PACIFIC CONCRETE CONFERENCE AND TRADE EXHIBITION
SHERATON HOTEL, AUCKLAND, NEW ZEALAND
Tuesday 8 to Friday 11 November 1988

CONFERENCE PROGRAMME AND SUMMARIES OF TECHNICAL PAPERS
Friday 11 November

2.00-3.30pm Concurrent Technical Sessions: 10

10A Arawa Room
Bridges

35. Design and Construction of a New Steel-Concrete Composite Slab Bridge
   *Department of Civil Engineering, Kyushu University, Japan

36. Cracking Strength and Deflection of Partially Prestressed Concrete Hollow Slab Bridge
   *Akita University, Japan

37. Design of a Curved 414m Long Continuous Concrete Railway Bridge to Resist Earthquake Motions
   – Jury RD, Hollings JP, Catley TJ
   Beca Carter Hollings & Ferner Ltd, Wellington

10B Tiri Room
Durability and Shrinkage

45. Prevention of Shrinkage Cracking in Concrete by Incorporation of Admixtures
   – *Shoya M, Sugita S, *Sugawara T
   *Hachinohe Institute of Technology

53. Corrosion of Reinforcing Steel in Concrete and its Prevention by Cathodic Protection
   – *Mussinelli G.L., Tettamenti M., Irwin R.W., Lawson M.J.
   *Oronzio de Nora, Switzerland

54. Development of New Corrosion Protection Prestressing Tendons and their Use in Bonded and Unbonded Prestressed Concrete Members
   – Muguruma H, Watanabe F, Nishiya M
   Kyoto University, Japan

4.00-5.00pm Plenary Session – Arawa Room

Panel Discussion:
Panel Members – the four Keynote Speakers

5.00pm Conference Closure – Arawa Room
A new steel-concrete composite slab system with pyramidal shear connectors has been developed by the authors. In order to investigate the structural characteristics of the proposed connectors and composite slab system, the push-out test and the flexural test were carried out, respectively. The first application of this system was carried out on the skew composite hollow slab bridge (Haruda Ryokudo Bridge) in Japan in the year 1987. The field test was also carried out to confirm the safety and to clarify the practical design method of such a bridge.

INTRODUCTION

Steel and reinforced (or prestressed) concrete elements are frequently combined into a structural system, because this combination generally results in greater economy and safety than could be achieved by other materials alone. A new steel-concrete composite slab system shown in Fig.1, which has been developed recently by the authors, is also a kind of them. It is composed of concrete and steel deck which consists of bottom plate, upper reinforcement and pyramidal shear connectors. The pyramidal shear connector, which may be hereafter called TSC connector, is expected to have higher horizontal shear resistance than the equivalent stud connector. The composite slab system with such connectors, called TSC composite slab, may retain some superior structural characteristics as well as advantages in construction and economy.

TSC composite slab system has been first applied to Haruda Ryokudo Bridge, to carry the National Highway Route 3 near Fukuoka in the Kyushu Island of Japan in 1987. The bridge which was constructed by Fukuoka National Highway

Figure 1 Composite slab system with pyramidal shear connectors
office of the Ministry of Construction, is a skew composite hollow slab bridge having a length of 11 m.

This paper describes the structural characteristics of TSC composite slab system and the design and construction of the above bridge. First, the horizontal shear resistance of TSC connectors is discussed comparing with the equivalent stud connector by the push-out tests. Next, the static flexural tests on TSC beams and the corresponding RC beams are carried out in order to clarify the flexural behavior of TSC composite slab system. Finally, the construction of Haruda Ryokudo Bridge and its analysis for the practical design are presented and the suitability of the analysis in comparison with the results of the field tests is discussed.

STRUCTURAL CHARACTERISTICS OF TSC COMPOSITE SLAB

Push-out tests for shear connectors

Specimen and test procedure

Three types of push-out specimens were tested in order to investigate the horizontal shear resistance for connectors. As shown in Fig.2, Type-B is the pyramidal shear connector, while Type-C is the plane truss-shaped connector which is perpendicular to the shearing direction. Type-A is a corresponding stud connector. In comparison, the height and cross-sectional areas of connectors and welding length were taken nearly equal in all the specimens. Dimensions of the specimen are shown in Fig.3; each specimen consisted of a wide flange beam, 200 x 200 mm, 390 mm long with concrete slabs attached to both flanges of the beam. Slab concrete was cast perpendicular to the interface between steel and concrete so that both slabs could be cast concrete from the same direction as the actual structures. The concrete strength for each specimen was kept nearly equal to 26 MPa. SS41 steel was used with a minimum yield strength of 245 MPa as specified in the Japanese Industrial Standards (JIS).

