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THEME D

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Railway Bridges of Corrosion Resistant Steel in Japan

Ponts ferroviaires en acier résistant à la corrosion au Japon

Eisenbahnbrücken aus korrosionsbeständigem Stahl in Japan

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1. INTRODUCTION

A considerable number of railway bridges have been constructed of corrosion resistant steel, so-called "weathering steel", without painting in these ten years in Japan (say, 70 sets of span and 11,742 tons in total). Some of them were furnished with test specimens for future observation. This type of bridge has been applied to only an open type bridge, where the structural members are well exposed to rainfall and located in rural areas where air is not polluted. Such conditions are favorable to formation of the stable patina, an anticorrosive surface layer, which is essential to the protection of steel from corrosion. At present, all the brides are generally in a very satisfactory condition and prove to be successful.

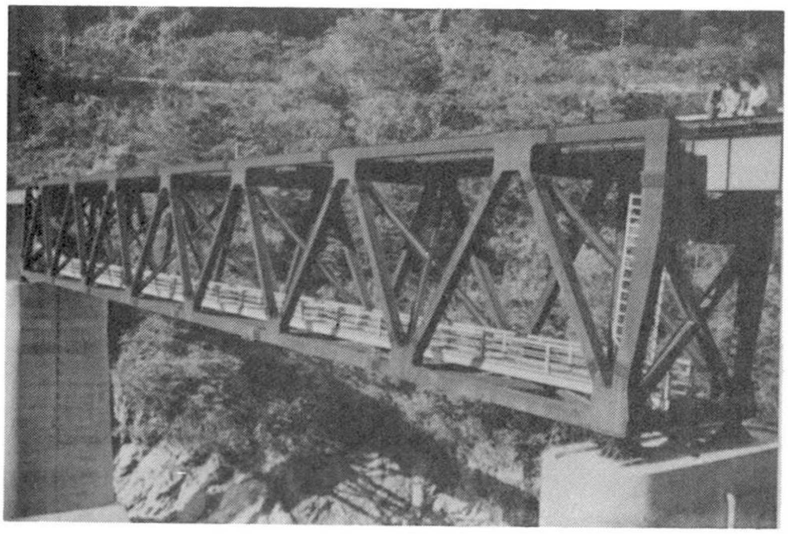

Photo 1. Third Okawa Bridge

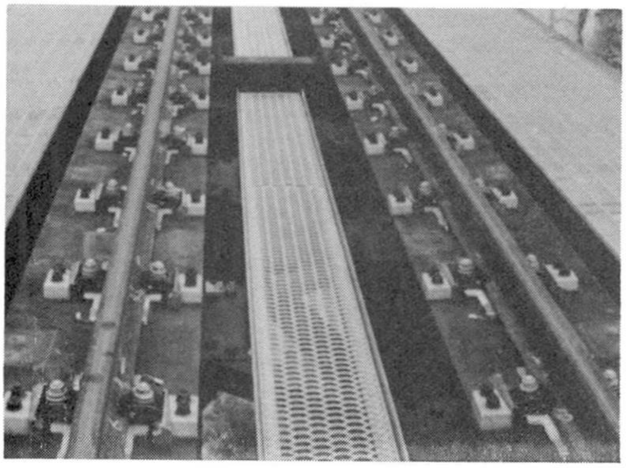

Photo 2. Track structure

2. STRUCTURAL DETAILS

The structural details of the weathering steel bridges for railways in Japan are so modified, as to fit the use of weathering steel without painting. They have been carefully designed, so that water and dust staying on the structural members should be minimized as shown in Fig. 1. Such considerations will be useful also to ordinary painted bridges for better durability. In Third Okawa Bridge (Photo 1), for instance,

