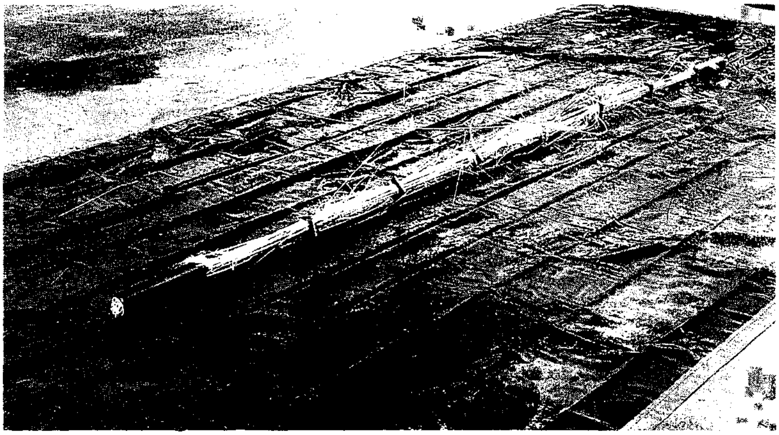

Fig. 3—HLV-anchorage after failure of tendon

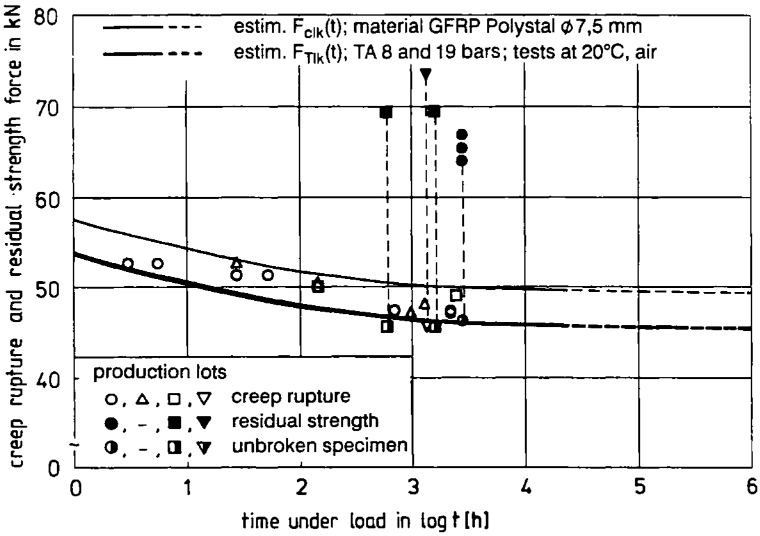

Fig. 4—Creep rupture tests with the HLV-anchorage

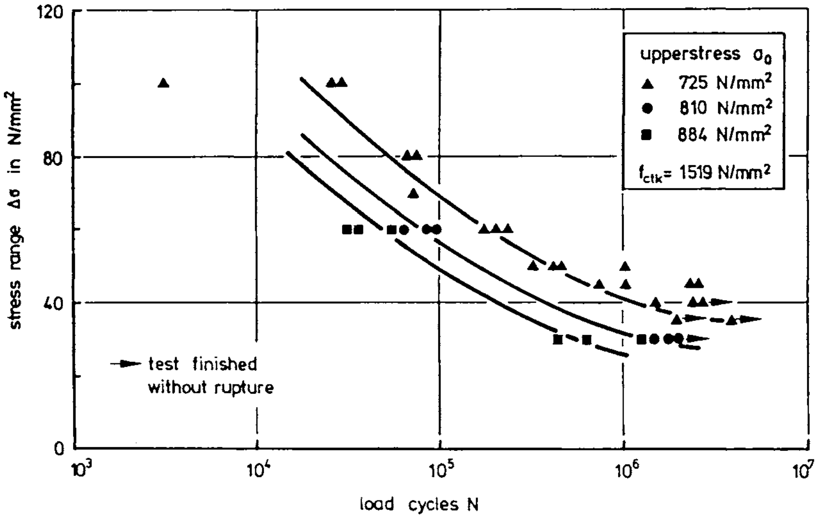

Fig. 5—S-N-lines of single Polystal bars, ϕ 7,5 mm in bond anchorage (bond length l_b = 300 mm)