All specimens were tested in a 980 kN (100 tf) universal testing machine. Load was applied repeatedly and the maximum load in each repetition was increased in increments of 19.6 to 49.0 kN. After the slip between concrete slabs and steel beam exceeded approximately 0.5 mm, the load was increased gradually until the specimen failed. At each loading stage, slip between slab and beam, and strains in steel and concrete were measured.

![Figure 2 Details of shear connectors](image)

![Figure 3 Push-out specimen and arrangement](image)
Test results

Fig. 4 shows the maximum load per flange-slip relationships in each repetition for tested connectors. As shown in this figure, the slip was less under lower loads due to the presence of the natural bond between slab and beam. The pyramidal and plane truss-shaped connectors provided considerably higher resistance to slip than the stud in all of the loading stages.

Test results are presented in Table 1. The allowable shearing load per flange for the stud connector $Q_a (tf)$ is prescribed in the Japanese Specifications for Highway Bridges[1] as follows;

$$
Q_a = 30n d^2 f_{ck}^{1/2} \quad (H/d \geq 5.5) \\
Q_a = 5.5n d H f_{ck}^{1/2} \quad (H/d < 5.5)
$$

where $n$:number of studs per flange, $H$:height of stud(cm), $d$:diameter of stud(cm), $f_{ck}$:design strength of concrete(kgf/cm$^2$).

The allowable shearing loads per flange $Q_a(tf)$ for pyramidal and plane truss-shaped connectors can be calculated from the following equation, with reference to the hoop reinforcing connector;

$$
Q_a = n f_{sa} A_s \cos \alpha \cos \beta
$$

where $f_{sa}$:allowable tensile stress of connector(kgf/cm$^2$), $A_s$:cross-sectional area of connector(cm$^2$), $\alpha$, $\beta$:angle of inclination for connector.

The critical load $Q_c$ was defined as the load per flange in which the residual interface slip was equal to 0.075 mm[4], and the ultimate load $Q_u$ was obtained from the maximum load per flange, respectively. The slip constant $k$ was determined by the secant modulus at the critical load from the load-slip curves.

Comparing values in Table 1, it is evident that the pyramidal connectors exhibited consistently higher values of the critical load as well as the ultimate load than the stud. This was caused by the help of the concrete confined in the space of connectors and beam. The effect of interactive resistance will increase the strength of the connector. Eq.(1) for the stud gives the safety factor of 3.2 and 7.8 for the critical and ultimate loads, respectively, which has been proved experimentally by previous investigations. The pyramidal and plane truss-shaped connectors has the critical

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Allowable load $Q_a$(kN)</th>
<th>Critical load $Q_c$(kN)</th>
<th>Ultimate load $Q_u$(kN)</th>
<th>$Q_c/Q_a$</th>
<th>$Q_u/Q_a$</th>
<th>Slip constant $k$ ($\times 10^3$kN/cm)</th>
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</thead>
<tbody>
<tr>
<td>A</td>
<td>32.3</td>
<td>104</td>
<td>253</td>
<td>3.22</td>
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<td>8.82</td>
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<td>B</td>
<td>44.1</td>
<td>181</td>
<td>426</td>
<td>4.10</td>
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<td>21.7</td>
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<tr>
<td>C</td>
<td>44.1</td>
<td>178</td>
<td>379</td>
<td>4.04</td>
<td>8.59</td>
<td>16.5</td>
</tr>
</tbody>
</table>

Table 1 Results of push-out test
and ultimate loads approximately 4 times and 8.6 to 9.7 times the design load obtained from Eq.(2), respectively.

**Flexural tests for composite beams**

**Specimen and test procedure**
Static flexural tests were carried out on two TSC composite beams and two corresponding RC beams, to clearly identify the difference between their structural behaviors[3]. Fig.5 shows the details of specimens. One is a TSC composite beam, which consists of bottom plate of 4.5 mm thickness, Φ10 pyramidal deformed steel bars at the pitch of 16 cm in the longitudinal direction, Φ19 compressive deformed bars and concrete. The pyramidal bars are expected to resist not only the horizontal interface shear but also the diagonal tension developed in concrete. RC beam had the same amount of effective depth and reinforcement as TSC beam, so that the test beams were designed to have equal ultimate flexural and shear strength and flexural rigidity. The concrete strength for the specimen was kept nearly 57 MPa. The mild steels used were SM50 plate, SD30 and SD35 bars with minimum yield strength of 323, 294 and 343 MPa as specified in the JIS, respectively.