1) The upper surfaces of horizontal members, even the flanges of stringer and

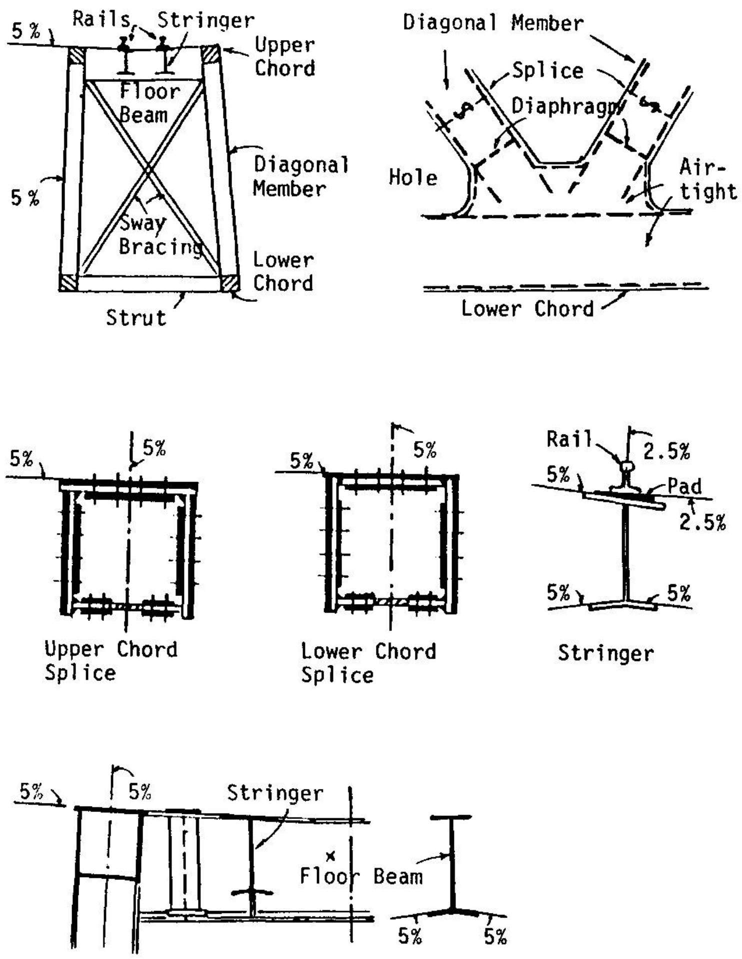

Fig.1. Examples of structural details

floor beam and chord members are inclined by 5%.

2) Larger horizontal gusset plates are provided with large openings.

3) The chord members and the panel point portions are closed for air- tightness, except for their splice portions, the inside of which is coated with tar - epoxy resin paint.

4) The rail track structure is different from an ordinary one with wood sleepers. Since the flange surface in contact with the sleepers is usually most severely damaged by corrosion, the rails are fastened to the flanges of the steel girders with a special device instead of wood ties as shown in Photo 2. It is drastically effective in reduction of maintenance work for the track as well as the bridge structure. And

5) The edge of the upper side of concrete pier is raised and water is gathered to the drain pipe, so that rainwater washing the bridge may not stain the flank of pier, as seen in Photo 1.

3. EXAMPLE OF OBSERVATION

Five years after Third Okawa Bridge was erected and opened to traffic, the bridge structure and the test specimens installed nearby were closely examined, by the visual inspection, X-ray test, Ferroxyl test, etc. In addition, some bolts used in the bridge were removed to see the state of bolts themselves and the inside of bolt holes. The summary of the results is as follows; 1) As a whole, the condition is so good, that reduction in the plate thickness can not be measured.2) According to Ferroxyl test on various parts of the bridge and X-ray test on the exposed specimens, the stable patina has been better formed on the surface which has been directly exposed to rain-fall and dried immediately. From this point of view, the upper surface is most favorable, if water does not stay, and followed by the vertical surface on a sunny side. In case water is apt to stay on horizontal surface, however, the patina is not easily formed. On the surface of underside, the patina is seldom formed and slight corrosion pits are recognized, but the rate of corrsion is very slow if the enviromental atmosphere is not humid, and 3) Some of the bolt holes were filled with water. Though it has resulted in no appreciable detrimental effect as yet, it may cause the corrosion of the bolts in the future. It will, therefore, be necessary to keep them water-tight.

Enhanced Durability of Post-Tensioning Tendons

Durabilité des fils d'aciers de post-contrainte

Verbesserte Dauerhaftigkeit von Spanngliedern

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For many years the corrosion resistance of conventional prestressing tendons has been assumed to be adequate by virtue of the concrete cover and the embedment of the prestressing steel in grout. This is true for the majority of the structures but recent evidence of corrosion damages in many countries has created increasing concern among engineers. Corrosion protection measures are of particular importance for partially prestressed concrete structures. Cracks can occur already under service conditions, resulting not only in an increased vulnerability to corrosion but also in a higher stress range of prestressing steel and reinforcement. Plastic ducts are insensitive to most chemical attacks. They are elastic and are therefore able to adapt to local crack propagation, and - as extensive experimental investigations at ETH Zurich have indicated [1] - show almost a doubling of the stress amplitude which can be withstood.

2. DEVELOPMENT WORK PT PLUS 1)

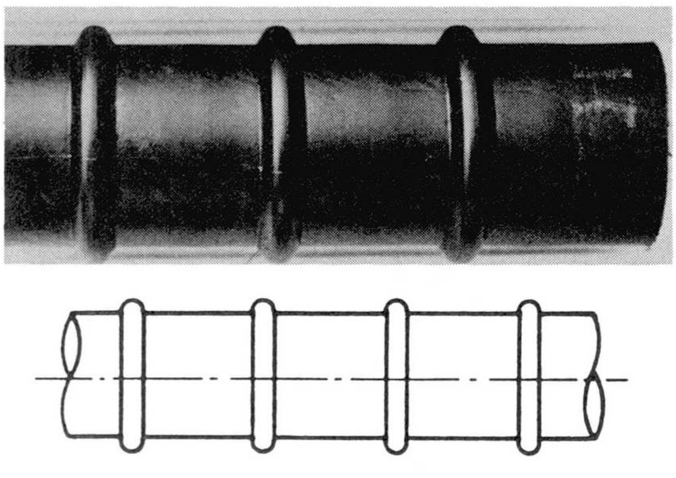

Figure 1: PT Plus duct

With the newly developed PT Plus duct (Fig. 1) a series of experiments were carried out and the behaviour evaluated.

a) Groutability: To have a direct comparison of the groutability of the PT Plus plastic duct and a conventional steel duct two 80 m long ducts with identical boundary conditions were grouted. Upon hardening both ducts were opened and the grouted cables were visually inspected and assessed. Both injections were of good quality. Important was the fact that also the corrugations in the plastic duct were fully filled with grout.