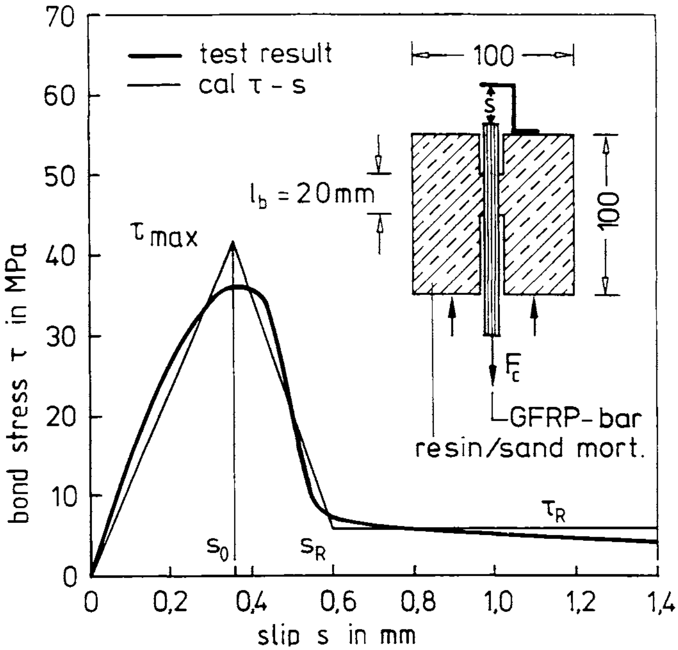

Fig. 6—Bond stress-slip relation for a GFRP bar embedded in a resin mortar; experimental relation and material law

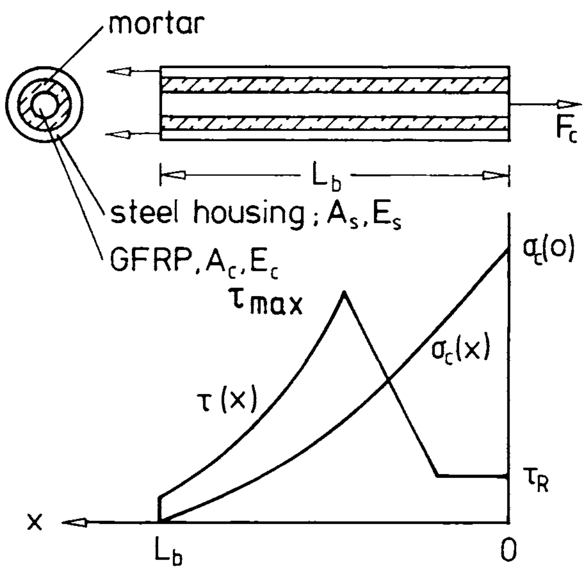

Fig. 7—Model of single bar bond anchorage. Schematic distribution of stresses

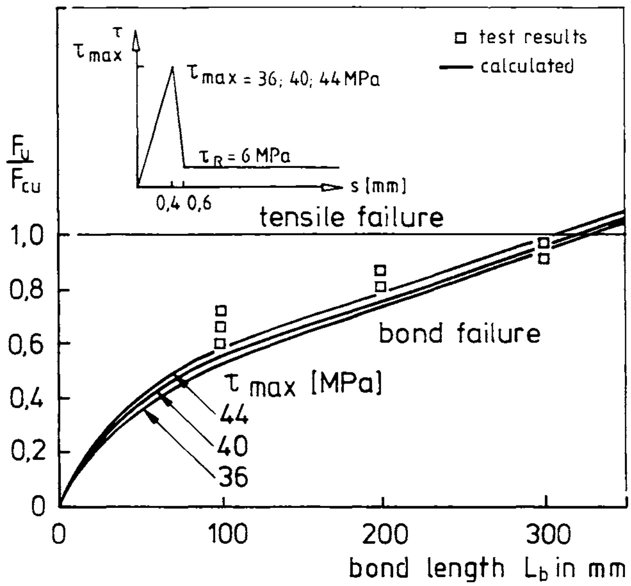

Fig. 8—Measured and calculated failure force of single bar bond anchors dependent on bond length