The beams were simply supported with a span of 1.8 m. At first, a concentrated load was applied through a spreader beam at the midspan and gradually increased up to failure for each of the TSC and RC beams. For each of the other beams, the concentrated loading and unloading were repeated alternately in increment of multiplies integrally as the deflection at the yield point obtained from the gradually incremental loading.

![Figure 5 Details of test beams](image)

**Test results**
Crack patterns of the test beams at the yield point are shown in Fig.6. It indicates the similar tendency of cracking between TSC and RC beams; flexural cracks were produced from the bottom surface and a relatively sufficient crack distribution was also observed, while diagonal tension cracks did not occur in TSC composite beam before failure. This proves that the pyramidal shear con-

![Figure 6 Crack patterns of test beams](image)
nectar also plays the role of the diagonal tensile reinforcement.

Fig.7 shows load-deflection relationships at the midspan of the beams. Both beams retained equal flexural rigidity and reached the yield point at the midspan deflection of approximately 10 mm. The deflections at maximum load and at failure were measured 26 mm, 178 mm for TSC beam, and 21 mm, 96 mm for RC beam, respectively.

Table 2 shows the test results with respect to carrying capacity for test beams. TSC beam as well as RC beam has maximum strength approximately 20 % more than the calculated values, which is indicated by the value in the parenthesis, obtained from the Japanese Standard Specification for Design of Concrete[2]. However, TSC beam retains ductility factor approximately two times that of RC beam.

<table>
<thead>
<tr>
<th></th>
<th>TSC beam</th>
<th>RC beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load pattern</td>
<td>Increment</td>
<td>Repeat</td>
</tr>
<tr>
<td>Maximum load</td>
<td>203 kN</td>
<td>200 kN</td>
</tr>
<tr>
<td></td>
<td>(167 kN)</td>
<td>(148 kN)</td>
</tr>
<tr>
<td>Load at failure</td>
<td>190 kN</td>
<td>184 kN</td>
</tr>
<tr>
<td></td>
<td>160 kN</td>
<td>164 kN</td>
</tr>
<tr>
<td>Ductility factor</td>
<td>16.7</td>
<td>16.2</td>
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<td></td>
<td>9.8</td>
<td>9.6</td>
</tr>
</tbody>
</table>

CONSTRUCTION OF TSC COMPOSITE SLAB BRIDGE

Outline of Haruda Ryokudo Bridge

Haruda Ryokudo Bridge, to carry the National Highway Route 3 near Fukuoka in the Kyushu Island, was the first TSC composite slab bridge in Japan. The bridge is the skew hollow slab bridge having a length of 11 m and a skew angle of 62 degrees, composed of a roadway of 5.5 m and a footway of 4.5 m in effective width, respectively. Fig.8 shows an overall view of the roadway.

TSC composite slab system was selected for the overbridge, because the
bridge required a depth less than 40 cm due to the restrictions by the clear headway under the bridge. A comparative estimate showed TSC composite structure to be approximately 30% more cheap than the steel plate deck. The roadway has a slab depth of 38 cm cast concrete on the bottom plate of 9 mm thickness, and the footway has a 30 cm depth of slab cast on 6 mm thick plate. The span/depth ratio is approximately 27. The design live load for roadway was 196 kN (20 tf) trucks of TL-20 prescribed in the Japanese Specifications for Highway Bridges.

Fabrication and construction

The thin bottom plate utilized for permanent form as well as tensile reinforcement was fabricated in three segments in the longitudinal direction. Flat bars as shear connectors and diagonal reinforcement and upper deformed reinforcing bars were spliced with the bottom plate by butt-welds, which resulted to form the space truss system. Then bottom plate became distorted irregularly during the welding process, but such distortions were eliminated by heat straightening. The plates and reinforcing bars used, were SS41 and SD30, with minimum yield strength of 235 MPa and 294 MPa, respectively, as specified in the JIS.

Three steel deck segments could be easily transported to the construction site and erected by the large block erection method without any staging on account of their high loading capacity and flexural rigidity, as shown in Photo.1. After the butt-weld splices were made for the
steel deck in field, the concrete was cast without any formwork and staging. The used concrete had design strength of 34 MPa, slump of 8 cm and a maximum size of coarse aggregate of 25 mm. With the purpose of decreasing the concrete slab weight, styrofoams of 15 cm x 15 cm in cross-section were set in concrete as shown in Fig.9 and Photo.2, which resulted to be equivalent to 4.5 cm thick concrete. Construction using TSC composite slab system allowed a considerably faster erection schedule. The speed of erection in the field was half a day. Consequently, the bridge was completed in less than three months, including all of the works in the field.

