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# PROPOSAL FOR NEW STRUCTURE TAKING ADVANTAGE OF HIGH-STRENGTH CONCRETE TO REDUCE THE WEIGHT OF PRESTRESSED CONCRETE BRIDGES

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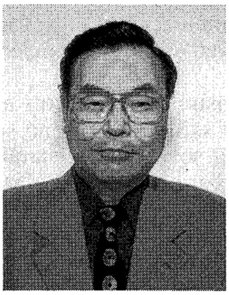
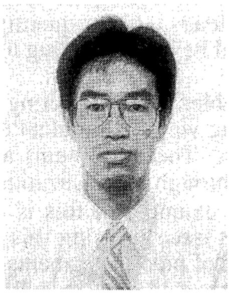


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This paper proposes a new type of structure, which we call the slab-truss structure, in which precast slabs and truss-shaped web sections of high-strength concrete are combined so as to avoid secondary stress resulting from prestress. The mechanical properties of this structure have been ascertained through loading tests on large-scale models, and by investigations of the shear transmission mechanism. Comparison with conventional structures shows that the new structure is rational and of practical benefit in reducing the weight of prestressed concrete bridges.

Key Words: high-strength concrete, prestressed concrete, weight reduction, precast concrete, slab-truss structure

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## 1. INTRODUCTION

The use of concrete structures in civil engineering has undergone a remarkable transformation since the invention of prestressed concrete (PC). A French engineer, E. Freyssinet, first made use of prestress in 1928, and the concept was introduced into Japan around 1949. The cantilever method (or Dywidag method) was then introduced from Germany in 1958. Since then, Japanese civil engineers have been constructing long-span PC bridges, including some of very large scale. In a long-span bridge, however, the weight of concrete becomes a disadvantage, and further development in PC bridge technology will not be possible unless concrete weight can be reduced. Various proposals have recently been made for solving this problem.

One such solution is to use high-strength concrete. If high-strength concrete were used to construct PC bridges,[1)2)] structural members would be lighter and as a result spans could be made longer. This would reduce the total weight of the bridge itself, thereby eliminating part of the cross-sectional force imposed by deadweight. (Deadweight generally accounts for 70% to 85% of the total sectional force on a bridge structure.) In addition, the bridge piers and foundations would bear smaller loads. Consequently, the total weight and count of structural members and materials would be lower, leading to improved economic efficiency.

At the same time, certain serious problems are afflicting the construction industry at the moment, including a shortage of young workers, gradual aging of skilled labor, and safety concerns in the working environment. These problems are being solved by streamlining construction methods and procedures through mechanization and other means of accelerating the overall construction process. An example of this is the precast segment method.[3)-6)] Weight reduction is also an important issue in using this method, since it streamlines the fabrication, transportation, and erection of precast segments.

In this paper, we propose a new structure based on the pre-cast segment method using a combined upper and lower slab members of 80-100 MPa high-strength concrete and truss-shaped web members of 80-100 MPa high-strength concrete in such a way as to avoid secondary stresses resulting from prestress. To verify the validity of the proposal, loading tests on large-scale models and investigations of the shear transmission mechanism were carried out to elucidate its physical characteristics. A model of the actual structure, consisting of three spans, was also built to allow comparison with conventional structures and identify the advantages and disadvantages of the proposal.

## 2. PROBLEMS RELATED TO WEIGHT REDUCTION

High-strength concrete is the key to reducing the weight of PC bridges built using the precast segment method, but there are some problems involved in its use. If the methods used in conventional structures are adopted, however, certain problems arise, as follows.

### 2.1 The PC Box Girder Structure

The thickness of the webs in a PC box girder structure is generally determined not according to bending stress or shear yield, but rather from structural considerations such as the PC steel bar spacing, the space needed for vibration compactors during concreting, or standard covering values. Making the structural members of a PC box girder thinner would reduce sectional rigidity and enhance yield, but this would also aggravate deformation, probably resulting in deflection or vibration problems.

In putting up a PC box girder bridge using the precast segment method, each segment would weigh as much as 25 to 80 tons, making transportation and erection dependent on the conditions under which a particular structure is being built.

Most erection methods for box girders are based on assembling precast members divided in the direction normal to the bridge axis. Not many methods have been developed with sectional arrangements or with structures designed to make the better use of the characteristics of the precast segment method.

## 2.2 Compound Structure with Steel Webs

With this type of structure, prestress is ideally introduced into the slab concrete, but because of the high rigidity of the steel webs in the direction of the bridge axis, the amount of prestress is constrained by the webs and full advantage of its benefits cannot be taken.[7] Also, the introduced prestress is further transferred to the steel webs with the progress of creep and dry shrinking. This transfer causes remarkable dislocational shear force at the joints between the steel webs and the concrete (Figure 1).

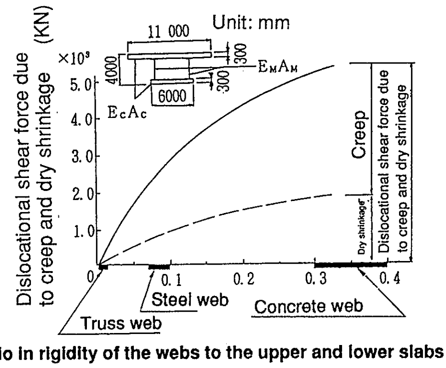



Fig. 1: Shear Force due to Creep and Dry Shrinkage

A modified method proposed to solve this problem with the steel webs involves pleating the steel webs, thus allowing free deformation in the axial direction. This modified method, however, lacks sufficient rigidity since the pleated webs are free to deform in the axial direction. Full practical implementation requires many problems to be clarified: how to evaluate torsional rigidity or shear rigidity, and how to evaluate the stress conditions when loaded with both a torsional moment and shear force simultaneously.