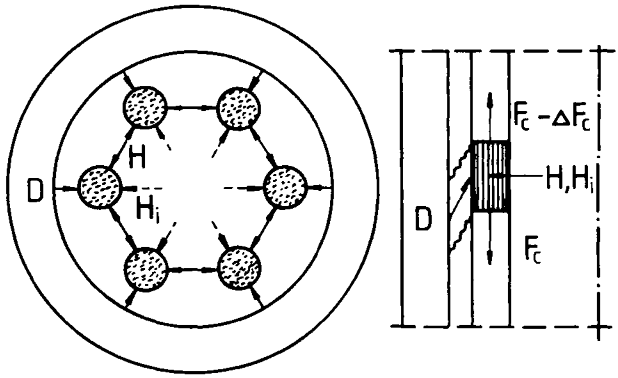

Fig. 9—Strut model for a multibar bond anchor

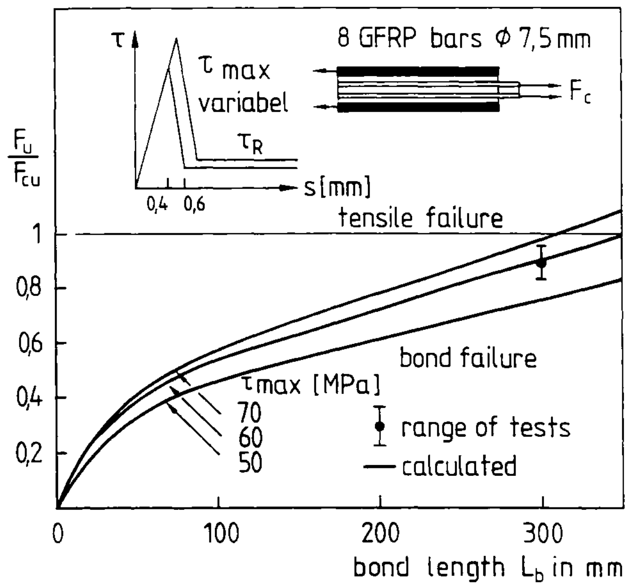

Fig. 10—Measured and calculated failure force of multibar bond anchors dependent on bond length

Ductile Behavior of Beams Using FRP as Tendons and Transverse Reinforcement

by H. Taniguchi, H. Mutsuyoshi, T. Kita, and A. Machida

Synopsis: It is known that PC members reinforced with FRP as tendons show brittle failure regardless of the failure mode. The objective of this paper is to improve the ductility of PC members reinforced with FRP as tendons. Firstly, the compressive properties of concrete confined with FRP as transverse reinforcement was investigated. Major improvement can be made in the stress-strain relationship of concrete laterally reinforced with FRP, and the concrete members could be given ductility characteristics similar to those of steel-reinforced members by confining the concrete with FRP. Secondly, several PC members reinforced with FRP as tendons and transverse reinforcements were tested and investigated. It was found that marked improvements could be made in the ductility of the PC members with FRP tendons by confining the part of concrete subjected to flexural compression with FRP and making the members undergo flexural compression failure.

Keywords: Beams (supports); confined concrete; ductility; failure; fiber reinforced plastics; flexural strength; prestressed concrete; prestressing steels; stress-strain relationships

H.Taniguchi is a senior research engineer, Civil Engineering Department, Technical Research Institute, HAZAMA Corporation, Japan. He obtained his master of engineering degree from Kanazawa Univ. in 1988. His recent research has been on new materials and mineral admixtures.

