INVESTIGATIONS TARGETING THE APPLICATION OF ULTRA-HIGH DURABLE SLAB IN A HIGHWAY BRIDGE

超高耐久床版の実橋への適用に向けた実験的検討

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道路橋床版の維持管理費用の削減を目的に，超高耐久床版を開発した。超高耐久床版は腐食する可能性のある鉄筋や PC 鋼材を一切使用しないプレキャスト PC 床版である。本稿は超高耐久床版について，疲労性能を含む基本開発を完了後に，中国自動車道蓼野第二橋下り線における床版取替工事への初適用に先立ち，施工性と品質の確保を目的に実施した実験的検討を報告するものである。具体的には，１）長さ11mの実証橋を施工し供用中の性能をモニタリングした結果および２）これまでの実績を超える長さ 100m のアラミド緊張材の挿入，緊張およびグラウト注入の実験結果についてまとめたものである。

キーワード：超高耐久，繊維補強コンクリート，アラミドＦＲＰロッド，床版更新

Targeting to reduce future maintenance of bridge deck slabs, Ultra-high durable slab was developed. It does not utilize any corrosion prone steel members such as reinforcements or prestressing strands. After the basic development, to utilize it in a real highway bridge, additional experiments were done. A 11 m long model bridge was constructed and used for 3 months to investigate the constructability and the structural performance. 100 m long Aramid tendon tensioning and grouting experiment was carried out as well. Developed deck slab was utilized in deck renewal project in Tadeno No.2 bridge in Chugoku expressway.

Key Words: Ultra-high durable, Fiber reinforced concrete, Aramid FRP rods, Deck renewal

1. BACKGROUD

There is an increment of slab deck replacement projects in Japan to secure the serviceability of deteriorated bridges. Ultra-high durable slab (Dura-Slab), a pre-stressed concrete bridge deck slab system, was developed to overcome the corrosion damage of concrete deck slabs aiming to utilize in deck replacements in plate girder bridges. It is made of fiber reinforced concrete, Aramid Fiber Reinforced Polymer (AFRP) rods as prestressing tendons and does not contain any steel components. By eliminating corrosion damage, Dura-Slab is expected to be requiring very little maintenance during the service life which will lead to low carbon emission and lifecycle cost. The new structure is estimated to reduce the carbon emission nearly 40%1).

1.1 DURA-SLAB STRUCTURAL CONFIGURATION

As shown in **Fig. 1**., Dura-Slab is a ribbed slab made of high strength fiber reinforced (polyvinyl alcohol fiber-PVA) concrete with 80 N/mm^2 design compressive strength. Prestressing tendons made of multiple AFRP rods are utilized instead of steel tendons. Dura-Slab is made as pre-tensioned precast panels. Longitudinal prestressing tendons are installed and post-tensioned at the site to integrate the precast panels in the longitudinal direction.

Dura-Slab features a fully non-metal structure by removing all corrosion prone elements. Longitudinal AFRP tendons are bond anchored eliminating the conventional steel anchorage components2). Reinforcement near the anchor, consists of GFRP (Glass Fiber Reinforced Polymer) rods.