In addition to the above two methods, a structure in which the upper and lower concrete slabs are joined with steel trusses[6] has been proposed. With this structure, though, the problem is the considerable complexity of force transfer at truss nodes where two different materials — steel and concrete — come together.

## 2.3 PC Truss Structure

The truss structure features a prevailing axial tension, and all structural members can be placed under compression by introducing prestress to tension them. With this structure, if members with smaller sections are joined, extra flesh can be removed from the cross section of each, giving satisfactory weight reduction. This feature would appear to make the PC truss structure relatively useful in extending the length of PC bridges. A structure in which the truss and the floor framing are not integrated is the norm in PC truss bridges constructed so far.[9)-14)] This is probably for ease of structural analysis. In the overall structural scheme, the floor framing is regarded simply as an additional load and is never treated as a load bearing component. Cross framing is also required to resist torsion. As a result, the truss dead weight is greater than that of a PC box girder by about 20%.

The truss structure comprises many components, with each truss member acting on its neighbors to introduce prestress. As a result, secondary stress arises, dramatically reducing the prestress efficiency. There are some examples, although still very few, of the slabs and truss members being compounded (in a structure called the composite truss structure). These include the Bubiyan Bridge, the Sylans Bridge, and the Glacieres Bridge.[15),16)] Unfortunately, with this modified method, the prestressing process — which follows the assembly of precast members — causes large secondary stress and fails to reduce the sectional area of members, so weight reduction is not possible.

With conventional precast truss bridges, nodes are usually cast in place so as to absorb construction errors which arise during assembly. However, since the strength of the cast-in-situ joints is lower than that of the precast sections, the high strength of the precast concrete members cannot be fully utilized.

It is thus clear that the conventional truss structure fails to make full use of the characteristics of high-strength concrete in reducing the amount of concrete needed.

## 3. PROPOSED STRUCTURE

High-strength concrete may be effectively used in reducing the weight of a PC bridge in the following ways:

(1) Utilizing the compressive strength of high-strength concrete to reduce the sectional area of structural members while maintaining the efficiency of the sectional coefficient for the entire structural system.

(2) Introducing prestress to minimize the secondary stress while minimizing the loss of introduced prestress.

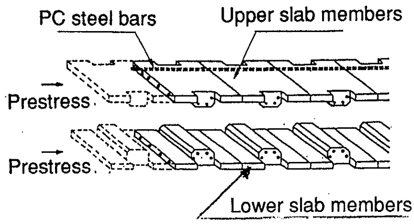
(3) Precasting structural members in a workshop to improve precision and quality, while using a connection structure that requires no in-situ concrete casting.

The structure proposed here is a slab-truss structure consisting of upper and lower slabs and truss members. The slabs provide all the components of the truss structure — chords, cross beams, cross frames, and floor framing — in one element, while the truss members act as the web. Each member is precast using high-strength concrete. The structure is divided into a suitable number of units considering manufacture, transportation, and erection. The method of assembling the precast components is shown in Fig. 2 and the features of this structure are described below.

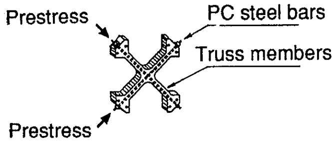
(1) Prestress is introduced into all precast slabs and trusses (Fig. 2 (a) and (b)). Once assembled as shown in the figure, the members are prestressed with no constraint. The prestress introduced into the upper and lower slabs for any given section should be designed to cause equal strain in the upper and lower slabs. This ensures that the secondary stress caused by concrete creep after completion is lower than that with conventional methods in which prestress is introduced after assembly.

(2) Precast members are held together by the friction between members caused by prestress in the perpendicular PC steel (Fig. 2-(c)). Using this connection method, the temporary shoring needed with conventional cast-in-situ nodes is unnecessary, thus helping to reduce labor requirements. Further, construction can proceed to the next stage after assembly without waiting for concrete to harden. This improves the speed of the precast segment method. After assembly, a filler (non-shrinking mortar) is injected into the joint clearance in the connections, thus integrating neighboring truss members.

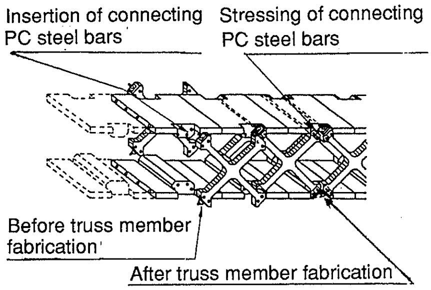
(3) By shop-casting all structural members, the quality of the high-strength concrete can be properly controlled and values of creep and dry shrinkage kept small.