ACI member, H.Mutsuyoshi is an associate professor of Civil Engineering at Saitama Univ., Urawa, Saitama, Japan. He obtained his doctor degree from Univ. of Tokyo in 1984. His current research interests include application of FRP to concrete structures, external PC and seismic analysis of R/C structures. He is a member of ACI Committee 440

T.Kita is a chief research engineer, Civil Engineering Department, Technical Research Institute, HAZAMA Corporation, Japan. He obtained his master of engineering degree from Ehime Univ. in 1971. His recent research has been on new materials and anti-washout underwater concrete.

ACI member, A.Machida is a professor of Civil Engineering at Saitama Univ., Urawa, Saitama, Japan. He obtains his doctor degree from Univ. of Tokyo in 1976. His research interests include seismic design for R/C structures, composite structures consisting of steel and concrete and properties of fresh concrete.

INTRODUCTION

Fibre reinforced plastics (FRP) are being used in an increasing number of cases as substitutes for reinforced concrete (RC) steels and prestressed concrete (PC) steels and a large number of studies have been published on this matter. When compared with conventional steels, FRP 1) has greater tensile strengths than PC steel rods, 2) is free from corrosion which leads to lowering of strengths, 3) is free from magnetization and 4) is light. While it has these merits, it also has drawbacks, namely: 5) it has no plasticity regions in its stress-strain relationships and generally its elongation is small, and 6) its strength is reduced in regions subjected to bending during forming. Special considerations also need to be given to 7) its small Young's modulus and 8) its small linear expansion coefficients, which are neither advantages nor drawbacks. Several problems in the application of FRP to actual structures that result from the above dynamic characteristics have been pointed out, including those to do with the form of failure under ultimate conditions. Concrete members reinforced with FRP as tendons show brittle failure regardless of the failure mode. This makes the determination of the ultimate conditions an important matter. If the sizes and types of loads are known, plastic properties, or the ductility, of the structure need not necessarily. When designing structures with newly developed structural members , one need not be constrained by conventional design methods for RC; it is desirable rather to develop new design methods suited to the new materials. However, 1) as long-standing practice, types of failure for which there are warning signs are generally used in the design of RC structures, and 2) redistribution of moments may sometimes be required to prevent the collapse of the structure, while in seismic structures it is safe and rational to ensure a certain amount of ductility. These considerations make it desirable, if possible, to provide the members with ductility. It is with such considerations in the background that studies have been conducted on methods of improving the ductility of concrete members reinforced with FRP. Mutsuyoshi et al.(1) have suggested the possibility of safe and rational design by using concrete collapse as the ultimate failure form, while Akiyama et al.(2) have shown that the ductility may be raised by combining bonded and unbonded FRP or prestressed and unprestressed FRP. The methods proposed to date, however, have failed to provide the FRP-reinforced members with the mechanical properties of the type observed with RC and PC members, which undergo flexural yielding.

The objective of this study is to improve ductility of PC members reinforced with FRP as tendons. First, the compressive properties of concrete confined with FRP as transverse reinforcements was investigated. Secondly, several PC members reinforced with CFRP as tendons and transverse reinforcement were tested and investigated.

COMPRESSIVE CHARACTERISTICS OF CONCRETE CONFINED WITH FRP

Outline

It is known that the compression-deformation characteristics of concrete confined by steels tends to give larger values than that of unconfined concrete(3). Here, experimental studies were conducted on the behaviour of concrete under compression when FRP, with its different dynamic properties from steel, was used to confine the concrete.

Outline of Test

The specimens were either prisms with sections of 15 cm by 15 cm or cylinders with diameters of 15 cm, their heights in both cases being 30 cm (Figure 1). Carbon fibre reinforced plastics (CFRP) and high-strength steels were used as the transverse reinforcement. The CFRP rods were either of the 7-piece strand type or the single type (rod-like type). The transverse reinforcement was in spirals, forming rectangles 11 cm by 11 cm or 7.8 cm by 7.8 cm or circles 11 cm in diameter when viewed from above. Five test parameters of transverse reinforcement pitch, shapes, types, concrete compressive strengths, and existence of cover were used as shown in Table 1. The properties of the transverse reinforcement is given in Table 2.