The precast slab panel joint in Dura-Slab is shown in **Fig.1.**

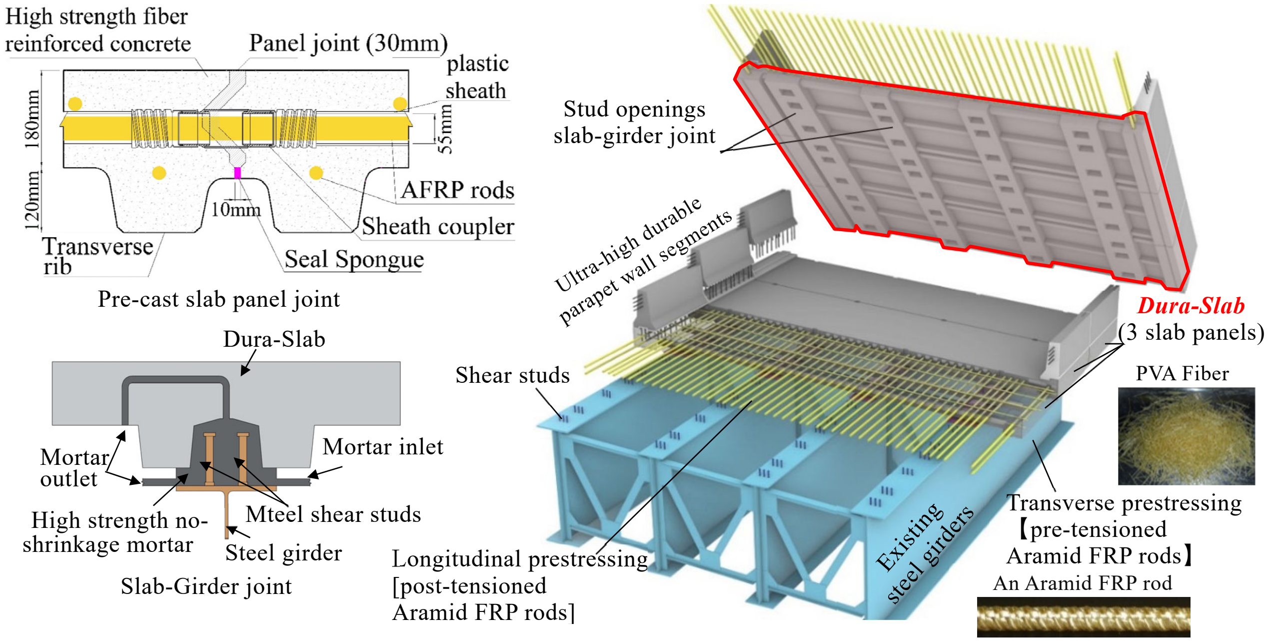


Fig. 1. Components of Ultra-high durable slab and structural arrangement in a slab deck renewal project

By introducing longitudinal prestressing with a transverse ribbed structure, the panel joint was made smaller compared to a conventional joint. Elimination of reinforcement allows significant time and labour saving in joint construction.

Conventionally, precast concrete panels are connected to the steel girders using shear studs. Stud openings which penetrate the full slab thickness are provided. In addition, bolt openings are provided to adjust slab level during placement. Steel anchors are used in the top slab surface for panel lifting. Girder-slab joint of Dura-Slab shown in **Fig. 1.** is constructed by injecting high strength low shrinkage mortar from bottom of the slab avoiding slab penetrating openings. Level adjustment bolt holes were not provided, and ceramic inserts were set in the sides next to slab keys for panel lifting.

This paper summarizes the experimental studies which were carried out targeting the application of the Dura-Slab in Tadeno No.2 bridge located in Shimane prefecture.

2. DEVELOPMENT OF DURA-SLAB

After designing the main structural form of the slab, overall slab performance including the panel joint was verified with a real scale wheel load running test3). Thereafter, the stud connection of the slab with steel plate girders was investigated with a series of stud pushout tests4). It was verified that the stud connection without additional reinforcement around the slab-girder joint was possible to be utilized. Additional experiments that were carried out targeting the application of the Dura-Slab is explained next.

2.1 CONSTRUCTION AND LOAD TEST OF A REAL SCALE MODEL BRIDGE

2.1.1 OBJECTIVES

A real scale model bridge was constructed at an entrance to a pre-cast concrete factory to verify the constructability of the Dura-Slab as well as structural performance.

Three major constructability related objectives were investigated. First, the construction of precast slab panel joint. The deck slab panel joint is only about 30 mm as shown in **Fig. 1.** However, it is necessary to provide the longitudinal tendon sheath couplers at each panel joint. Constructability of the joint including the sheath coupler was investigated. Second, Panel level adjustment method without deck pentrating level adjustment bolts. And the third was the mortar injection method to the slab-girder joint form the bottom side of the slab as shown slab-girder joint in **Fig. 1.**

To confirm the structural safety, static load tests were carried out just after construction and after putting the bridge to service for 3 months. During the service, continous monitoring of the structre was carried out targeting slab deformation, strain and so on.

