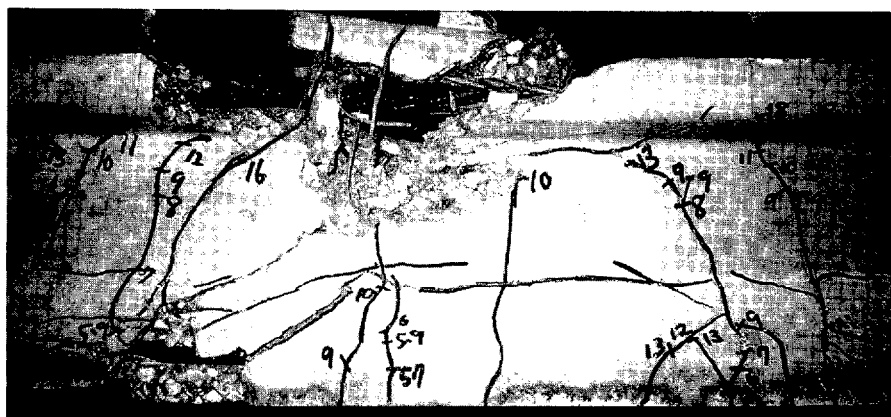
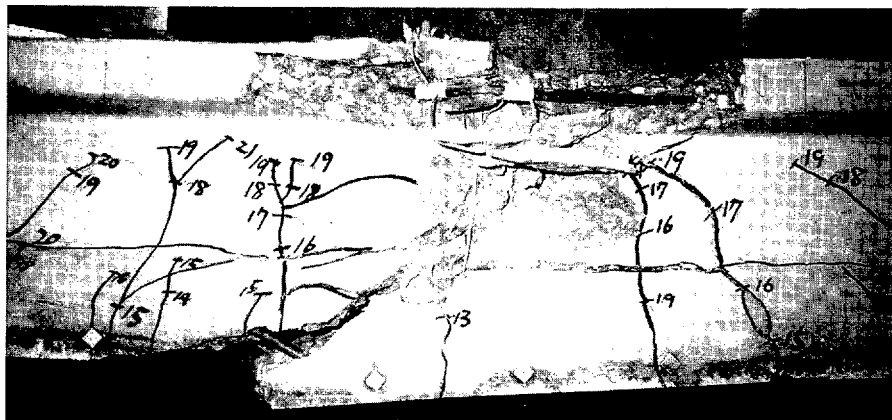


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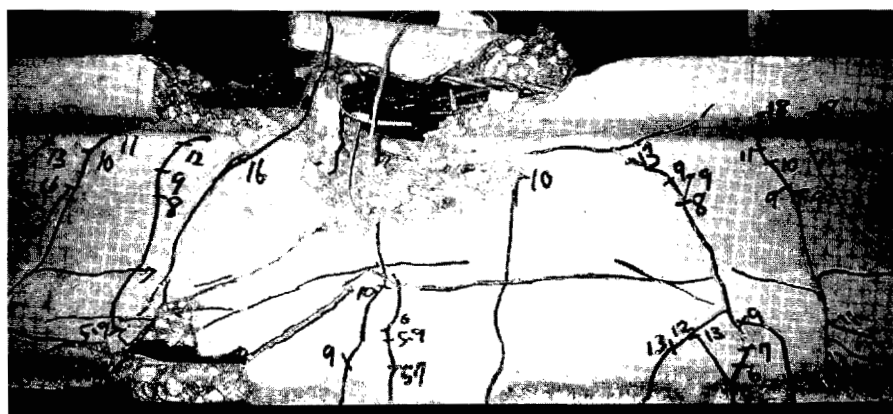
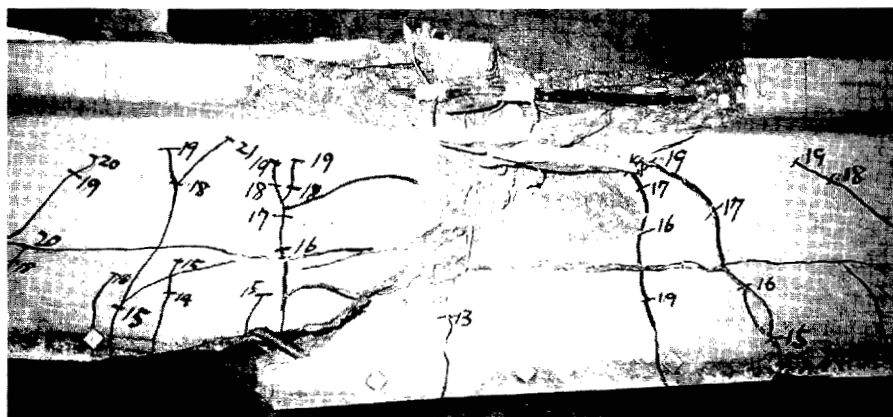