1) Trademark applied for

b) Bond behaviour: The bond between strands, grout mortar, duct and concrete ensures that, after cracking, cable forces can be activated which exceed the initial prestress. The question was therefore whether the polyethylene duct would be able to develop the yield strength of the prestressing cable. Results based on pull-out tests (with cables of 4 and 7 strands 0.6" dia., duct dia. of approx. 60 mm and 70 mm) allow the following statements:

- The yield force can be reached with this type of polyethylene duct.
- In the serviceability limit state the crack behaviour with a PT Plus duct is similar to the one with a normal steel duct.
- With a PT Plus duct the bond length required to anchor the force difference between the yield force and the actual prestressing force is about twice as big as compared to a conventional steel duct.
- In the ultimate limit state the crack widths are approx. double when comparing PT Plus and steel ducts.

c) Abrasion: The abrasion tests carried out at the ETH allowed a 0.6" dia. strand to be stressed to 75 % of its nominal tensile strength with a simultaneous simulating of an elongation of 1000 mm under various lateral pressures ranging up to 9 kN on a 25 mm specimen. With minimum tendon curvatures, depths of penetration of the prestressing steel into the duct wall of max. 0.5 mm to 1.0 mm are not exceeded. These values are clearly smaller then the wall thickness chosen for the new PT Plus duct and therefore an intact encapsulation of the prestressing steel is ensured.

3. CONCLUSIONS

The results of these tests show that the new plastic duct PT Plus can be safely used for post-tensioning systems. Its application is recommended to wherever an improved corrosion protection is desired (i.e. bridge decks, parking garages, marine structures) but also for structures subjected to fatigue loading.

[1] Oertle J.: Reibermüdung einbetonierter Spannkabel (Fretting fatigue of bonded prestressed tendons). Institut für Baustatik und Konstruktion, ETH, Report No. 166, September 1988.

Influence of Steel Bars on Structural Behaviour of Reinforced Concrete

Influence des barres d'armature sur le comportement du béton structural

Einfluss neuer Bewehrungsstähle auf das Tragwerksverhalten

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1. INTRODUCTION

Due to the evolution of manufacturing processes, reinforcing steel bars possessing new mechanical characteristics, different from those of traditional ones, are now available on the market. Therefore Model Code 90 and Eurocode 2 take into account two classes of ductility, related to characteristic elongation at maximum load, and characteristic tensile strength to yield stress ratio (class A with $\varepsilon_{uk} \geq 5\%$, $(f_t/f_y)_k \geq 1.08$ and class B with $\varepsilon_{uk} \geq 2.5\%$, $(f_t/f_y)_k \geq 1.05$).

The different ductility characteristics, greatly affect the structural behaviour in the plastic range.

2. TESTS ON STEEL BARS.

Experimental tests have been carried out by strain controlled procedure on two types of high bond bars with 12mm diameter:

-hot rolled bars, which can be classified as class A (30 specimens);
-cold rolled bars, which can be classified as class B (50 specimens).

The determination of ε_u has been performed in three ways:
(a) by means of extensometer;
(b) by measuring the deformation after failure outside the necking zone and away from the grips (adding the elastic deformation f_t/E);
(c) by measuring the deformation after failure of a 5 diameter (A_5) and a 10 diameter (A_{10}) base, including the necking zone, as follows:

$$\varepsilon_u = 2A_{10} - A_5 + f_t/E.$$

	Steel class A			Steel class B		
Measured charact.	Mean value	Stand dev.	Char. value	Mean value	Stand dev.	Char. value
f_y (N/mm²)	587	24.14	540	596	6.22	583
f_t (N/mm²)	672	19.94	633	641	4.12	633
f_t/f_y	1.150	0.021	1.108	1.076	0.008	1.061
A_5 (%)	17.85	1.58	14.74	15.10	1.08	12.97
A_{10} (%)	12.58	1.27	10.08	10.03	0.91	8.24
ε_u(a)(%)	7.00	0.98	5.06	4.18	0.76	2.68
ε_u(b)(%)	6.29	0.81	4.69	4.47	0.85	2.80
ε_u(c)(%)	7.66	1.21	5.27	5.28	1.09	3.13

Table 1 Characteristics of steels

3. TESTS ON REINFORCED CONCRETE BEAMS.

Plastic rotation capacity of the beams was taken as a parameter to evaluate the influence of steel type. Midspan deflection controlled tests were carried out on 28 simply supported r.c. beams with various steel percentages, loaded by one load applied at midspan or by three symmetrical loads (Fig. 1). Beam depth was 400mm for 13 specimens and 600mm for 15 specimens; depth to width ratio was two whereas the span was 4000mm and 6000mm, respectively.

Plastic rotation, Θ_p, is obtained by integration, along the plastic zone, l_p, of the difference between mean curvature $1/r_m$ and that obtained at the yield limit of steel, $1/r_{my}$, according to:

$$\Theta_p = \int_{l_p} (1/r_m - 1/r_{my})\, dz$$

Bending moment versus plastic rotation curves, referred to the beams with 600mm of depth, are plotted in Fig. 2. Total plastic rotations, evaluated at 90% of the maximum bending moment in the descending branch of the moment-rotation diagram, are reported in Fig. 3 versus the x/d ratio, x being the compressive depth calculated at ultimate limit state assuming the design values for materials and d being the effective depth.