![Photo.1 Erection of steel deck](image)

![Photo.2 Detail of steel deck](image)

**ANALYTICAL PROCEDURE FOR DESIGN**

The design of Haruda Ryokudo Bridge was based on the following assumptions; (a)Before concrete hardens, the dead loads of slab except curb and pavement are carried by the steel deck, consisting of bottom plate, upper reinforcement and pyramidal shear connectors. (b)After concrete hardens, concrete and steel deck are so interconnected with pyramidal shear connectors that the composite action is provided for live loads and dead loads of curb and pavement.

Therefore, in the design for this bridge, safeties of the steel deck before concrete hardens and of the steel-concrete composite slab after concrete hardens are examined. This chapter briefly explains the analytical procedure for design.

**Before Concrete Hardens**

The steel deck was modeled by an orthogonal grid shown in Fig.10. A generalized slope-deflection procedure was used in the analysis and the rigidity of an individual member of the grid was estimated as follows; for the rigidity of a longitudinal member, a row of space truss system in longitudinal direction is considered first (Fig.11). Its flexural rigidity is supposedly provided by upper reinforcement and bottom plate, hence it is easily determined.

Torsional rigidity is approximately calculated by the following simplified procedure. Let a row of space truss system be idealized as a quasi-trigonal prism member. Then the equivalent thickness $t_{eq}$ of the plates of an oblique side can be given by;

$$t_{eq} = \frac{(E/G) A (ab/d^3)}{3} \quad (3)$$

where $A$:cross-sectional area of diagonal member for pyramidal shear connec-
tor, a: a half of the spacing of diagonal member, b: width of oblique side, 
d: length of diagonal member, E: Young's modulus, G: shear modulus of elas-
ticity.
Using $t_{eq}$ obtained by Eq.(3), the torsional rigidity of a quasi-trigonal 
prism member may be derived as:

$$J = \frac{4F^2}{(2b/t_{eq} + B/t_p)} \quad (4)$$

where $F = Bh/2$, $B$: width of space truss in transverse direction, $h$: height of 
shear connector, $t_p$: thickness of bottom plate.

Each rigidity is multiplied by a ratio of effective width of the 
longitudinal member of grid to the width of space truss and the products are 
treated as the rigidity of member. In the transverse member, each apex of 
pyramidal shear connector is not so interconnected that both flexural and 
torsional rigidities are given by only bottom plate.

**After Concrete Hardens**

For the analysis of steel-concrete composite slab, the orthogonal grid with 
the same geometrical form as the above grid was used. The rigidity of each 
member was also estimated in a similar manner.

First, the rigidity of a lon-
gitudinal member is calculated for 
one closed section (Fig.12); the 
flexural rigidity is estimated for 
cracked section consisting of 
concrete in compression, upper 
reinforcement and bottom plate. 
The torsional rigidity is esti-
mated by Bredt's formula with 
respect to concrete and bottom 
plate except diagonal members and 
upper reinforcement. Next, these 
values are adjusted for the effec-
tive width of longitudinal member 
of grid. The rigidity of a trans-
verse member is calculated for the section consisting of concrete in 
compression and bottom plate (Fig.12). These rigidities are transformed into 
the values for steel by using the ratio $n$ defined as follows;

For the flexural rigidity: $n = E_s/E_c$

For the torsional rigidity: $n = G_s/G_c$

where the subscripts $s$ and $c$ indicate steel and concrete, respectively.
FIELD TEST PROCEDURE

The field tests were carried out for two states before and after composite behavior took place. First, the steel deck subjected to the weight of uncurved concrete was tested in order to investigate the behavior of the steel deck system by dead load. Next, after concrete hardened, the loading test of the steel-concrete composite slab was carried out by using two trucks with approximately 196kN weight (rear axle weight was 143kN). The loading patterns of trucks were applied as follows. As shown in Fig.13, one truck was placed at point a1, a2, - - , c3 in order, in which number a1 to c3 indicates the position of the center line of rear wheel. Two trucks were also applied on a-line and c-line, in which the loading pattern was varied from a1-c1 to a3-c3.

At each loading test, deflections and strains in steel and concrete were measured by using deflection meters of cantilever type with precision of 1/100 mm and 1/200 mm and electrical resistance strain gages, respectively.