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Fiber-Reinforced-Plastic Reinforcement for Concrete Structures

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PREFACE

The American Concrete Institute sponsored an unprecedented six technical sessions on FRP reinforcement for concrete at the Vancouver Conference on March 28-31, 1993. Speakers and attendees were present from Europe, Japan, Canada and the United States. The papers in this Special Publication are organized in the same subject areas as the conference. The subject topic areas and symposium sections are:

1. FRP Material Properties and Testing Methods
2. FRP Reinforcement for Reinforced Concrete
3. FRP Reinforcement for Prestressed Concrete
4. Analysis And Design
5. The Japanese National Project for FRP Development
6. Applications of FRP Reinforcement

The 55 technical papers in this report represent the most comprehensive compilation to date of FRP research, design, and application information. A comparison of the papers provides an insight to the approach to the use and development of FRP reinforcement within the research communities of Europe, Japan and North America. The two symposium volumes are also significant because substantial portions of the extensive Japanese national research project have been translated into English. The Japanese papers provide an insight to both the magnitude of the technical work being conducted in Japan and the organization of the Japanese research program.

The technical sessions generated substantial discussion, technical concerns were articulated, and differences of philosophy emerged. The following is a synopsis of the most important discussion points raised at the symposium.

MATERIAL PROPERTIES

FRP reinforcement may be constructed from a variety of fibers and resins. Comparison of test data and development of design recommendations is dependent upon knowing the composition of the FRP reinforcement. As a minimum, this includes the fiber type, fiber characteristics, fiber volume and the resin used to bind the fibers. FRP manufacturers further may add fillers or release agents to assist in the manufacture of the FRP product.

Researchers were encouraged to provide as complete identification of the FRP reinforcement as possible when reporting project results. Closer collaboration with FRP specialists is needed in order to develop systems, resins in particular, which complement the fiber properties and have the desired properties for concrete reinforcement. It is possible to capitalize on the knowledge acquired by composite industry in the last 30 years through such a collaborative effort.

CREEP-RUPTURE

Creep-rupture, the tendency of a material subjected to a sustained load to fail at a load much lower than its static strength, was identified as one of the principal differences between FRP and steel reinforcement. The time to failure of an FRP tendon is a function of the sustained stress level. Consequently, the selection of a sustained stress level is equivalent to selecting a true service life for the structure. Available test data are insufficient to fully define the creep-rupture performance of FRP materials currently being used. Some statistical testing to define the time dependent behavior of FRP reinforcement is in progress and accelerated testing promises to provide additional insight to the creep-rupture performance. During the discussion there was general agreement among the researchers that sustained loads in the range of 50-60 percent of the static tensile strength of FRP reinforcement are sufficiently low that a 100 year structure life is likely.