The existence of two branches, theoretically given, was confirmed by the tests performed so far. These branches, qualitatively shown in Fig. 3, depend on steel class, beam depth and form of bending moment diagram.

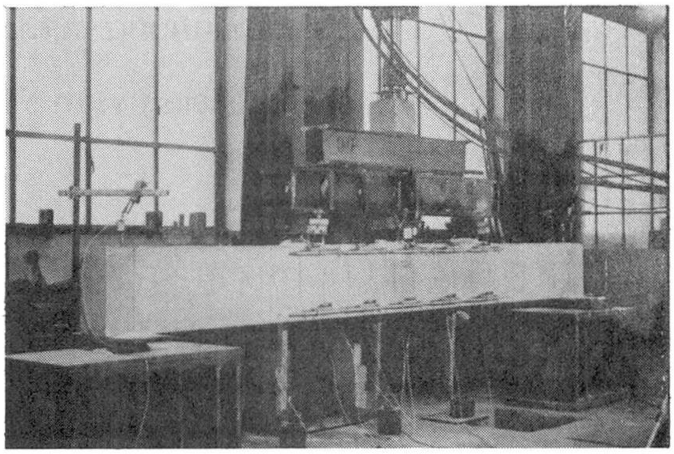

Fig. 1 Testing apparatus for three symmetrical loads

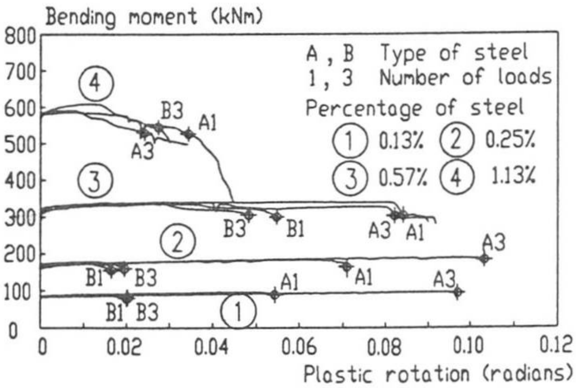

Fig. 2 Bending moment versus plastic rotation, varying the steel percentage, for beams with depth of 600mm

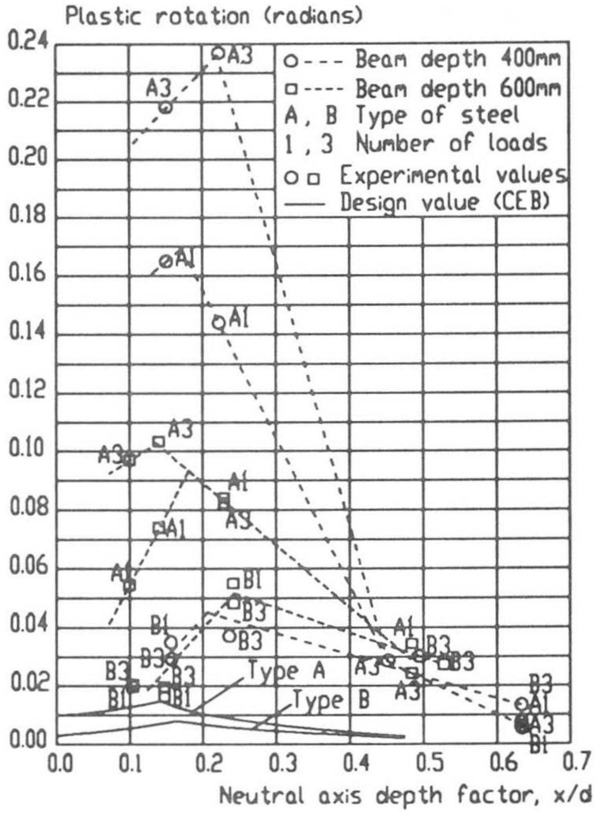

Fig. 3 Total rotation versus x/d ratio

4. CONCLUSIONS

Compared to the traditional reinforcing steel bars the $(f_t/f_y)_k$ value requested by European Codes seems significantly reduced. As a consequence a reduced structural ductility can be observed, in particular when low percentages of class B steel are used. This behaviour may be significantly unfavorable in presence of imposed deformations (e.g. settlement of supports) and can reduce the possibilities of load effect redistribution.

Précontrainte à l'aide de câbles en fibre de verre

Vorspannung mit Glasfaserkabeln

Prestressing with Fiberglass Cables

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Le matériau fibre de verre présente des caractéristiques mécaniques de traction similaires à celles des aciers de précontrainte, et certaines caractéristiques physiques et chimiques intéressantes.