In the tests, the 1960-kN capacity testing machine shown in Figure 2 was used as the loading device. This was a simple uniaxial compression tester with pin support at the top. The axial deformation of the specimens was obtained by measuring the central displacement with a compressometer with a measurement section of 15 cm and the load was applied until the specimens underwent failure or the axial strain reached about 3%.

Test Results and Discussion

Effects of Transverse Reinforcement Pitch on Stress-Strain Relationship -- Representative examples of stress-strain curves obtained in the tests are given in Figure 3. The figure shows the effects of the reinforcement pitch on specimens with covers confined with circular FRP reinforcements. It was observed that the yield strength was slightly larger with plain concrete. As regards the behaviour after yielding, with specimens confined with FRP, the strength decreases at first due to the separation of the cover concrete and the rupture of the axial erection bars, but the confining effect comes into operation once the axial strain reaches about 1% and the rate of decrease of strength becomes more gentle despite the complete separation of the cover concrete. In other words, it was found that marked improvements could be achieved in the deformation characteristics of the concrete after yielding by confining the concrete with FRP. When the pitch of the FRP transverse reinforcements was altered, it was observed that the rate of strength reduction decreased with the pitch, and especially at a pitch of 3 cm the strength did not in fact decrease but continued to rise even at a strain of 2.5%.

Effects of Transverse Reinforcement Shapes -- The effects of the transverse reinforcement shapes on the stress-strain relationship are shown in Figure 4. In the behaviour after the strain has reached about 0.7% when the cover is separated and the confining effect becomes apparent, it can be seen that the rate of strength decrease is smaller with specimens confined with circular reinforcements than with specimens with rectangular reinforcements. This is thought to be due to the fact that, whereas uniform action of tensile forces only is observed during axial compressive loading with circular reinforcements, with rectangular reinforcements bending comes into operation together with the tensile forces at the corners, and also due to the lowering of the tensile strength at the bends in the rectangular reinforcements.

Effects of Transverse Reinforcement Types -- The effects of the differences in the transverse reinforcement types (strand or single types, same diameters) on the stress-strain relationship are shown in Figure 5. No clear differences due to the reinforcement types were observed in the tests despite the differences in the effective cross-sectional areas and the direction of the fibres in the reinforcements.

Effects of Concrete Compressive Strength -- The effects of the differences in the compressive strength of the concrete on the stress-strain relationship are shown in Figure 6. The figure indicates that the rate of strength decrease after yielding is smaller in confined specimens prepared from concrete with low compressive strengths. This is the same tendency as has been observed in concrete confined with steels.

Effects of Concrete Cover -- Investigations were made on the effects of the existence of concrete cover on deformation characteristics after yielding. Shown in Figure 7 are the stress-strain curves for specimens without cover. When this is compared with Figure 3, it is to be noted that lowering of the strength due to the separation of the cover is not observed in the plastic region and the strength either shows a linear increase or is maintained at the same level; with the specimen confined at 3 cm pitches in particular, the strength continued to rise even after the strain reached around 3%. Specimens confined with high-tension steels had shown similar behaviour to those confined with FRP at the same pitches. This fact indicates that steel and FRP provide the same level of confining effect on concrete despite the differences in their dynamic characteristics. For an assessment of the effects of concrete cover, plain concrete was used in the cover, and on the assumption that the stress-strain relationship will obey the " e " function(4), the compressive force taken by the core of the confined concrete was estimated by subtracting the compressive force taken by the cover from the total compressive force. The stress-strain relationship for the core as estimated in this way for a specimen with concrete cover and the stress-strain relationship for a specimen without cover as obtained from the test are compared in Figure 8. The close agreement between the two indicates that confined concrete members with cover may be considered as composite members consisting of the core concrete surrounded by the transverse reinforcement and the plain concrete in the cover.