DUCTILITY

There was a general lack of agreement on the definition of ductility as it applies to FRP reinforced structures. Since the reinforcement remains elastic to failure, energy absorption remains mainly in the elastic range and plastic dissipation of energy is limited. It was agreed that FRP reinforced structures can be designed to undergo substantial deformation prior to failure, and that these structures give ample warning of overload. At the same time, plastic energy absorption is achieved primarily by fracturing the concrete. Methods to incorporate additional ductility, such as hybrid tendons, controlled bond failure, partial prestressing or staggered reinforcement stress levels need to be incorporated into design and testing programs.

DURABILITY

FRP reinforcement is extolled as superior to steel because of its superior corrosion resistance. Accelerated aging tests on aramid, carbon and glass fiber based reinforcement were reported. These accelerated aging tests examined FRP response when exposed to concentrated chloride and alkali solutions. Aramid and carbon based products indicated excellent resistance to these environments. Several researchers reported serious loss of strength and fiber cross section for glass FRP reinforcement. The alkali corrosion of glass

led researchers to a heated discussion of the extraordinary methods which may be necessary to protect glass FRP reinforcement. Long term durability of FRP reinforcement is the single most important factor needed for the successful application of composites as reinforcement to concrete. It is imperative that questions and doubts related to specific systems be addressed before field implementation of FRP materials is initiated.

REINFORCEMENT FORM

FRP systems consisting of 1-D, 2-D, and 3-D shapes were discussed. The 2-D and 3-D shapes are a significant innovation with respect to conventional steel reinforcement. Manufacturing techniques for composites allow for the development of multi-dimensional systems that hold out exceptional promise for considerable installation and labor savings.

FIRE RESISTANCE

A widespread belief that FRP reinforcement could not meet fire resistance requirements was challenged by some Japanese research. FRP prestressed concrete members were subjected to standard fire tests. While no firm conclusions were provided, some data would suggest that FRP prestressed members came close to meeting a 2 hour fire rating. Additional research on FRP at elevated temperature indicated that below 250 degrees centigrade, FRP reinforcement regains its full tensile strength upon cooling.

JAPANESE NATIONAL PROJECT

The format and organization of the Japanese National Project for the development of FRP reinforcement was presented. The five year project involved funding from the government and private industry. The private industry contribution includes contractors, FRP manufacturers and fiber suppliers and firms normally associated with distributing the final product. Research is conducted by government laboratories, universities and construction company R&D departments. Consequently, the entire country is involved in the development effort and the information is quickly disseminated to all participants.

ACKNOWLEDGMENTS

No work of this magnitude is possible without the help of many contributors. A special recognition is due to the Symposium Steering Committee whose efforts were invaluable in organizing the sessions and the paper review process.

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The editors also wish to extend their thanks to the 48 individuals who served as technical reviewers for these papers. Their efforts substantially helped improve the quality of the presentations.

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CONTENTS

PREFACE	
by A. Nanni and C. Dolan	iii
COMMITTEE MEMBERSHIP	
	vii
FRP MATERIAL PROPERTIES AND TESTING METHODS	
SPLITTING FORCES IN FRPR PRETENSIONED CONCRETE	
by W.R. de Sitter and R.A. Vonk	1
INVESTIGATION OF S-2 GLASS/EPOXY STRANDS IN CONCRETE	
by R. Sen, D. Mariscal, and M. Shahawy	15
CHARACTERISTICS OF ARAMID FRP RODS	
by K. Mukae, S. Kumagai, H. Nakai, and H. Asai	35
BOND CHARACTERISTICS OF FRP ROD AND CONCRETE AFTER FREEZING AND THAWING DETERIORATION	
by M. Mashima and K. Iwamoto	51
EVALUATION OF FRP AS REINFORCEMENT FOR CONCRETE BRIDGES	
by A.H. Rahman, D.A. Taylor, and C.Y. Kingsley	71
CREEP RUPTURE BEHAVIOR OF FRP ELEMENTS FOR PRESTRESSED CONCRETE — PHENOMENON, RESULTS AND FORECAST MODELS	
by H. Budelmann and F.S. Rostásy	87
PROPERTIES OF FIBER REINFORCED PLASTIC RODS FOR PRESTRESSING TENDONS	
by T. Uomoto and H. Hodhod	101