(a) Fabrication of upper and lower slabs and introduction of prestress




(b) Prestressing of truss members




(c) Truss and slab fabrication

Fig. 2: Fabrication of Precast Sections

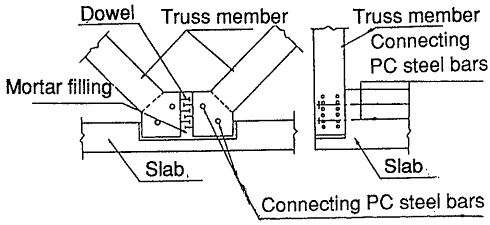



Fig. 3: Example of Connection of Floor Slabs and Truss Members

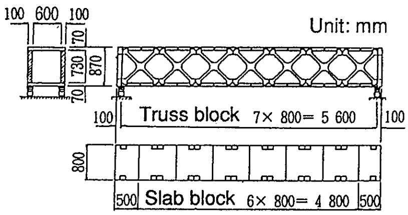



Fig. 4: Shape and Size of Models

## 4. MODEL TESTING

### 4.1 Specimens

#### (1) Design

The test models were designed such that the form of failure would be shear failure, resulting from torsion induced by a concentrated eccentric load. Tension-induced yielding of the PC steel bars in the truss members was designed to precede concrete failure in either the slabs or the truss members.

Figure 4 shows the model used in the tests. It is a double Warren-type truss structure with a span of 5.6 m, a height of 0.87 m, and top and bottom slabs without vertical members.

The structural members were eight upper and lower slab blocks, each 80 cm wide x 7 cm thick (six standard blocks and two ridge blocks); seven truss members, 6 cm x 10 cm, along each side (for a total of 14 on the two sides); and a cast-in-situ edge cross beam. The slabs and truss members were prestressed independently and then integrated into one monolithic structure.

The slabs and truss members were joined using PC bars such that they were held together by the friction resulting from the prestress. To be more specific, the axial tension acting on a truss member is transferred to the slabs by the friction resulting from prestress in the connecting PC bars, and then, also by friction, on to the adjacent truss member as axial tension. The coefficient of friction $\mu$ between members is usually 1.0, assuming that epoxy resin adhesive is used,[17] but so as to incorporate a safety factor of 2.0, the coefficient was assumed to be 0.5 for design purposes.

In the structural analysis of the model multilayered truss structure, the joints between trusses and slabs were taken to be rigid. Locally, small bending moments act on the truss members, but this was solved by adding reinforcing bars to the truss members.

To analyze the stress around the joints, a simulation of the truss members (tensile and compressive) and slab members at any connection was analyzed using three-dimensional FEM. The results of this analysis led us to change the structure, such as by increasing the tensile strength of the reinforcing bars at the joint and local reinforcement at the joints (Fig. 5).

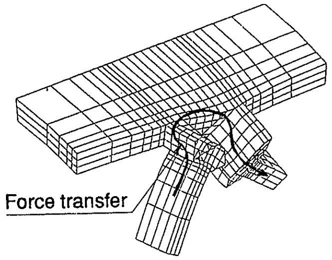


Fig. 5: FEM Analysis of Joint

The design strength of the mortar was 80 MPa. Steel bars (SBPR1080/1230) were used.

(2) Fabrication

Each block making up the test model was made of mortar. The mortar mixture used in the tests is shown in Table 1. The cement was high-early-strength Portland cement (specific gravity: 3.14), , the fine aggregate was river sand (specific gravity: 2.60; water absorption: 1.90%; fineness modulus: 2.82), and the admixture used was naphthalene high-performance water reducing agent (HWRA) (specific gravity: 1.20). PC steel bars 13 mm in diameter were positioned as the transverse links between upper and lower slabs and truss members, and 11 mm bars joined the truss members. SD295 D6 mm bar was used for reinforcement.

Table 1: Mortar Mix Proportions

| W/C (%) | W ($kg/m^3$) | C ($kg/m^3$) | S ($kg/m^3$) | HWRA ($kg/m^3$) |
|---|---|---|---|---|
| 34 | 240 | 706 | 1392 | 10.6 |

The model was manufactured in exactly the same way as the full-size structure shown in Fig. 2. Slabs and truss members were independently fabricated as precast segments and then assembled. The assembly procedure began with the laying of upper and lower slabs on the shoring to add stress to the PC steel bars in the slabs. The prestressed truss members were then moved into place from the side and engaged with the appropriate points on the slabs and joined with PC bars. As the joint was made, epoxy resin adhesive was applied to ensure the transfer of shear forces, and after applying stress, grout was applied to the PC bars. This procedure absorbs any fabrication errors that may occur during assembly, making the structure easier to assemble.

4.2 Loading Tests

(1) Test equipment

Two-point static loading tests were carried out using the loading apparatus shown in Fig. 6 and Photo 1. The loading beam, which was parallel to the bridge axis, was positioned off-center from the bridge axis so as to induce strain in the model.

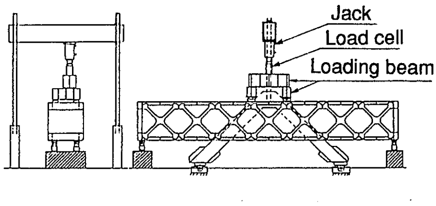



Fig. 6: Loading Methods

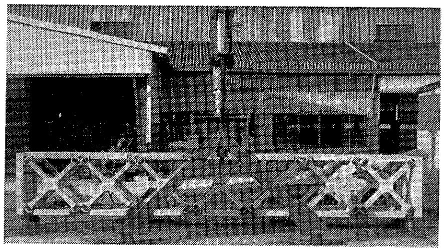


Photo 1: Loading Apparatus

(2) Loading method

Loading was applied until the crack limit state was reached, at which point it was released. The load was then reapplied, increasing it till the crack limit state was exceeded, once again removing it after noting the elastic behavior, cracking load, and crack positions. Final loading then continued until the ultimate loading condition (yielding of the diagonal tensioning PC steel bars), noting the ultimate shear strength and the type of failure.

