by wet joint, 4) the deck slab and the girders were jointed together by twin PBL (Fig. 12, Fig. 13 and Fig. 14).

The composite combination of the UFC girders and the conventional concrete slabs has already been achieved in Horikoshi C-Ramp Bridge and the combination of the UFC U-shaped girder and the UFC slabs has also already been accomplished in Akakura Onsen Yukemuri Bridge and Mikaneike Footbridge. However, this road bridge is different from those bridges at the points that 1) the design live load of this road bridge is heavy, 2) the road width is wide and 3) the bridge height to span ratio is small; therefore, the further new technologies have been developed. The major role of the PBL joint between the main girder and the top slab is the shear transfer in longitudinal direction. As this road bridge has the wide overhanging slab, the PBL joint should also resist to pulling forces in vertical direction due to the moment in the transverse direction (Fig. 15). For this reason, twin PBL joints were adopted instead of single one. Most pre-stressing cables applied to the UFC bridges accomplished in Japan were external cables. The advantage of external cables makes it possible to decrease the member thickness and easy to install the cables. On the other hand the disadvantages are that the ultimate resistance bending moment capacity would be reduced by 10 ~20% compared with internal cable and the anchoring structures such as the end UFC blocks would become larger and need more reinforcing. The internal cable system was adopted for this project then the width of the wet joints between the U-shaped pre-cast segments was set up 15cm to joint the internal cable sheaths.

6.2. Experiments for PBL joint and wet joint

6.2.1. Loading test for PBL joint

The shear transfer design method of the PBL joint between the UFC girders and the conventional concrete slabs in longitudinal direction has already been clarified in Horikoshi C-Ramp Bridge. However the resistant bending moment of the twin PBL joint due to the reversed cyclic live load in transverse direction has not been
investigated before. The objective to the loading experiment for the twin PBL joint is to verify the structural performance that 1) the joint exhibits the elastic behaviour for SLS and 2) the joint does not fail at ULS. Three full-scale specimens were manufactured composed of the UFC web and the ordinary PC slab and those specimens were loaded by using the loading test equipment (Fig. 16). The loading steps of the test was as follows; first the design load for SLS was applied three times then the design load for ULS was applied and finally the test specimen was loaded until it failed.

The load versus displacement curve is illustrated in Fig. 17. The test specimen did not crack under the SLS design load of 86kN after applying three times in the alternate cyclic loading. Subsequently, the ULS design load of 103kN still kept the elastic behaviour. The tensile side corner between the slab and the web started to make a gap gradually under a load of about 120kN. The crack width of 0.06mm was observed on the surface of the UFC web at around the load of 250kN. The final bearing capacities of those specimens were about 320–350kN that were about three times the ULS load.

### 6.2.2. UFC wet joint

The first application of UFC wet joint was accomplished in Sakata-Mirai Footbridge project and the design method of this wet joint is reported in the UFC recommendation. This UFC wet joint method has been thereafter applied to many footbridge projects. The UFC wet joint for this project is different from those wet joint on those aspects as shown in Table 3. The shear loading tests using the full-scale specimens of UFC joint were conducted to clarify the shear resistant capacity. Two types of test specimens were prepared; i.e. one without a shear key and one with a shear key (Type-1 and Type-2, respectively). The set up of loading test is just punching the middle part of the wet joint.

The load and shear displacement curves are illustrated in Fig. 18. The cracks of the type-1 specimen developed in the wet joint under the loads of 1,600–1,700kN. Under the load exceeding 2,000kN, number of cracks increased. With the type-2 specimen, the cracks developed from the corner of the shear key under the load of 2,400kN. The number of cracks increased sharply under near the maximum load. The design shear transfer capacity was based on the formula described in the UFC recommendation. One of the important factor β representing the shape of the wet joint shear plane was

![Fig. 16 Loading test equipment](image)

![Fig. 17 Load vs. displacement of PBL joint](image)

![Fig. 18 Load vs. shear displacement of wet joint](image)
recommended 0.4 by the experiment of Sakata-Mirai project. From the test results of this new size wet joint, the factor $\beta$ was derived to be 0.7.

6.3. Fabrication of pre-cast segments

In order to efficiently produce the segments, the UFC material was poured into the mould that was installed upside down. The U-shaped pre-cast segments were manufactured under severe quality control in the factory. Especially the fresh property of the UFC material; i.e. the flow value was carefully managed because the pre-cast segments were produced in summer. Because the material for one segment was rather large volume, it was necessary to stock the mixed batched material in advance therefore it took one hour and half from starting to mix the UFC to finishing to cast it into the mould. Before the consecutive manufacture, the material flow test was conducted in advance in the condition of the atmosphere temperature was 31°C and the material temperature was 33°C. If the flow value was set to 280mm just after mixing the material, it was verified that the flow value may be kept 250mm at the elapsed time was one hour and half. Total 21 pre-cast segments of which maximum weight was designed to be 250kN were conveyed to the construction site by ordinary trailers (Photo 22).