La Société COUSIN, a mis au point des joncs composés de fibre de verre et d'une matrice en résine thermodurcissable qui les enrobe (polyester, époxy...). Le pourcentage pondéral résultant en fibre de verre se situe autour de 82 %. Il se distingue des produits existants sur le marché par un tressage périphérique sur toute la longueur. Ce jonc rassemble l'ensemble des caractéristiques propres à la fibre ainsi que des caractéristiques provenant de la matrice en résine :

CARACTE-RISTIQUES	UNI-TES	JONC/VERRE	ACIER DE PRECONTRAINTE	INOX 18 NCD 6
Densité		1,95	7,85	7,85
Contrainte rupture en traction	daN/mm²	170	180	113
Module d'élasticité	daN/mm²	4.400	19.400	21.000
Conductibilité électrique	Ohms x cm	Non conducteur env. 10	Conducteur 1,3	Conducteur 1,3
Conductibilité thermique	W/m °C	1	30	30
Contrainte rupture/densité	daN/mm²	87	22,9	14,4

Parmi ces propriétés certaines présentent un intérêt particulier dans le domaine de la précontrainte, et permettent d'envisager :

- La suppression de l'injection des armatures ;
- Des applications lorsque des déformations importantes de la structure sont prévisibles (faible raideur).

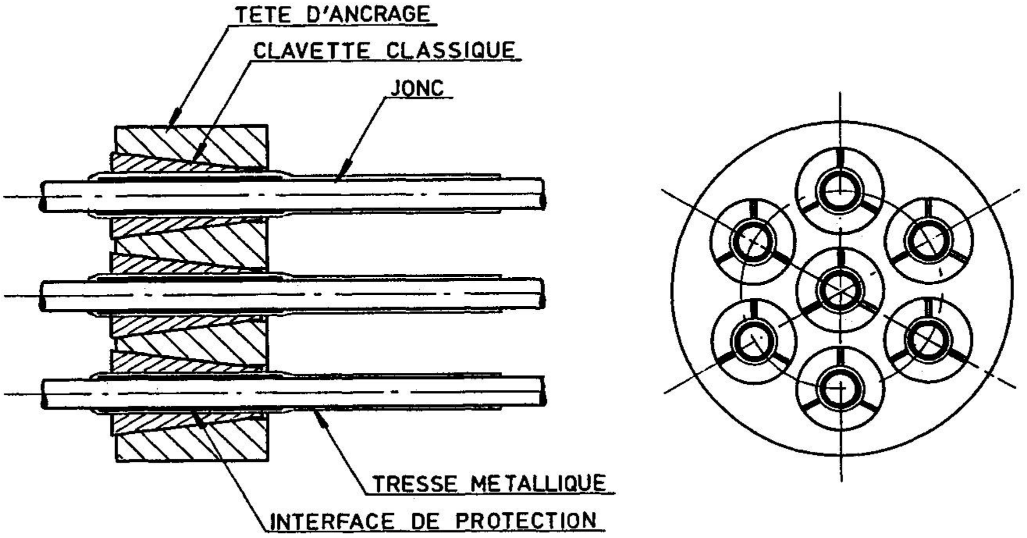
Cependant, deux problèmes limitent actuellement le développement de ce matériau dans la précontrainte des ouvrages d'art :

- Le prix du matériau brut, 10 fois plus cher que l'acier (cet écart est ramené à 2,5 si l'on prend comme critère l'effort utile) ;

- L'anisotropie du matériau : il s'agit d'un jonc unidirectionnel (l'axe de résistance étant l'axe du jonc) ; le jonc n'accepte pas de pressions radiales ou ponctuelles importantes : dans le cas où elles existent il y a un risque de décohésion entre les fibres par rupture de la résine.

La première phase de recherche et d'essais menée par GTM BTP et COUSIN, avec le concours du LCPC, avait pour objectif de démontrer la faisabilité d'un ancrage de précontrainte dans les conditions de chantier. Elle a été orientée vers l'utilisation maximale de matériel et de pièces standards des procédés SEEE FUC.

Après de nombreux essais en laboratoire, nous avons retenu le dispositif suivant :



Les résultats obtenus sont satisfaisants, et ont permis d'atteindre la rupture nominale r dans plus de 95 % des cas (essais effectués en traction déviée = 60°).

Dans un deuxième temps, une expérience en vraie grandeur a été menée sur un pont poussé en construction près de Genève (GTM BTP - Viaduc de Bardonnex - France). Nous avons ainsi mis en tension deux câbles droits de 50 m de longueur composés de 7 joncs chacun, tendus à 60 % de la rupture nominale, soit 350 kN par câble.

Les ancrages sont équipés de capteurs de force.

Il faut noter que ce procédé d'ancrage utilise du matériel classique de mise en oeuvre exploité en précontrainte sur toron 0,6" acier et permet donc d'envisager des applications aisées dans les cas d'indication de la fibre de verre.

Shear Strength of High Strength Concrete Beams

Cisaillement de poutres en béton armé à hautes performances

Schubtragfähigkeit hochfester Betonbalken

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1. INTRODUCTION

The primary objective of this study is to investigate the shear strength of high strength concrete beams with and without stirrups. Thirty-three beams, of which twelve were without stirrups, having compressive strength in the range of 64 MPa to 90 MPa were tested. All specimens were 200 x 300 mm in cross section. The main variables considered were concrete strength, shear span to effective depth ratio (a/d), and the amount of stirrups ($p_v f_y$). The beams were tested by two point loadings symmetric about midspan as shown in Fig. 1. The a/d used were 2.0, 2.5, 3.0, and 3.5 respectively. The amount of stirrups designed were 0.5, 0.75, and 1.0 times the estimated V_{cr}. In the analytical study, the effectiveness factors of high strength concrete beams, according to the test results of authors and other investigators, were calculated based on the theory derived by M. P. Nielsen.