(3) Measurements, measuring equipment, and methods

The measurements, apparatus used, and measuring methods used in the loading tests are described below:

- The load and reaction at the support were measured from the hydraulic jack placed on the model and load cells under the model supports.
- The displacement at the lower slab nodes was measured using displacement gauges (dial gauges) placed directly below the right and left trusses and the lower slab nodes.
- The strain in the slab concrete between the upper and lower supports and in the PC steel bars was measured with strain gauges pasted on the surface.
- The strain in truss member concrete and PC steel bars was measured with strain gauges on the concrete surface of tensile and compressive truss members and with strain gauges on the PC steel bars of the truss members under the greatest tensile loading.
- The local strain at the joints between trusses and slabs and at truss member cross points was measured with strain gauges affixed to the concrete surface.

4.3 Test Results and Discussion

(1) Strain in truss members

The relationship between the strain in the PC steel bars and the load acting on the truss member under the greatest tensile loading is shown in Fig. 7. In the truss members measured, cracks appeared at a load of 350 kN. The PC steel bars in the truss members yielded at a load of 560 kN since, after initial cracking, all tensile force acted on them, causing a sudden rise in stress. This type of behavior is not that generally observed in the shear failure of PC-reinforced webs, but is in fact closer to that of truss members under pure tension.

The relationship between load and mortar strain in the truss members under the greatest compressive force is shown in Fig. 8. The strain in compressive truss members gradually increases after the load reaches the level at which cracking first begins to appear in tensile truss members (350 kN). The final strain was far greater than theoretical values for ultimate loading conditions. This is because the rigidity decreases as a result of cracks in the tensile truss members, causing the axial tension acting on the truss members to be re-distributed and concentrated on the compressive truss members.

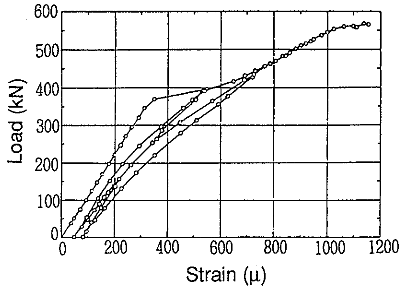



Fig. 7: Load-Web PC Bar Strain Relationship

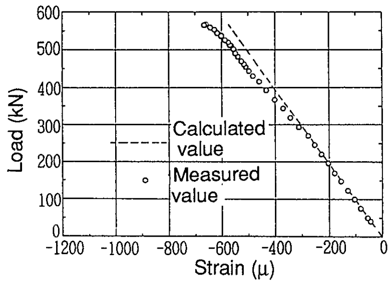



Fig. 8: Compressive Truss Member Load-Mortar Strain Relationship

Measured values of strain in the upper and lower slabs compare well with the results of three-dimensional FEM analysis for averaged stress on the upper and lower faces, but stress values on the upper and lower surfaces are different. This may be because the slabs, although under axial tension, are locally affected by bending.

(2) Deformation

Figure 9 shows the relationship between imposed load and vertical deformation on the right-hand and left-hand sides of the lower slab cross section. The difference in deformation between the left and right sides remains small until the loading level at which cracking begins to appear in the truss members (350 kN). This small difference in deformation due to torsion agrees quite well with the value calculated in consideration of warping torsion in the upper and lower slabs.

Although slightly greater than the calculated value when loading was relatively light, the deformation grew to exceed the calculated value as the load increased.

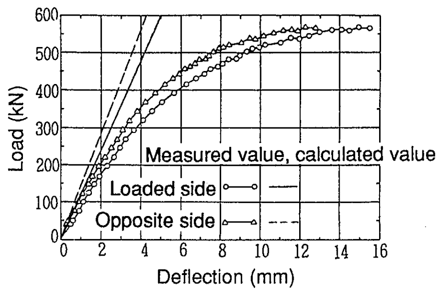



Fig. 9: Load-Deflection Relation

In the theoretical model, the joints between truss members and the slabs are assumed to be completely fixed. However, in reality, small rotational forces act on the joints when the load is still small. This would probably explain the above observation. When the load rose over 400 kN, deformation suddenly soared as the rigidity of the truss members fell due to cracking and crack-induced plastic rotation at the slab joints.

(3) Bearing capacity of the structure

PC steel bars in the truss members loaded with the greatest tensile force yielded at a load of 560 kN, compared with the theoretical yield load of 520 kN. The actual yield point thus agrees well with the theoretical value obtained from the analysis of the entire structure as a truss.

The joints between the slabs and truss members suffered little deviation at the design load, proving that their yield strength is adequate. However, under ultimate loading conditions, cracking occurred around the slab joints. This is because the structure depends solely on friction between the slabs and truss members, so the compressive forces on truss members are transferred to the area around the slab joints and then on to adjacent truss members as tensile force through the joints. This caused cracking as significant local stress built up on the slab joints (Fig. 5).

## 5. LOADING TESTS FOR SHEAR FORCE TRANSFER MECHANISM

### 5.1 Purpose of the Tests

On the basis of the results given in the previous section for full-length model loading tests, a mechanism for directly transmitting the shear force through the truss members was added to the joints. This was achieved by integrating adjacent truss members using insert bolts and high-strength mortar, thus ensuring that the vertical component (shear force) of axial tension on one truss member would be directly transferred to the adjacent truss member (Fig. 10).