EVALUATION ITEMS AND METHODS OF FRP REINFORCEMENT AS STRUCTURAL ELEMENTS	
by S. Mochizuki, Y. Matsuzaki, and M. Sugita	117

MECHANICAL PROPERTIES OF COMPOSITE BEAMS BY FRP	
by E. Sueoka, K. Yasuoka, O. Kiyomiya, M. Yamada, M. Shikamori	133

FRP REINFORCEMENT FOR REINFORCED CONCRETE

ARAMID FABRIC AS A REINFORCEMENT FOR CONCRETE	
by J. Kasperkiewicz and H.W. Reinhardt	149

EXPERIMENTAL STUDY ON TENSILE STRENGTH OF BENT PORTION OF FRP RODS	
by T. Maruyama, M. Honma, and H. Okamura	163

RESEARCH AND DEVELOPMENT OF GRID SHAPED FRP REINFORCEMENT	
by T. Fujisaki, T. Nakatsuji, and M. Sugita	177

LATERAL CONFINEMENT OF CONCRETE USING FRP REINFORCEMENT	
by A. Nanni, M.S. Norris, and N.M. Bradford	193

APPLICATION OF THREE-DIMENSIONAL FABRIC REINFORCED CONCRETE TO BUILDING PANELS	
by H. Nakagawa, M. Kobayashi, T. Suenaga, T. Ouchi, S. Watanabe, and K. Satoyama	211

STRENGTHENING OF REINFORCED CONCRETE CHIMNEYS, COLUMNS AND BEAMS WITH CARBON FIBER REINFORCED PLASTICS	
by C. Ballinger, T. Maeda, and T. Hoshijima	233

BEHAVIOR OF EXTERNALLY CONFINED CONCRETE COLUMNS	
by H. Saadatmanesh, M.R. Ehsani, and M.W. Li	249

FLEXURAL BEHAVIOR OF CEMENTITIOUS COMPOSITES REINFORCED BY PITCH-BASED HIGH MODULUS CONTINUOUS CARBON FIBERS	
by T. Yamada, K. Yamada, and K. Kubomura	267

**CONTROL OF CRACKING BY USE OF CARBON FIBER
NET AS REINFORCEMENT FOR CONCRETE**
by T. Makizumi, Y. Sakamoto, and S. Okada 287

**ABSORBING CAPACITY OF CUSHION SYSTEM USING
CONCRETE SLAB REINFORCED WITH AFRP RODS**
by T. Tamura, H. Mikami, O. Nakano, and N. Kishi 301

**INVESTIGATION OF BOND IN CONCRETE MEMBER WITH
FIBER REINFORCED PLASTIC BARS**
by E. Makitani, I. Irisawa, and N. Nishiura 315

**BOND OF GFRP REBARS TO ORDINARY-STRENGTH
CONCRETE**
by M.R. Ehsani, H. Saadatmanesh, and S. Tao 333

FRP REINFORCEMENT FOR PRESTRESSED CONCRETE

**FRP TENSILE ELEMENTS FOR PRESTRESSED CONCRETE —
STATE OF THE ART, POTENTIALS AND LIMITS**
by F.S. Rostásy 347

SHOULD FRP BE BONDED TO CONCRETE?
by C.J. Burgoyne 367

**EPOXY SOCKETED ANCHORS FOR NON-METALLIC
PRESTRESSING TENDONS**
by L.E. Holte, C.W. Dolan, and R.J. Schmidt 381

**BEHAVIOR OF PRESTRESSED CONCRETE
BEAMS USING FRP AS EXTERNAL CABLE**
by H. Mutsuyoshi and A. Machida 401

**FLEXURAL CHARACTERISTICS OF PRESTRESSED
CONCRETE BEAMS WITH CFRP TENDONS**
by T. Kato and N. Hayashida 419

**PARTIALLY PRESTRESSED BEAMS WITH CARBON FIBER
COMPOSITE STRANDS: PRELIMINARY TESTS EVALUATION**
by A.E. Naaman, K.H. Tan, S.M. Jeong, and F.M. Alkhairi 441