2.1.2 CONSTRUCTION OF THE MODEL BRIDGE

Model bridge dimensions are shown in **Fig. 2.** Typical panel transvesrse tendons consisted of 4 AFRP rods with diameter 7.4 mm. Each precast deck panel consisted of 6 transverse pre-tensioned AFRP tendons or equal. longitudinal

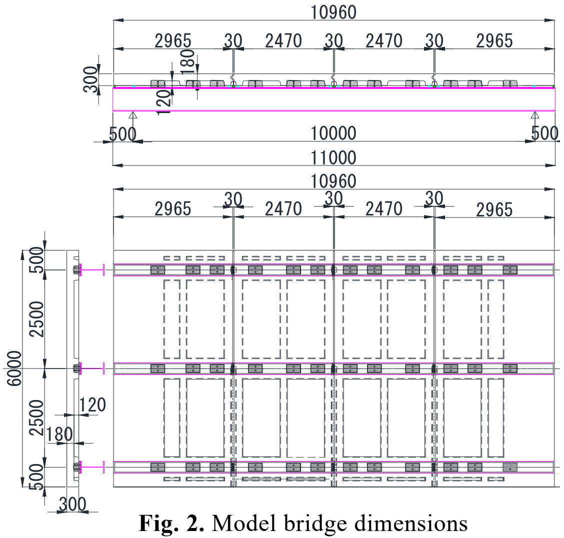


Fig. 2. Model bridge dimensions

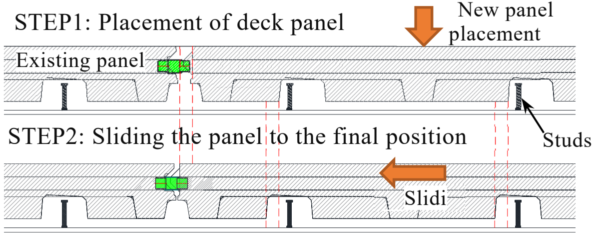


Fig. 3. Slab panel joint construction

(a) Panel placement (b) Joint mortar injection

(c) Completed model bridge during load test

Fig. 4. Construction of the model bridge

prestressing consisted of 14 tendons where each tendon consisted of 9 AFRP rods with diameter 7.4 mm.

Construction method of the precast slab panel joint is shown in **Fig. 3.** To make the longitudinal sheath joint, the new panel should be placed on the girder and slide towards the panel placed before. Studs are welded in advance to the panel placement and the stud opening size was decided considering the sliding length. Panel placement is shown in **Fig. 4.(a).**

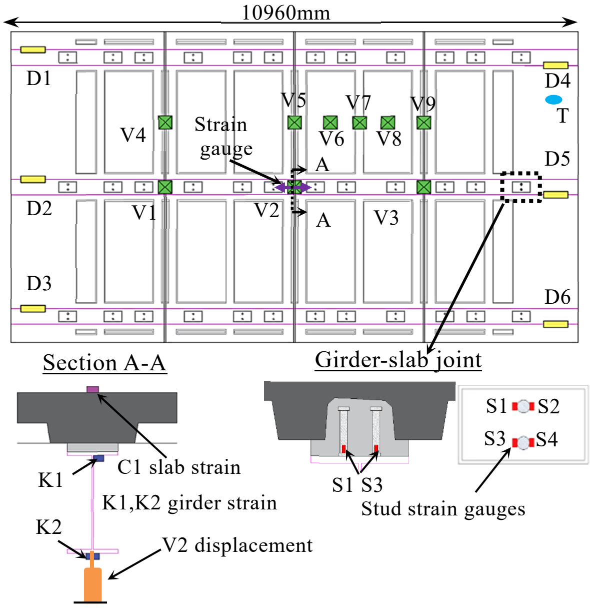


Fig. 5. Model bridge sensor locations

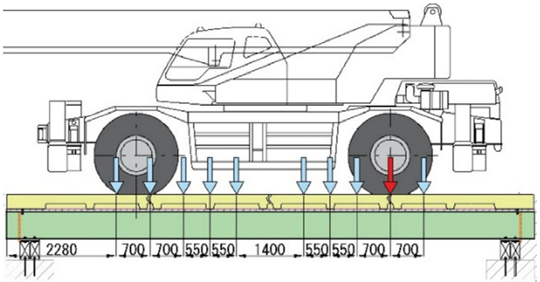


Fig. 6. Loading locations in static load test

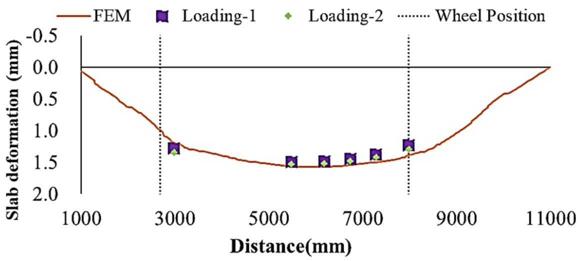
Contrary to the conventional method of providing slab penetrating bolt holes, level adjustment of Dura-Slab was done from bottom side of the slab utilzing plasitc plates. Construction could be done without any major difficulties.