![Figure 13 Truck arrangement for loading test](image)

COMPARISON OF THE TEST RESULTS WITH CALCULATED RESULTS

**Before Concrete Hardens**

Upper reinforcement and bottom plate can be considered equivalent to chords of truss like the flanges of a beam, which resist the tensile and compressive force induced by bending. Fig.14 shows the test results of strain variations of upper reinforcement and bottom plate as well as deflection along A-line. In the figure, the calculated results are also shown, including the calculated ones by the plane truss model that is the projection of a row of space truss system. As is seen in Fig.14, the difference between calculated results obtained by plane truss model and by orthogonal grid is small, and the test results also agree with both calculated ones. This implies that the plane truss model may be applied to the analysis of flexural members such as upper reinforcement and bottom plate in the steel deck system.

On the other hand, diagonal members for pyramidal shear connectors as well as web members of a truss provide the required shear capacity. In this case, the resultant axial force is induced in the diagonal member by torsional moment in addition to shear force. That is, the axial stresses in four diagonal members connected at a joint may be illustrated in Fig.15, in which $\sigma_Q$ and $\sigma_T$ indicate the axial stresses by shear force and torsional moment, respectively.
Fig.16(a) and (b) show axial strains by shear force and by torsional moment, respectively. It is clear from Fig.16(a) that the test results agree well with the calculated ones and the maximum value occurs at the adjacent support. Another observation is made in Fig.16(b) that the effect by torsional moment gradually increases from the support towards the center of the bridge, and the maximum value at the center of bridge is nearly equal to a half of that by shear force. The axial strain of diagonal member induced by torsional moment was estimated from shear stress that occurred in the plate of an oblique side of a quasi-trigonal prism member. The calculated value agrees qualitatively with the test ones, but may be quantitatively insufficient. It is evident that more work using different approaches must be necessary. Fig.16(c) shows the axial strain for each member. The resultant strain by torsional moment and shear force is greater than that only by shear force as shown in this figure. This implies that the application of plane truss model for the analysis of shearing member in the skew steel deck in which torsional moment in addition to shear force produces, may possibly raise a problem.

![Figure 14 Behavior of Steel deck](image)

![Figure 15 Axial stress of diagonal member](image)

![Figure 16 Axial strain of diagonal member](image)
After concrete hardens

A series of loading tests were carried out as mentioned above. Here, we describe a typical example of loading pattern a2-c2 because of limitation of the available space.

Fig.17 shows shear force and torsional moment which are resisted by the member with a closed-section. It can be seen from this figure that the diagonal member insignificantly contributes to resistance of those forces. Therefore, the diagonal members for pyramidal shear connectors can be ignored in the calculation of the rigidity of steel-concrete composite slab after concrete hardens, although they play an important role in the state of steel deck before concrete hardens.

Fig.18 shows deflection and strains in upper reinforcement and bottom plate along B-line. In the usual design of bridges, stiffness of a curb should not be considered. But the curb of this slab bridge which is the shaded portion in Fig.8, is large and continuous so that it acts like a girder. The dotted line and the solid line in Fig.18 show calculated results of the slab without the curb and with one, respectively. The test results are significantly smaller than the calculated ones without the curb, but agree well with the calculated ones with the curb. This indicates that the effect of the curb cannot be ignored when such a continuous and large curb is built on the composite slab.

A new composite slab system with pyramidal shear connectors has been developed. A series of push-out tests for connectors and flexural tests for composite beams were carried out to clarify their structural characteristics. The field test of Haruda Ryokudo Bridge, which was the first composite slab bridge using TSC composite slab system in Japan, was also conducted to confirm the safety and to verify the suitability of the analytical procedure.
From the results, it can be concluded that:

(1) The pyramidal shear connector retains considerably higher shear resistance to the slip than the stud connector with equivalent dimensions.

(2) The TSC composite beam has ductility factor approximately two times that for the equivalent RC beam.

(3) In this bridge, the dead load of slab except curb before concrete hardens can be sufficiently carried by the steel deck without any staging. On the other hand, the live load and the other dead load can be carried by composite slab consisting of concrete and the steel deck.

(4) For the analysis of such a skew slab before concrete hardens, the plane truss model can apply to flexural members, but cannot hold for diagonal members due to torsional moment. Therefore, the orthogonal grid model that can take account of torsion, should be used in analysis of the skew steel deck.

(5) In application of the orthogonal grid model to the skew composite slab bridge with a continuous and large curb, the effect of diagonal members can be ignored but the curb may be considered for the design.

ACKNOWLEDGMENT

The field test and its discussion in this paper was supported by the Technical Committee of Haruda Ryokudo Bridge, organized in the Fukuoka Highway Office, Ministry of Construction, in 1987.

REFERENCES

3. Takahashi, Y. et. al. 1988. Static and repeated flexural behavior of TSC beams using high-strength concrete. 15th Annual Conference of Kanto Branch of Japan Society of Civil Engineers.