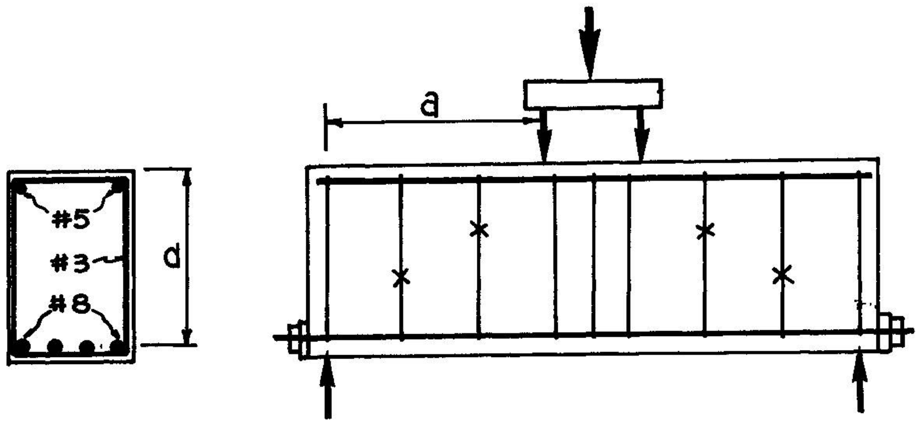

Fig. 1 Test specimen and loading arrangement

2. MATERIALS, SPECIMEN, AND MEASUREMENTS

The cement used was ordinary portland cement. River sand with F.M. of 3.0 and crushed stone with maximum size of 9 mm were used. Two mix proportions of high strength concrete, having water cement ratio of 0.26, were prepared in the laboratory. The first mix for those having compressive strength up to 70 MPa has a cement content of 600 Kg/m³. In the second mix, 5 per cent of cement by weight was replaced by silica fume to improve the strength. The volumetric ratio of fine to total aggregate (S/a) was 0.37. At least five 10 x 20 cm cylinders from the same batch of beam were cast to determine the compressive strength. The cylinders and beams were covered by wet burlap until test. The strains of stirrups and longitudinal steels, deflections at midspan and quarter span were monitored throughout the test. The load-strain relationship of concrete strut was measured using clip-on gage with both ends attached to the appropriate holes drilled prior to the start of test.

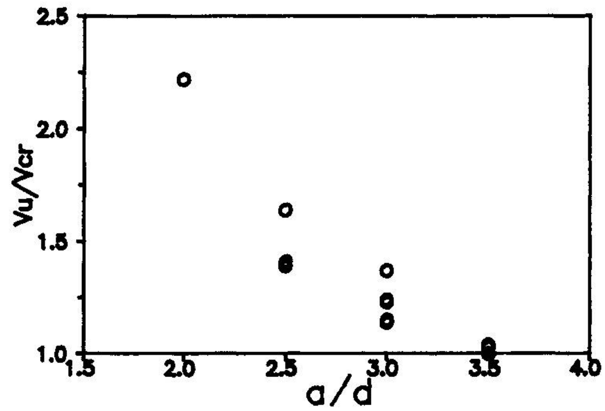

Fig. 2 Measured V_u/V_{cr} of beams without stirrups

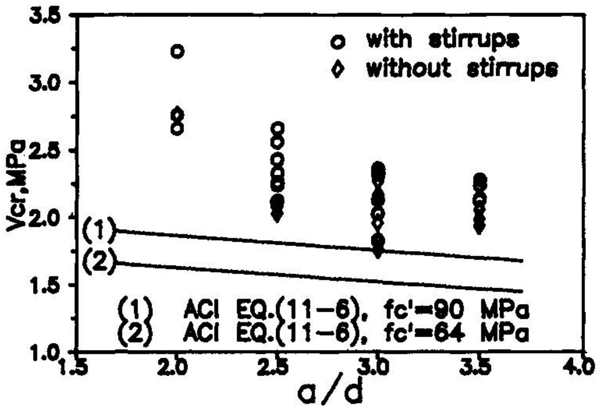

Fig. 3 Comparisons of V_{cr} measured vs. predicted

3. CONCLUSIONS

Some conclusions were drawn based on the test results:

1. Most of the beams without stirrups failed in shear - tension while those with stirrups failed in shear - compression.

2. The ratio of measured ultimated shear strength (V_u) to inclined shear strength (V_{cr}) for various a/d of beams without stirrups is shown in Fig. 2.

3. The inclined shear strength is slightly higher for beams with stirrups. The shear strength contribution of concrete predicted by current design provision (1989 AASHTO or ACI 318-89) is still conservative as shown in Fig. 3.

4. For the same a/d, the shear strength contribution of stirrups is more significant for beams with lower $p_v f_y$.

5. The effectiveness factors of high strength concrete beams without stirrups vary from 0.2 to 0.4, while those with stirrups vary from 0.4 to 0.6.