Mortar injection to the slab-girder joint is shown in **Fig. 4.(b).** It could be successfully done with a mortar injection hand pump from bottom side of the slab. Completed model bridge is shown in **Fig. 4.(c).**

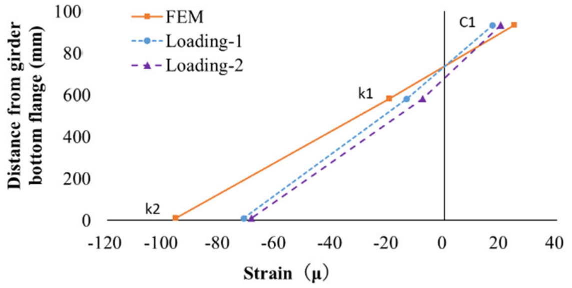
2.1.3 STATIC LOAD TEST OF THE MODEL BRIDGE

Major structural measurement locations during the load test are shown in **Fig. 5.** Measurements included vertical displacement of middle steel girder (V1~V3), vertical displacement of the slab at the center of the girders (V4~V9), relative displacement of the deck slab and the steel griders in the bridge axial direction(D1~D6), stud strain at the end of the middle girder (S1~S4) and the longitudinal strains at section A-A which includes the slab top surface strain (C1), girder top and bottom flange strains(K1,K2).

Static load tests were carried out utilizing a 65 t rough terrain crane (front wheel load 19,710 kg, rear wheel load 19,885 kg). During the service time of 3 months, the number of vehicles pass across the model bridge was calculated to be

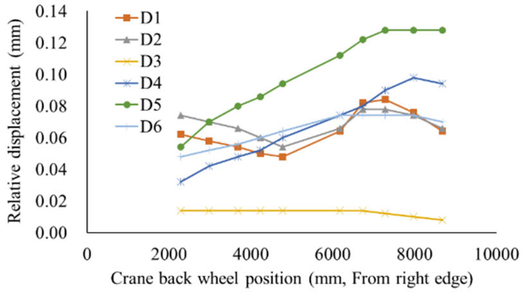


(a) Slab vertical deformation

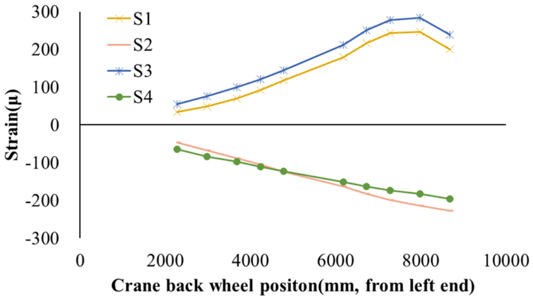


(b) Bridge cross sectional strain distribution

Fig. 7. Static load test results (loading at span center)

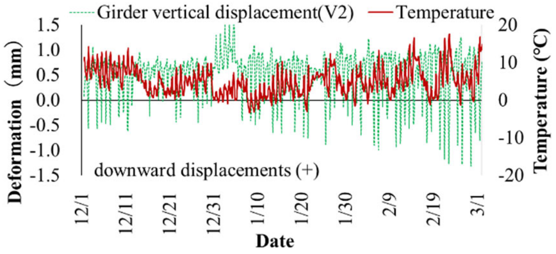


(a) Slab-Girder relative deformation

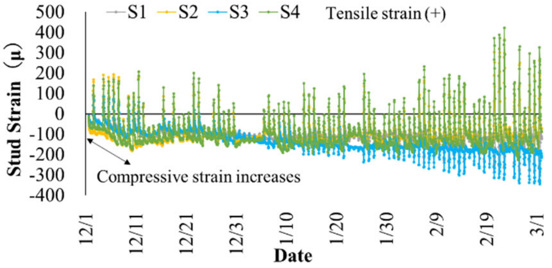


(b) Stud strain

Fig. 8. Static load test results after in service for 3 months



(a) Vertical displacement of girder and temperature



(b) Stud strain variation

Fig.9. Model bridge structural monitoring data

1170 where most were dump trucks. The crane was placed in the center line at the width of the bridge and in the longitudinal direction, 10 loading points were defined based on the rear wheel location of the crane in such a way to include the top of the transverse rib, panel joint and normal slab section as shown in **Fig. 6.** The behaviour of the bridge during the static load test was estimated using three-dimensional liner finite element analysis in advance.