Stress-Strain Model of Concrete Confined with FRP

Several proposals have been made to date for model equations for the stress-strain relationship for concrete confined with steel. These existing equations, however, are not adequate for estimation of the stress-strain characteristics of concrete confined with FRP. The main reasons for this are thought to be the fact that the steel is considered to have yielded in the plastic region of the confined concrete in the model equations for specimens with steel and the fact that the elasticity coefficients of FRP differ from those of steel. An attempt was made, therefore, to propose an equation for the stress-strain relationship for concrete confined with FRP reinforcement. The method proposed by Sakai(5) for concrete confined with steels was used in devising the model, and corrections were made to parts of the equations proposed by Sakai using the test results to allow for the differences in the dynamic characteristics between FRP and steel. The corrected equations are given below. Figure 9 is a stress-strain model for confined concrete. The curve equations proposed by Muguruma et al.(6) were adopted in view of the values for the stress and strain obtained in this way. The stress-strain model for concrete confined with FRP obtained in this way is compared with examples of the test values in Figure 10. The proposed model agrees closely with the test values.

$$\sigma r = \frac{0.015 \times (\varepsilon z - \varepsilon zs)}{0.024 \times \exp(Er \times 6 \times 10^{-5})} \times Er \qquad (1)$$

$$\sigma cm = \left(1 + 4\frac{\sigma rm}{\sigma m}\right)^{0.5} \times \sigma m \qquad (2)$$

$$\varepsilon cm = \left(1 + 65\frac{\sigma rm}{\sigma m}\right) \times \varepsilon m \qquad (3)$$

$$\sigma cu = \left(1 + 6\frac{\sigma ru}{\sigma m}\right)^{0.5} \times \sigma u \qquad (4)$$

$$\varepsilon cu = 0.024 \times \exp\ (Er \times 6 \times 10^{-5}) \quad (5)$$

$$Er = (1 - S/D) \times (2 \times Aw) / (S/D) \times Es \quad (6)$$

where, σ r: lateral confining stress, ε z: axial strain, σ cm: maximum stress of confined concrete, σ m:maximum stress of plain concrete, ε cm: axial strain at maximum stress of confined concrete, ε m: strain at maximum stress of plain concrete, ε cu: ultimate stress of confined concrete, σ u: ultimate stress of plain concrete, Er: lateral confining rigidity, Es: Young's modulus for transverse reinforcement, Aw: sectional area of transverse reinforcement, S:transverse reinforcement pitch, D: diameter of confined core

BENDING CHARACTERISTICS OF PC MEMBERS USING FRP AS TENDONS AND TRANSVERSE REINFORCEMENT

Outline

It is generally held that when reinforced or prestressed concrete is subjected to bending, the yielding deformation of steel prevents sudden collapse of the structure and there will be warning signs before the structure undergoes failure. An attempt has been made here to provide members reinforced by FRP with such ductile properties. That is, a way of thinking opposite to that conventionally used with RC and PC was adopted and an attempt was made to give the members ductility by exploiting the deformation capacity of the concrete. As was discussed in the previous chapter, it was found that major improvements could be made in the compressive deformation characteristics of concrete by confining it with FRP. In the experiments described below, the main aim was to improve the ductile capacity of PC members which use FRP in their tendons and transverse reinforcement.

Outline of Test

The specimens used in the test were beam members 15 cm high, 20 cm wide and 180 cm long,as shown in Figure 11. For the tendons, 2 or 3 CFRP rods were placed at distances of 10 and 11 cm from the compressive edge. The quantities of tendons and the prestress were determined so that the ultimate failure would take the form of flexural compression failure. The properties of the CFRP used in the specimens are given in Table 3. The CFRP rods were made of entwined threads and those with diameters of 12.5 mm and 15.2 mm were used for the tendons, while those used as transverse reinforcement had a diameter of 5.0 mm. The transverse reinforcement was in the form of rectangular and circular spirals at 3 cm and 5 cm pitches respectively. This transverse reinforcement was positioned in the 70 cm section including the uniform moment segment, while in the shear section the required amounts of deformed reinforcements (D6) were placed as shear reinforcements.

Prestress corresponding to 60% of the guaranteed failure load of the CFRP was introduced 14 days after the placement of the concrete (compressive strength: 29.4 MPa or more). The prestress was introduced simultaneously via jigs with a jack by the pre-tension method on the 2 or 3 tendons.