External Reinforcement Beams for Road Bridges

Poutres à armatures extérieures pour les ponts-routes

Beton-Strassenbrücken mit äusserer Stahllaschenbewehrung

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The standard superstructures for motor-road bridges of ordinary reinforced concrete that have lately received wide acceptance in the USSR possess reinforcing cages with multi-raw arrangement of operating bar reinforcement in the beam bottom chord. The beams have a number of advantages. However, manufacture of reinforcing cages requires considerable labour input with the use of manual are welding. Substitution of working external bar reinforcement with external sheet metal sheet reinforcement makes it possible to mechanize production of beams using a highly efficient welding equipment and save metal.

On basis of the performed research,new design-technological solution of beams, method of their calculation and design requirements have been developed. The beams consist of the non-stressed metal sheet located on the lower fibre without the concrete protective layer (see Fig). The combined operation concrete with sheet reinforcement is provided by vertical anchors from bar reinforcement. The anchor bars of the length close to the height of the beam rib simultaneously perform the role of transverse reinforcement assigned by calculation of inclined sections on the cross force effect. The anchor bars are attached by tee butt automatic welding. On some sections along the beam length the sheet reinforcement may be strengthened by the bar one, located above the sheet inside the beam (combined reinforcement).

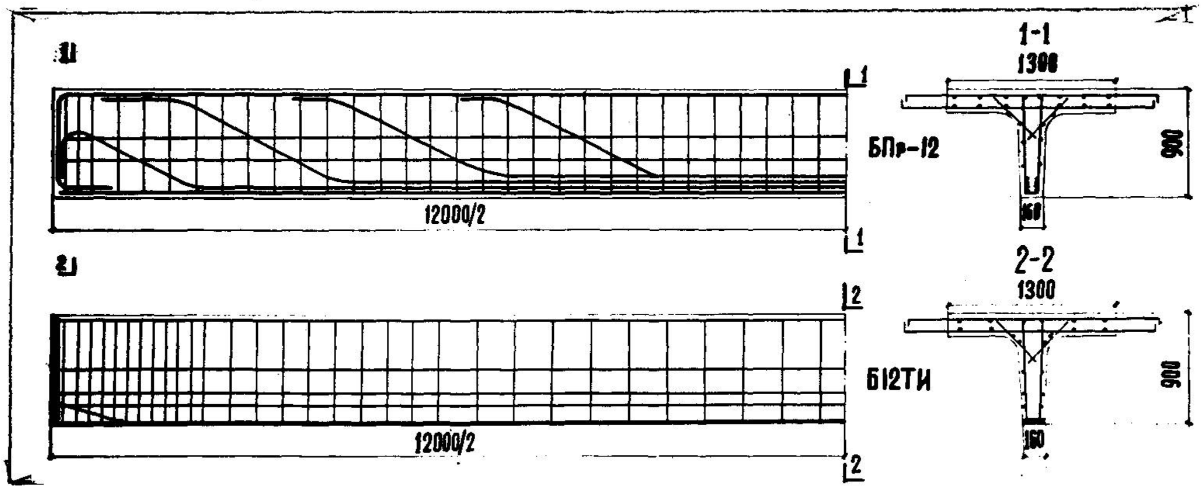

Fig. Beams of Motor-road Bridge Superstructures
1- standard, reinforced with bars;
2- with external sheet reinforcement.

The new design has found practical use in superstructures for bridges on roads in the oil-gas bearing area of Western Siberia. Application of beams of new design allows lowering of labour consumption in

manufacture of a rib frame by 2,8 times as compared with the standard and the total labour consumption in manufacture of a single beam up to 1,5 times.
Investigation of bridges of 12 m long beams erected in Western Siberia and in the Urals has revealed that condition of beams with external reinforcement does not differ from that of the standard beams.
Having considered advantages of the external reinforcement beams, improvement of labour conditions at manufacture, as well as the positive results of operating bridge investigations, four designs of 12 to 18 m long external reinforcement superstructures of various cross sections(arch plate, T-beams with and without diaphragms, arch beams with small external cantilevers). To study operation of the designed structures comparison tests with a static load of two 12 m long T-beams without diaphragms were carried out. One beam had an external sheet reinforcement, the other was according to the standard design with the bar reinforcement. The tests have revealed higher properties of the structure with external reinforcement. No violation of external reinforcement sheet-to-concrete contact at all loading stages were detected.
The new design-tehnological solution can find further development in the railroad bridge beams in the form of composite constructions with external sheet reinforcement and internal stressed bar one or cables.

Fibre RC Deck Slabs with Diminished Steel Reinforcement

Dalles en béton renforcé de fibres et avec une armatures métallique réduite

Fahrbahnplatten aus Faserbeton mit reduzierter Stahlbewehrung

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1. INTRODUCTION

It is now well-established that when concrete deck slabs of slab-on-girder bridges are subjected to concentrated loads an internal arching system develops in the slabs, preventing them from responding in a purely flexural mode. The presence of the internal arching system, by reducing the tensile stresses in the slab, causes the slab to fail in punching shear rather than in flexure [1,2,3]. Recognizing this mode of behaviour of deck slabs, some jurisdictions around the world (e.g. [4]) require the deck slabs of their bridges to be designed by taking account of this internal arching. Overlays of concrete mixed with chopped polypropylene fibres and without any steel reinforcement have already been applied successfully on existing decks. From this application there has emerged the idea of deck slabs with polypropylene Fibre-Reinforced Concrete (FRC) with little or no steel reinforcement. Since the deck slab is subjected to predominantly compressive stresses, the low modulus of elasticity of the fibres is not expected to be of concern, as it is in purely flexural slabs. The fibres are practically inert to deicing salts; accordingly a deck slab reinforced with these fibres should prove to be very durable.