The static load test results are shown in **Fig. 7.** when the crane in the middle of the bridge span. The vertical deformation of the slab matched the FEM analysis output as shown in **Fig. 7.(a).** Moreover, the experimental difference of the deformations before and after the model bridge was put to use was not significant. The strain variation in the vertical direction of section A-A is shown in **Fig.7.(b).** From experimenal strain varitaion in the vertical direction, it can be thought that the grider-slab system behaves as a composite system. The strain variation was not significantly different for two loading tests as well.

The girder-slab relative displacements measured by D1~D6 and the stud strain in the second loading test is shown in **Fig. 8.** The relative displacement affects the performance of the girder-slab joint. The maximum relative displacement was 0.13 mm at D5 while the maximum stud strain was 284 μ. However, the relative displacment and stud yield strain observed at the joint yeilding during the stud pushout experiment[4)] was 0.59 mm and 1,390 μ respctively. With saftety factor above 4.5 with respcitive to the relative displacement and yield strain, the joint safety was considered to be sufficient.

2.2 MODEL BRIDGE MONITORING

Continous measurements of all the installed senors was done once every hour during the model bridge was in service. The purpose of the monitoring was to observe whether any unexpected, abnormal structrual behaviours occur during the continous use.

Structural monitorng data of the model bridge is shown in **Fig. 9.** The vertical displacement of the middle steel grider is shown in **Fig. 9.(a).** The measurement is largely affected by the ambiant temperature variation. Besides that, there was no out of ordinary variations of the girder displacement. Stud strain variation is shown in **Fig. 9.(b)**, at the initial stage an increment of compressive strain was observed. This was thought to be due to the effect of the plasitc plates used for panel levelling as shown in **Fig.10.** Initially the dead load of the deck is supported by the plastic plates, however, gradually with the vehicular loading, the load might tranfer to joint mortar excertin a compressive force on the studs. Both the grider displacement and stud strain measurements variation is increased gradually, this was due to the gradual temperature variation increment as shown in **Fig. 9.(a)**.

2.3 REAL LENGTH TENDON INSERTION AND CONSTRUCTION EXPERIMENT

Tadeno No.2 bridge, where the Dura-Slab was first applied, is nearly a 100 m long bridge. One of the issues at the time was the lack of experience of using aramid AFRP internal tendons in a 100 m long structure. A 100 m long specimen was created as shown in **Fig. 11.(a)** and experimented targeting the tendon insertion, tensioning, and grouting.

The specimen was constructed using 2.4 m long blocks as in the precast panel width of the actual bridge. In addition, the horizontal curvature of the actual bridge was also considered.

Longitudinal AFRP tendons consisted of 9 AFRP rods with diameter 7.4 mm. As the 100 m long tendon was only 56.7kg in total, insertion could be done with manpower as shown in **Fig. 11.(b).**

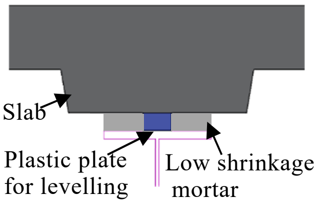


Fig.10. Slab panel levelling

Friction during the tensioning was not significant. Therefore, tendon tensioning was carried out similar to an external tendon based on the load and the tendon extension length. Tendon tensioning is shown in **Fig. 11.(c).** Tensioned tendon was kept a day in tensioned condition and de-tensioned to observe the tendon condition. The tendons were not damaged due to the friction or due to the horizontal curvature of the specimen.