**PROPERTIES OF HOLLOW CONCRETE MASONRY
REINFORCED WITH FIBERGLASS COMPOSITE**
by A. Hamid, J. Larralde, and A. Salama 465

**PRESTRESSED CONCRETE BEAMS USING NON-METALLIC
TENDONS AND EXTERNAL SHEAR REINFORCEMENT**
by C.W. Dolan, W. Rider, M.J. Chajes, and M. DeAscanis 475

**FLEXURAL BEHAVIOR OF MASONRY WALLS
STRENGTHENED WITH COMPOSITE FABRICS**
by M.R. Ehsani, H. Saadatmanesh, I.H. Abdelghany,
and W. Elkafrawy 497

**FLEXURAL FATIGUE BEHAVIOR OF PRESTRESSED
CONCRETE BEAMS USING ARAMID-FIBER TENDONS**
by K. Iwamoto, Y. Uchita, N. Takagi, and T. Kojima 509

**FLEXURAL AND SHEAR BEHAVIOR OF PRESTRESSED
CONCRETE BEAMS USING FRP RODS AS
PRESTRESSING TENDONS**
by A. Yonekura, E. Tazawa, and H. Nakayama 525

ANALYSIS AND DESIGN

**DESIGN CONCEPT FOR CONCRETE MEMBERS USING
CONTINUOUS FIBER REINFORCING MATERIALS**
by H. Okamura, Y. Kakuta, T. Uomoto, and H. Mutsuyoshi 549

**PULTRUDED FRP GRATING REINFORCED
CONCRETE SLABS**
by L.C. Bank and Z. Xi 561

**FLEXURAL BEHAVIOR AND ENERGY ABSORPTION OF
CARBON FRP REINFORCED CONCRETE BEAMS**
by T. Kakizawa, S. Ohno, and T. Yonezawa 585

**THEORETICAL AND EXPERIMENTAL CORRELATION
OF BEHAVIOR OF CONCRETE BEAMS REINFORCED
WITH FIBER REINFORCED PLASTIC REBARS**
by S.S. Faza and H.V.S. GangaRao 599

**SHEAR CAPACITY OF RC AND PC BEAMS USING
FRP REINFORCEMENT**
by S. Tottori and H. Wakui 615

**PRINCIPLES OF DESIGN OF FRP TENDONS AND
ANCHORAGES FOR POST-TENSIONED CONCRETE**
by F.S. Rostásy and H. Budelmann 633

**DUCTILE BEHAVIOR OF BEAMS USING FRP AS
TENDONS AND TRANSVERSE REINFORCEMENT**
by H. Taniguchi, H. Mutsuyoshi, T. Kita, and A. Machida 651

**ULTIMATE STRENGTH AND DEFORMATION
CHARACTERISTICS OF CONCRETE MEMBERS
REINFORCED WITH AFRP RODS UNDER COMBINED
AXIAL TENSION OR COMPRESSION AND BENDING**
by N. Kawaguchi 671

**EVALUATION OF PERMISSIBLE CRACK WIDTH ON
FRP REINFORCED CONCRETE MEMBER**
by Y. Kaneko, H. Seki, M. Matsushima, and K. Matsui 685

THE JAPANESE NATIONAL PROJECT FOR FRP DEVELOPMENT

**OUTLINE OF THE NATIONAL RESEARCH PROJECT ON
THE USE OF FRP REINFORCEMENT IN CONCRETE
BUILDING STRUCTURES IN JAPAN**
by Y. Sonobe 695