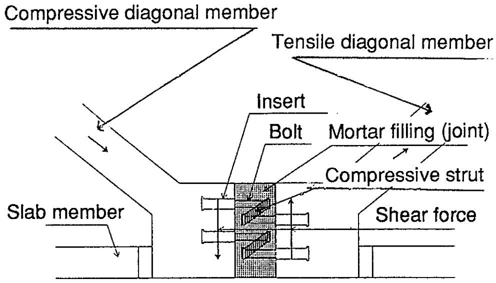



Fig. 10: Shear Transfer Mechanism

In these tests, the purpose was to identify the mechanism by which shear force is transferred between truss members and to quantify the yield strength. Thus, taking as parameters the presence of washers at bolt heads, the shape of the concrete finish at the joints, the quantity of reinforcement, and the angle of the construction joints, a model was constructed for use in loading tests.

### 5.2 Model

The shape and dimensions of the model are shown in Fig. 11. The joint clearances between members were filled with high-strength mortar. The parameter changes made during the tests were (1) the absence or presence of washers at bolt heads, (2) the concrete finish at the construction joints, (3) the quantity of reinforcement in the direction normal to the bolts, and (4) the angle between the loading axis and the face of the joints. Test conditions are shown in Table 2.

The design standard strength for the high-fluidity, high-strength mortar used in the model and to fill the joint clearances was 80 MPa. The bolts used to limit deviation were 12 mm in diameter (M12) with a thread length of 70 mm.

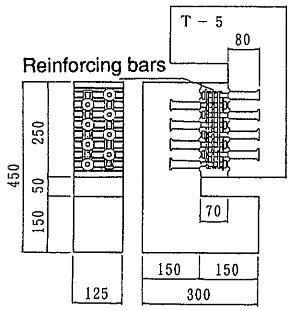


Fig. 11: Shape and Size of Models

Table 2 Experimental conditions

| Model | Bolt dia. | No. of bolts N | Bolt area ratio q (%) | Bulk area $A_{bc}$ ($cm^2$) | Construction joint condition | Rebar ratio at connection p (5) | Angle of joint θ |
|---|---|---|---|---|---|---|---|
| T-0 | M12 | 8 | 2.4 | 0 | Flat | 0 | 0 |
| T-1 | | | | 2.19 | | | |
| T-2 | | | | 10.39 | | | |
| T-3 | | | | | | 6.5 | |
| T-4 | | | | | Irregular | 0 | |
| T-5 | | | | | | 4.3 | |
| T-6 | | | | | | 0 | 22.5 |
| T-7 | | | | | | | 45 |

5.3 Test Results and Discussion

**(1) Effect of the bulk area of the washers**

According to the relationship between shear force and bulk area per bolt, the shear force increases in proportion with bulk area (Fig. 12).

(2) Effect of concrete finish at construction joints

The construction joints were filled with mortar to give an irregular surface finish, thereby increasing adhesion at the joints. This inhibited shear dislocation between the block and the joint (Fig. 13).

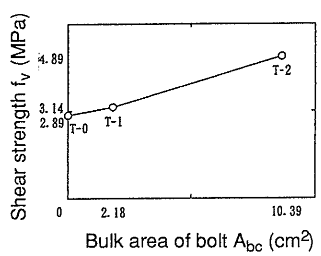


Fig. 12: Relationship Between Shear Strength and Bolt Bulk Area

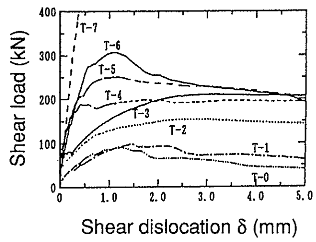


Fig. 13: Relationship Between Shear Load and Shear Dislocation

(3) Effect of reinforcement

This model, in which reinforcing bars were placed in the loading axial direction — or perpendicular to the bolt — to increase resistance to shear at the joints, failed through collapse of concrete in the vicinity of inserts in the block. The relationship between shear strength and the quantity of reinforcement can be used to deduce whether a particular quantity of reinforcement will ensure adequate shear strength, thus giving some information about the condition of construction joints and the effect of reinforcement (Fig. 14).

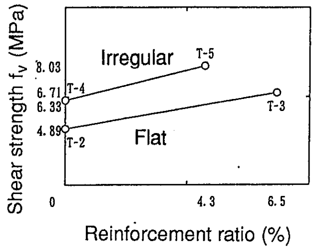



Fig. 14: Relationship Between Shear Strength and Reinforcement Ratio

(4) Proposed shear yield formula

The shear yield, $V_u$, is expressed as the sum of $V_s$, the proportion borne by the bolts, $V_c$, the concrete's ability to transfer shear on the crack surface, and α, the proportion borne by the reinforcement.

$V_s$ and Vc were obtained, respectively, using the stud equation proposed by Fisher et al. based on the dowel effect[18] and the shear transfer yield equation given in the Standard Concrete Specifications of the Japan Society of Civil Engineers.[19] To find α, an empirical equation for $V_u$ is determined from test results in which mortar with a concrete strength of 80 MPa is used, as follows.[20]

$$V_u = 1.3NA_b\sqrt{f_c} + 0.09(n-1)A_{bc}\sqrt{f_c} + \mu(91p + \sigma_{n)})A_c + 1053q$$
$$(\mu = 0.0085\sqrt{f_c})$$

where

- N : number of bolts per block
- $A_b$ : bolt cross section ($cm^2$)
- $f_c$ : compressive strength of concrete (MPa)
- $A_{bc}$ : bulk area, other than bolt, including bolt head and washer ($cm^2$)
- p  bolt area ratio
- $\sigma_n$ : vertical compressive stress (MPa)
- q: shear reinforcements ratio
- $A_c$ : sheared section area ($cm^2$)

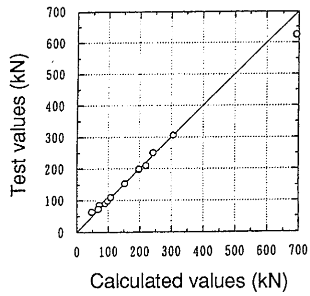



Fig. 15: Comparison Between Test Values and Calculated Values

The ratio of test values to values calculated using this shear yield equation was satisfactory, ranging from 0.90 to 1.37 as shown in Fig. 15.