To facilitate the bond anchoring of tendons, a bond length of 2 m from the slab end was sheath-free (**Fig. 13.**). High strength mortar was injected at the bond length and normal tendon-grout was injected in the rest. In the experiment, first bond mortar was injected in the lower end of (lower height) the specimen, then tendon grout was injected in middle and finally mortar was injected at the second end. The inlet outlet hose arrangement and mortar injection procedure at the last end is shown in **Fig. 12.** The mortar injected in the middle was flown to the bond segment as shown in top part of **Fig. 12.** When the mortar is injected, the weight of the out flowing mortar was measured to verify that the grout is fully removed from the duct. The specimen was cut in longitudinal direction after strength gain to observe the injection condition. It was observed that the grout has remained in the bond section.

Mortar and grout injection was improved as in **Fig. 13.** First, tendon was inserted, and temporary anchor used for tensioning was attached. Then, tensioning was carried out. As in step 2, a mortar barrier was created with high viscous mortar. And then, high strength low shrinkage mortar was injected into the bond zone followed by normal grout injection to the opposite side. After leaving for enough time to gain the mortar strength, the excess AFRP was cut off and the temporary anchor was removed as shown in step 4.

(a) 100 m long specimen (b) AFRP tendon insertion by manpower (c) Tendon tensioning

Fig. 11. 100m long tendon insertion, grouting, and tensioning experiment

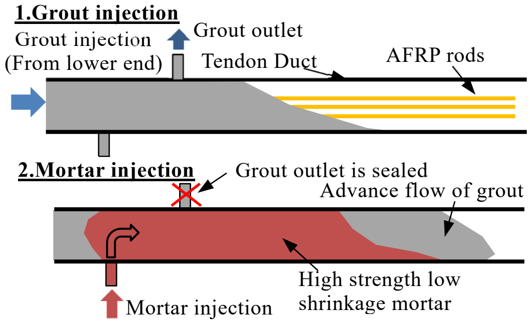


Fig. 12. Mortar injection at the high end of the specimen

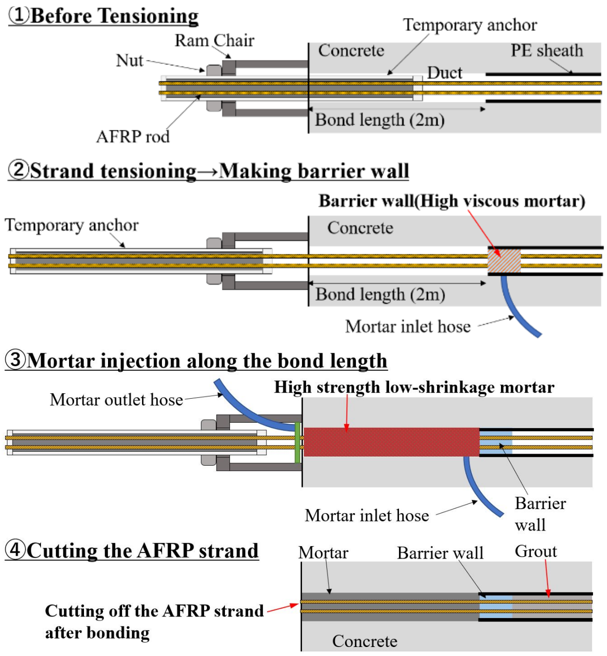


Fig. 13. Mortar and grout injection for longitudinal tendons

3. CONCLUSIONS

Targeting a lower lifecycle cost by reduced maintenance, Ultra-high durable deck slab was developed. In addition to the major structural development, additional experiments were conducted targeting the application of the structure in a real highway bridge.

A model bridge was constructed to investigate the constructability of the dura-slab. Construction of the deck panel joint was successfully done by panel sliding after placing on the girder to make the longitudinal sheath coupling. Panel levelling was done with plastic plates put on the steel girder. finally, the high strength low shrinkage mortar injection for the girder-slab joint could be completed adequately.

To investigate the structural performance of the model bridge, two static load tests were carried out just after construction and after using the bridge for 3 months. The difference of slab deformations and strains before and after did not differ confirming the sound structural performance. Continuous monitoring was carried out of the model bridge during the service. Monitoring data including girder deformation and stud strain was largely affected by the ambient temperature. However, no abnormal structural phenomenon was observed.

A real scale longitudinal tendon insertion, tensioning, and grouting experiment were carried out. Tendons did not damage during construction. Mortar and grout injection method was improved based on the experimental observations.

Based on the experimental results, Dura-Slab was applied in Tadeno No.2 bridge successfully and in service from November 2021.

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