Before erection of the pre-cast segments, the temporary supports and girders were set up on the final position but on the higher level than the final one because there is not enough space upwards of the existing road. The erection of the pre-cast segments was achieved by using the 3,600 kN-capacity crane in the night because of the traffic control (Photo 23). It was easy to adjust the final position of the segments because the gaps of the UFC wet joint can absorb the geometrical errors of the segments. A part of the top cast-in-situ slabs by ordinary concrete and the transverse girder were built in advance to the wet joint (Photo 24). 18 wet joints spots were needed to cast-in-situ UFC of which volume was 0.18m$^3$ per one spot. As the total volume of UFC for wet joint was 3.3m$^3$, two 100-litter mixers were prepared to cast UFC at one time for one wet joint. After cast-in-situ UFC for wet joint, the moulds were removed and the temporary pre-stressing provided, the contact stress on the wet joint to be 0.3N/mm$^2$ by using some of the main PC cables. As the design compressive strength of UFC in the wet joint was 120N/mm$^2$, the warm air with 40–50°C applying heater was ventilated into the girder for four days. After checking the compressive strength to be 160N/mm$^2$ using the test specimens inside the same condition of the girders, the rest of the top slabs were completed and the final pre-stressing process was achieved by the main cables. The final bridge of total 12,000kN-weight was set down to the final level by 1.2m using 6 jacks with 5,000kN capacity.

Fig. 19 Allocation of UFC slabs
7. Mass production of slab in Haneda Runway D

7.1. Introduction of UFC slabs in Haneda Runway D

The construction of Runway D is a national project of the extension of the Haneda International Airport, Tokyo. About one third of Runway D is composed of the pile-elevated platform and the rest of it is composed of the reclaimed land. The reason why such a complex structure was adopted is to avoid blocking the river flow of Tama River. The outside area (blue area in Fig. 19) of the pile-elevated platform is made from pre-cast slabs applying UFC. The area of this part is 192,000m² and about 6,900 pieces of UFC slabs are going to be applied. The reasons for applying UFC to the deck slabs were as follows; 1) Application of the UFC material to the deck slabs made it possible to realize the reduction in self weight of deck slabs by 56% compared to the ordinary concrete slabs. As a result, the weight of the steel jacket and the steel piles could be decreased so the construction cost could be reduced. 2) The UFC material is extremely durable so the maintenance cost must be reduced. 3) In September 2004, the UFC Recommendation was published by JSCE, so it became possible to conduct the objective check of the required performance.

7.2. Structural features of UFC slabs

The principal direction of the slab coincides with the direction of the shorter side and the ribs are set along the principal direction (Fig. 20). The height including the ribs is 250mm and the thinnest thickness of the UFC slab is only 75mm. The averaged slab thickness is about 135mm on the other hand the ordinary concrete slab thickness with compressive strength 50N/mm² becomes 320mm. This means the reduction in dead load compared with the ordinary concrete slab is about 56% (Table 4). One of the featuring points of the UFC slabs is the adoption of the two-directional pre-tensioning newly developed technology in order to efficiently produce the UFC slabs. There was no precedent such a two-directional pre-tensioning method for manufacturing the slabs, however this method was more advantageous from the cost point of view than the usual method, i.e. pre-tensioning in one direction and post-tensioning in one direction.

The design load for each limit state is as follows; 1) SLS (normal condition) --slab dead load + vehicle loading, 2) ULS (emergency condition) --slab dead load + aircraft loading (aircraft deviating from the runway and taxiway). The items checked for SLS are the tensile and compressive stress should be less than -8.0N/mm² and 108N/mm², respectively. Those for ULS are factor of safety against collapse should be greater than one and the stress in pre-stressing steel should be less than yield stress.