(5) Application of this mechanism to truss member joints

Adopting this mechanism in the truss members integrates adjacent units into a monolithic structure, ensuring a direct transfer of the vertical component of axial tension. As a result, only the horizontal component of axial tension and the load imposed by the slabs are transferred via the slab joints, significantly reducing the force transferred by the joints. This minimizes the local stress that causes cracking at the slab joints. Use of this mechanism also prevents excessive deformation without risking plastic rotation at the joints.

## 6. VERIFICATION USING ACTUAL STRUCTURE

### 6.1 Structures Used

#### (1) Design conditions

Two structures were adopted for trial design: the slab-truss structure, as proposed in this paper, and the composite truss structure, examples of which are the Bubiyan, Sylans, and Glacieres bridges. The cross-sectional force, prestressing force, amount of concrete used, and the quantity of PC bars used were compared.

The cross-section was that of a standard road bridge with an effective carriageway width of 10.0 m. Structurally, it was a continuous-deck bridge with three spans. In order to investigate the applicability of the structure to various bridge scales, test designs were carried out for three center span lengths: 50 m, 75 m, and 100 m. The side spans were made 80% of the length of the center spans.

A typical structure is shown in Fig. 16, while the main dimensions are given in Table 3. All of the PC steel bars are incorporated into the members. The length of each truss in the direction of the bridge axis was made 2.5 m for ease of manufacture and transportation. To reinforce the transversal rigidity of the bridge, intermediate cross-beams (precast) were placed at 25 m intervals.

The concrete used for the slab-truss structure was a high-strength concrete with f'ck = 1000 kgf/cm$^2$ (98.0 MPa), while the composite truss structure was made with f'ck = 400 kgf/cm$^2$ (39.2 MPa) concrete taking into account the in-situ casting of the slabs. According to the JSCE's High-Strength Concrete Design and Construction Guidelines (draft), the allowable bending compressive stress at the design load is f'ca = 250 kgf/cm$^2$ (24.5 MPa) and 140 kgf/cm$^2$ (13.7 MPa) in these two cases, respectively.

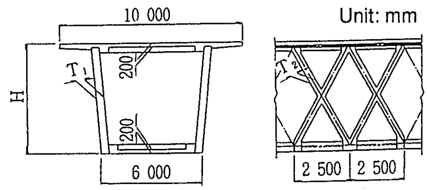



Fig. 16: Structure Diagram

Table 3: Major Structural Dimensions

| Type studied | | Slab-truss structure | | | Composite truss structure | | |
|---|---|---|---|---|---|---|---|
| Span (m) | | 50 | 75 | 100 | 50 | 75 | 100 |
| Height H (m) | | 3.3 | 5.0 | 6.7 | 3.3 | 5.0 | 6.7 |
| Slab thickness t (cm) | Upper slab | 20 | 20 | 20 | 27 | 27 | 27 |
| | Lower slab | 20 | 20 | (20 (30) | 27 | 27 | 27 (35) |
| Sectional area A (m$^2$) | Upper slab | 2.24 | 2.40 | 2.60 | 3.03 | 3.18 | 3.40 |
| | Lower slab | 1.44 | 1.60 | 1.80 | 1.95 | 2.10 | 2.32 |
| Truss $T_{1,2}$ (cm) | | 30x30 (35) | 40x40 | 45x50 | 40x55 | 50x60 (70) | 60x70 (90 |

Figures in parentheses indicate slab thickness and truss width above support.

(2) Erection

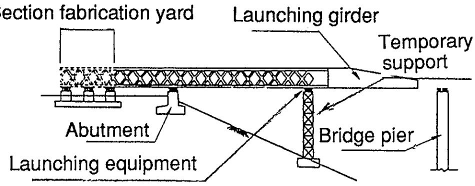



Fig. 17: Erection Diagram

Precast components of the slab-truss structure were fabricated in a yard set up behind the abutment. Upper and lower slab sections were fabricated independently and prestressed in the direction of the bridge axis. Then the truss members, already prestressed, were placed between the upper and lower slabs and joined with connecting steel bars (Fig. 2).
The length extended in each launching was four sections (10 m). Fabricated sections were moved forward using a launching machine, and the process was repeated until the bridge was complete (Fig. 17).

In the case of the composite truss structure, after integrating the precast members with the cast-in-situ upper and lower slabs, the assemblies were lined up in the direction of the bridge axis for prestressing. A temporary support was constructed in each span for the launching process. A launching girder, of length equal to half the distance to the temporary support, was attached to the tip of the truss to reduce the sectional force during erection.

Large compressive forces act on the truss members as they pass the support point during launching. Structural members were thus designed to ensure that this compressive stress during erection remained within tolerable limits.