2. EXPERIMENTAL SETUP

The Technical University of Nova Scotia (TUNS) has initiated a research program to investigate experimentally and analytically, the feasibility of constructing a deck slab with FRC and with limited or no steel reinforcement. The laboratory model used in this program, represents at half-scale, a two-girder bridge having a 200 mm thick slab on girders spaced at about 2.2 m and with a span of about 7.3 m. The details of the model are shown in Fig. 1 which also illustrates the setup for the application of a concentrated load at the centre of the slab. The first model was provided with no diaphragms at the supports and three intermediate diaphragms, at mid-span and each of the quarter span locations. For the second test, fairly substantial diaphragms were added at the supports. For both models, the necessary workability of the concrete mixed with the polypropylene fibres was achieved by adding extra water rather than by the use of customary super-plasticizers.

3. TEST OBSERVATIONS

The first model failed under a concentrated load of about 177 kN. The mode of failure was not that of pure flexure, nor did it resemble the punching shear type of failure observed in deck slabs incorporating steel reinforcement. The deck failed along two lines which were close to and roughly parallel with the girders. The second model failed at about 222 kN in practically the same mode as that of the first model. A 25% increase in the failure load clearly establishes the significance of diaphragms at the supports. Despite the fact that the failure mode of the FRC slab is somewhat different from its steel-reinforced counterpart, the failure load for the second model corresponds to about 890 kN, which is about 18 times the maximum permissible weight of one half of the axle of a vehicle. It is pointed, however, that unless the failure zone could be restricted to within the close proximity of the zone of load application, the presence of other concentrated loads on the deck cannot be neglected as it can be in the case of the punching shear type of failure.

4. FINITE ELEMENT MODELLING

The model was analyzed using the finite element program ADINA [5]. The analysis model incorporated 20-noded isoparametric solid elements to model the deck slab; 3-D beam elements to model longitudinal girders; and 3-D truss elements to model the transverse diaphragms. Figure 2 plots the load-deflection relationships under the load for different degrees of restraint provided by the transverse diaphragms. It can be seen from this figure that the restraint provided by the diaphragms has a significant influence on the failure load.

5. CONCLUSIONS

Tests on the models have clearly pointed towards the feasibility of FRC slabs with little or no steel reinforcement. However, before such slabs are recommended to be incorporated in bridges, it is necessary to know more about their behaviour through full-scale static and fatigue-type tests. It remains to be seen whether localized failure under a concentrated load, as experienced with steel reinforcement, can be achieved in FRC slabs in the total absence of transverse steel reinforcement, or whether some diminished amount of transverse steel within, or just below, the deck slab will still be found to be necessary.

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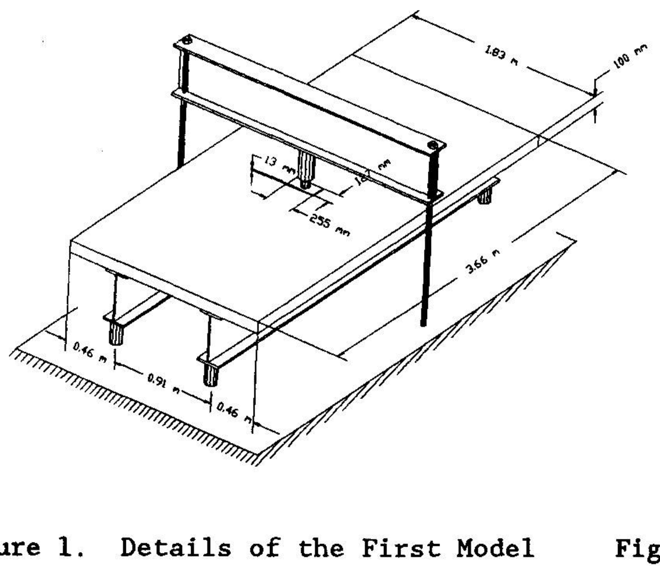

Figure 1. Details of the First Model

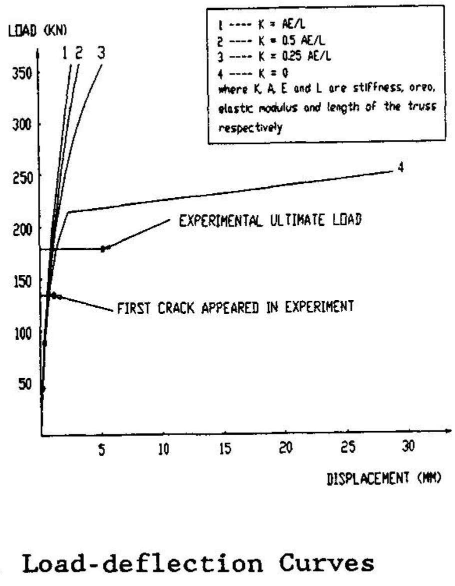

Figure 2. Load-deflection Curves

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