FUNDAMENTAL PROPERTIES OF CONTINUOUS FIBER BAR
by Y. Yamasaki, Y. Masuda, H. Tanano, and A. Shimizu 715

**TENSILE STRENGTH OF CONTINUOUS FIBER BAR UNDER
HIGH TEMPERATURE**
by S. Kumahara, Y. Masuda, H. Tanano, and A. Shimizu 731

**FLEXURAL PERFORMANCE OF CONCRETE BEAMS
REINFORCED WITH CONTINUOUS FIBER BARS**
by K. Nakano, Y. Matsuzaki, H. Fukuyama, and M. Teshigawara .. 743

**BOND PERFORMANCE OF CONCRETE MEMBERS
REINFORCED WITH FRP BARS**
by T. Kanakubo, K. Yonemaru, H. Fukuyama, M. Fujisawa,
and Y. Sonobe 767

**SHEAR PERFORMANCE OF CONCRETE BEAMS
REINFORCED WITH FRP STIRRUPS**
by T. Nagasaka, H. Fukuyama, and M. Tanigaki 789

APPLICATIONS OF FRP REINFORCEMENT

FRP DOWEL BARS IN REINFORCED CONCRETE PAVEMENTS

by V.L. Brown and C.L. Bartholomew 813

ADVANCED COMPOSITE DEMONSTRATION BRIDGE DECK

by S.L. Iyer 831

PRACTICAL APPLICATIONS OF ARAMID FRP RODS TO PRESTRESSED CONCRETE STRUCTURES

by K. Noritake, K. Mukae, S. Kumagai, and J. Mizutani 853

PRACTICAL APPLICATION AND PERFORMANCE OF PPC BEAMS REINFORCED WITH BRAIDED FRP BARS

by T. Okamoto, S. Matsubara, M. Tanigaki, and K. Hasuo 875

REPORT ON THE USE OF CFCC IN PRESTRESSED CONCRETE BRIDGES IN JAPAN

by N. Santoh, H. Kimura, T. Enomoto, T. Kiuchi, and Y. Kuzuba . . 895

STRENGTHENING OF DAMAGED CONCRETE BEAMS BY EXTERNAL PRESTRESSING OF ARAMID FIBER CABLE

by N. Saeki, T. Horiguchi, M. Inomata, S. Hata, and T. Ikeda 913

RC SLABS STRENGTHENED BY BONDED CARBON FRP PLATES: PART 1 — LABORATORY STUDY

by H. Ichimasu, M. Maruyama, H. Watanabe, and T. Hirose 933

RC SLABS STRENGTHENED BY BONDED CARBON FRP PLATES: PART 2 — APPLICATION

by H. Ichimasu, M. Maruyama, H. Watanabe, and T. Hirose 957

SI (Metric) TABLES 971

INDEX 973

SP 138-1

Splitting Forces in FRPR Pretensioned Concrete

by W.R. de Sitter and R.A. Vonk

Synopsis: In concrete pretensioned with non-metallic Fiber Reinforced Plastic Reinforcement (FRPR) the Hoyer effect leads to high splitting stresses due to confinement of radial deformations of bars or strands in the transfer zone. Incompatible linear temperature expansion can aggravate the splitting stresses. Bond in the transfer zone is heavily influenced by the confined radial expansion as demonstrated by tests with Arapree bars in light weight concrete. Very short transfer lengths (80 mm) have been measured. Three calculation approaches for splitting stresses are presented; the Elasto Plastic approach, the Concrete Deformation approach and the Fracture Energy approach. The Elasto Plastic model has been checked using a discrete element model including tensile softening of concrete. The presented formula are confirmed by a few tests on Arapree pretensioned prisms.