Table 4 Comparison of UFC slab and conventional concrete slab

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<tr>
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<th>UFC Slab</th>
<th>Conventional Concrete Slab</th>
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<tbody>
<tr>
<td></td>
<td>W = 97 kN slab</td>
<td>W = 221 kN slab</td>
</tr>
<tr>
<td>Average thickness</td>
<td>=135 mm</td>
<td>=320 mm</td>
</tr>
<tr>
<td>Unit dead load</td>
<td>=3.83 kN/m²</td>
<td>=7.84 kN/m²</td>
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Particularly in the case of aircraft running, it was required that there would be limited damage so that the slabs could be subsequently used. In order to satisfy these conditions, the limiting state was imposed that the stress in the pre-stressing steel should be less than the yield stress when aircraft loading is applied. The UFC slabs are simply supported on the frame of the jacket platform. Three-dimensional FEM analyses including the jacket frame model was applied to calculate the stresses in the slabs and it resulted in that all stress responses and all final limiting states for SLS and ULS satisfied the design items.

7.3. Loading tests using full scale UFC slabs

The loading tests using full scale UFC slabs were conducted in order to confirm that 1) the response stresses and loading bearing capacity could be obtained in accordance with the UFC slab design calculations, 2) the production system such as material, formwork, casting, curing, pre-stressing, de-moulding and fibre orientation to manufacture the UFC slabs could be stably under quality control. The loading tests were carried out on two specimens in order to clarify the variation due to the UFC material and due to manufacturing system. In the tests, the vehicle loading was firstly applied three repeated times and then the aircraft loading was secondary applied two repeated times. Then finally the loading in excess of the aircraft loading was applied (Photo 25). The wheel loading position (six wheels on one UFC slab) was determined in accordance with the wheel layout of the aircraft B777-200ER where the ultimate cross-sectional forces were most severe. The UFC slabs would be actually supported on the flexible steel frames of the steel jacket but the support condition of the loading tests was obliged to be rigid. Therefore the equivalent loads were set to reproduce the cross-sectional forces and stresses calculated during the design.

The loading test results of the load per wheel versus the displacement at the centre of the specimen and three-dimensional FEM analyses taking the UFC material nonlinearity into account are compared with each other in Fig. 21. The displacements under the repetition of vehicle loading due to SLS demonstrated linearity and there were no significant residual displacement. The first crack was observed in the beam in the short direction at the stage when the load slightly exceeded 200kN. There was no significant difference between the
behaviour under the 1st and 3rd loads at the equivalent aircraft load (321kN/wheel). The crack width was only 0.1mm or less under a load (600kN/wheel) in excess of the bending collapse load (501kN/wheel) based on the design calculation. In order to confirm that the variation in the two tests results were within the postulated range, the input data for FEM model were set to be upper and lower limits on the variation in the material quality postulated for the production of the actual slabs. The tests results were within the upper and lower limits FEM results. In addition two test results due to two specimens respectively were almost same therefore it can be concluded that the variation in the slabs is very small.

7.4. Production of the UFC slabs
The production factory for the UFC slabs was newly constructed in Footsu City in Chiba Prefecture. As shown in the factory layout plan (Fig. 22), the factory is equipped with two-directional pre-tensioning abutments which can not be seen in the world and this factory to produce the UFC material must be the world's largest. The roofed area of the factory had a width of 45m and a length of 200m (Photo 26). There are two lines, A and B, and each line had its own production yard, secondary curing tank, pre-stressing steel end processing area, and inspection area. A dedicated UFC batching plant capable of mixing 15m³ per hour was installed within the factory site (Photo 27), and can produce 70m³ of UFC (quantity of 20 slabs) in 5 hours. In the production yard of each line, concrete abutments were provided for pre-tensioning, and 20 sets of formwork were provided within the abutments (Photo 28). The UFC mixed in the batching plant is transported to the production yards, where it is poured into the formwork with the pre-tensioning cables in two directions (Photo 29).

On the following morning after it has been confirmed that the strength exceeded 45N/mm², the pre-tensioning is introduced, the pre-tensioning tendons are cut, and the slabs are transported to the secondary curing tank by a gantry crane. Three secondary curing tanks were provided per line, based on considerations of the production cycle. The UFC slabs transported from the production yards are placed within one of the curing tanks, and steam curing is carried out at 90°C for 48 hours.

8. Conclusions
Starting from Sakata-Mirai Footbridge in 2002, the innovative design and construction technologies such as wet joint, dry joint, PBL joint, non-linear FEM analyses considering UFC tension softening and efficient manufacturing have been developed in Japan. The UFC material has excellent mechanical and durability performance however it requires specialized techniques to apply to bridge construction. Through our experiences on the design and construction of UFC bridges for recent five years, it can be said that the key points of UFC application in Japan must be 1) reduction of dead weight, 2) small ratio of bridge height to span, 3) maintenance cost free and 4) assembly of standardized UFC members. From those points of view, UFC mass production project for Runway D was most efficient and effective. We hope that this UFC technology would help the future development of concrete technology.
9. References