6.2 Analysis Results

(1) Cross-sectional force

In computing the cross-sectional force on components of the slab-truss structure, the nodes were regarded as rigid; that is, the structure was analyzed as a truss structure with axial tension, bending moment, and shear force acting on each member.

The members of the slab-truss structure were reduced greatly in size by using high-strength concrete and ensuring that prestress causes no secondary stress. As a result, the sectional force due to the dead weight was reduced. The axial tension at the design load was about 18% and 25% lower, respectively, for spans of 50 m and 100 m as compared with the composite truss structure. This improvement increases with longer spans.

(2) Prestressing levels

The number of 32 mm-diameter PC steel bars (SBPR930/1230) — which can take a loading of 50 t each — required for the major cross-section is shown in Table 4 for the cross-sectional forces calculated for the completed structure and those during launching.

In placing PC steel bars in the slab-truss structure, the number of bars needed in the upper and lower slabs for any given section was calculated as required according to the tension expected in the members. In doing this, the bars were placed so that the strain due to prestressing in the upper and lower slabs would be equal, and the compressive stress on the members was then checked. Similarly, the number of PC bars required during launching was calculated.

For the truss members, the number of PC bars was chosen such that equal strain would occur in all truss members, whether tensile or compressive, in order to check tensile and compressive stress on the members. In addition, the number of bars was chosen to allow jointing between the slabs and truss members.

Table 4: Number of PC Bars per Major Section

| Type studied | | Slab-truss structure | | | Composite truss structure | | |
|---|---|---|---|---|---|---|---|
| Span (m) | | 50 | 75 | 100 | 50 | 75 | 100 |
| Side span | Upper slab | 24 (10) | 42 (18) | 60 (26) | (14) | (24) | (38) |
| | Lower slab | 18 | 28 | 42 | 26 | 40 | 62 |
| Support | Upper slab | 24 | 42 | 64 | 22 | 42 | 70 |
| | Lower slab | 16 (6) | 28 (12) | 44 (18) | (16) | (22) | (32) |
| Intermediate span | Upper slab | 24 (10) | 42 (18) | 60 (26) | (14) | (24) | (38) |
| | Lower slab | 20 | 30 | 42 | 32 | 48 | 70 |
| Truss | | 4 (2) | 6 (4) | 8 (4) | 4 (2) | 6 (4) | 10 (6) |

Figures in parentheses indicate number of bars required for erection by launching.

The introduction of prestress to the slabs and truss members is made possible, as explained above, by using a high-strength concrete with good tolerance to compressive stress.

In the composite truss structure the quantity of steel was calculated in the usual way for the design cross section and for the stress during erection. This calculation takes into account the secondary stress occurring during construction. It was also necessary to release some of the prestress, since the compressive stress on the members would exceed limits if the bars remain in full tension after completion.

(3) Secondary stress due to prestressing

In the slab-truss structure, the prestress changes with concrete creep, since the structural type during erection differs from that after completion. The change in secondary stress induced by prestressing was calculated using the creep theory equation proposed by Prof. Inomata.[21)] The creep value adopted in this calculation was $\phi = 1.5$ at the time construction was completed and thereafter. As mentioned above, however, the PC bars were placed so as to eliminate as far as possible differences in strain levels in the upper and lower slabs and the truss members. As a result, very little secondary stress occurs. On the contrary, with the composite truss structure, a secondary axial tension of 25% to 30% of the primary axial tension arises, so additional PC bars are needed to counteract it.

6.3 Quantitative comparison

(1) Amount of concrete

We express the amount of concrete used as the ratio of average structural member thickness to deck surface area ($m^2$).

Figure 18 compares a number of structures: slab-truss, composite truss, non-composite truss, and PC box girder (cantilevered and launched construction). The average structural member thickness is lower in the case of the slab-truss structure than for any other structural type, and is 25% to 30% less than the figure for a composite truss structure, depending on the span. It is also lower than the figure for PC box girder structures, which account for the greatest number of existing structures.

(2) Quantity of PC steel

Figure 19 compares the amount of PC steel bars used in the different structural types. In the case of the slab-truss structure, the requirement is lower by about 5% to 25% compared with the composite truss structure and the PC box girder structure as the span increases.

This is possible because the cross-sectional area of the structural members is less — resulting in improved prestressing efficiency — and because a new prestressing approach is introduced that eliminates secondary stress.

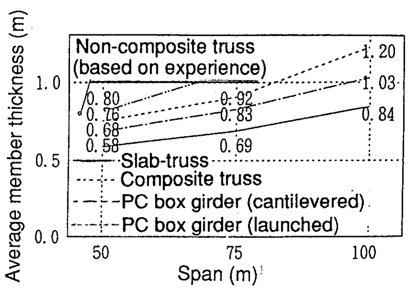



Fig. 18: Average Member Thickness

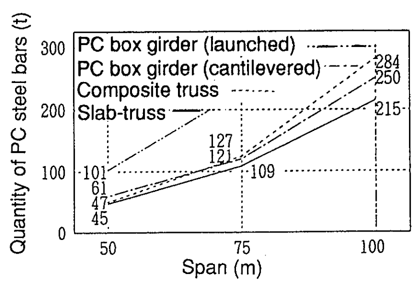



Fig. 19: Quantity of PC Steel Bars Used

## 7. CONCLUSION

The slab-truss structure proposed in this paper has been subjected to extensive testing, both large-scale loading tests and loading tests to determine the shear transfer mechanism. Test design of actual structures has also been carried out based on this structure. As a result of these tests, the following understanding has been reached.

(1) The stress behavior of a slab-truss structure under concentrated static loading agrees well with design values, verifying that this new structure can be analyzed as a normal truss structure. The structure was found to be highly rigid under torsional stress compared with a truss structure with no composite slabs.

(2) The proposed structure has sufficient yield strength against static load at the design load and ultimate loading condition.

(3) The frictional joints between truss members and slabs functions properly at the design load.

(4) A two-mode joint, making use of friction as well as a direct shear connection consisting of insert bolts and mortar filling, achieves connections between precast members of very high reliability.

(5) Effective use of high-strength concrete coupled with the new prestressing approach for reduced secondary stress allows significant reduction in the dead load.

(6) The proposed structure allows spans of 50 m to 100 m to be constructed by launching, thereby offering superior weight, reduced material use, and effective use of prestress.

In conclusion, the slab-truss structure proposed in this paper has been proven to be a rational and effective solution to weight reduction and precasting. In practical application, joints between truss members and slabs should be checked for the yielding in the case of the two-mode joints, and their constructability also needs to be verified. Currently, loading tests are being implemented on a large-scale model of a structure with these two-mode joints.

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## REFERENCES

[1] Murata, J., Kokubu, K, and Fukazawa, K.: High-Strength Concrete, Modern Concrete Technology, Vol. 8, Sankaido Publishers, 1987, pp. 89-242 (in Japanese)

[2] Ikeda, S., Kono, H., and Tezuka, M.: How High-Strength Concrete Alters PC Structures, Cement Concrete, No. 546, Aug. 1992, pp. 100-11 (in Japanese)

[3] Podolny, W., and Muller, M.M.: Construction and Design of Prestressed Concrete Segmental Bridges, pp. 325-331 (Japanese translation)

[4] Muller, J.: Ten years of experience in precast segmental construction, PCI JOURNAL, pp. 28-61, Jan.-Feb. 1975

[5] Ikeda, T., Nishiyama, K., Takaoka, S., Matsumura, Y., and Morimoto. M.: Precast Block Method, The Nikkan Kogyo Shimbun Ltd., pp. 1-27, 1969

[6] PB Kenkyukai (Sokuhisa, Y., et al): New Applications of Precast Block Method, Prestressed Concrete, Vol. 32, No.2, March 1991, pp. 32-42

[7] Chuah, C., Shima, H., Noritake, K., and Kumagai, S.: Strength and Deformation Behavior of Studs Embedded in High-Strength Prestressed Concrete, Annual Compilation of Concrete Engineers' Theses, 1991, No. 13-2, pp. 1033 1038

[8] Innovation dans le domaine des structures mixtes: Ponts mixtes métal-bêton précontraint, AFPT, Technique générale de la construction, No. 458 - Oct. et No. 459 - Nov., 1987

[9] Saito, T. and Kusama, H.: Design and Construction of Kuji Line Otanabe Bridge (PC Truss Bridge), Prestressed Concrete, Vol. 16, No. 4, Aug. 1974, pp. 5-10

[10] Machida, F., Miyasaka, Y., Osuda, N., Kobayashi, E., Yamamoto, T., Sue, Z., and Fukumoto, Y.: Design and Construction of Iwahana PC Truss (1) to (3), Prestressed Concrete

(1) Design: Vol. 17, No. 3, June 1975, pp. 40-55

(2) Construction: Vol. 17, No. 4, Aug. 1975, pp. 4-14

(3) Experiment: Vol. 17, No. 5, Oct. 1975, pp. 1-14

[11] Matsumoto, Y., Saito, T., Miura, I., and Mine, Y.: Design and Construction of Yasukegawa Railroad Bridge (Deck-Type Prestressed Concrete Truss Bridge), Collection of Theses for the Japan Society of Civil Engineers, No. 264, Aug. 1977, pp. 97-111

[12] Sawano, K., Iwasaki, T., and Nonomura, M.: Overhanging Construction of PC Truss Railroad Bridge — Kuji Line Masakisawa Bridge, Concrete Engineering, Vol. 16, No. 8, Aug. 1978, pp. 36-43

[13] Mine, T. and Ishihara, M.: Construction of PC Truss Bridge, Prestressed Concrete, Vol. 22, No. 3, June 1980, pp. 53-65

[14] Report on the Study of New Structural Applications of PC Bridges: Expressway Research Committee, Sep. 1990, pp. 40-43

[15] Boudo, J. et al: Les viaducs de Sylans et des Glacières, IABSE Symposium Paris-Versailles, pp. 493-499, 1987

[16] Giacomelli, D. and Greunerat, Y.: Les diaducs de Glacières et de Sylans, Travaux, pp. 105-114, Juillet-Aout, 1989
[17] Joint Research Report on Design Methods of Precast Block Bridges (I): Public Works Research Institute, Ministry of Construction, Association of Prestressed Concrete Construction Industry, March 1992, pp. 27
[18] Fisher et al: Shear strength of stud connectors in lightweight and normal-weight concrete, AISC Engineering Journal, April 1971, p. 55-64
[19] Standard Concrete Specifications (Design): The Japan Society of Civil Engineers, 1992, pp. 67-68
[20] Fukuda, E., Shima, H., Sokutake, K., and Sasaki, K.: Shear Connection of Precast Members using Insert Bolts, Collection of Annual Reports on Concrete Engineering, Vol. 15, No. 2, 1993, pp. 607-612
[21] Inomata, T.: Effect of Creep on Prestressed Concrete, Prestressed Concrete, Vol. 18, No. 2, April 1976, pp. 2-15