Keywords: Fiber reinforced plastics; lightweight concrete; pretensioning; splitting stresses; stress transfer

2 de Sitter and Vonk

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INTRODUCTION

The transfer of prestress in pretensioned concrete leads to tensile splitting stresses in the anchorage zone. These can be attributed to two causes (fig. 1).

1. The Hoyer effect; release of pretension causes radial expansion of the prestress material due to it's Poisson coefficient. This leads to radial compressive stresses resulting in increased bond in the anchorage zone.
2. Transfer of bond stresses along a prestress bar or strand in the anchorage zone introduces tensile stresses in the tangential direction in the concrete.

The Hoyer effect is restrained by the concrete leading to radial stresses p_r normal to the interface of the bar/strand with the concrete. A high level of p_r increases the bond stresses and the angle ϕ . Therefore both causes of splitting forces are interrelated. In section 3 it will be shown that even in the case of smooth surface texture and very low bond the Hoyer effect may lead to splitting. At the extreme end of the anchorage zone the shear stress τ_{tr} must be zero because τ_{r1} is zero at the free concrete surface. In this area the Hoyer effect will dominate the splitting forces. The material properties make FRPR sensitive to the Hoyer effect.

The coefficient of radial thermal expansion α_r of FRPR is governed by the (more or less) linear thermal expansion of the bonding resin. In many types of FRPR the value of α_r is 5 to 8 times the value for concrete. An increase in temperature then will lead to tangential splitting stresses in the concrete. The combination of thermal stresses with stresses caused by the Hoyer effect and prestress force transfer did lead to severe longitudinal cracking in thin pretensioned elements (fig.2).

Acknowledgement

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THE HOYER EFFECT COMBINED WITH TEMPERATURE EXPANSION

The E-modulus E_l in the longitudinal direction of FRPR is governed by the E-modulus E_{1f} of the fibers. The E-moduli vary from 80 000 to 140 000 MPA for aramid fibers and 4000 to 6000 MPA for resins. If the contribution of the resin is neglected:

$$E_l = A_f E_{1f} / A_1$$

The longitudinal strain ε_{1f0} and the radial contraction ε_{r0} in prestensioned state:

$$\varepsilon_{10} \approx \sigma_{1f0} / E_{1f} = \sigma_{10} / E_l$$

$$\varepsilon_{r0} = \nu_r \varepsilon_{10}$$

The radial contraction is governed by the poisson coefficient ν_{res} of the resin if we neglect the influence of the radial contraction of the fibers.

After release of pretension the expansion of the resin in the longitudinal direction is restrained by the concrete and by the fibers.

Transfer zone:

$$\varepsilon_{r1} = \nu_{res} \varepsilon_{10} + (\alpha_{res} - \alpha_c) \Delta T \quad (1)$$

and

$$p_{r1} = \varepsilon_{r1} E_{res} / (1 - \nu_{res}) \quad (2)$$

After the transfer zone radial expansion due to release of pretension does not play a role but longitudinal deformation of the resin is restrained by the fibers as well as the surrounding concrete. Hence, the apparent modulus varies between $E_{res}/(1 - \nu_{res})$ and $E_{res}/(1 - \nu_{res} - 2\nu_{res}^2)$

After the transfer zone:

$$\varepsilon_{r1} = +(1 + \nu_{res})(\alpha_{res} - \alpha_c) \Delta T \quad (3)$$

After the transfer:

$$p_{r1} = \varepsilon_{r1} E_{res} / (1 - \nu_{res} - 2\nu_{res}^2) \quad (4)$$

These expressions may be refined to include combined moduli of fibers and resin according to volumetric laws. However even a small amount of enclosed air may have a large influence on composite moduli in the transverse direction. Therefore composite moduli must be measured on representative samples; they can only be approximated from the moduli of the constituent materials.

4 de Sitter and Vonk

Comparing expression (2) and (4) it follows that, depending on the relative relation between the Hoyer stress and the temperature induced stresses cracking will start either in the anchorage zone or at some distance from the free end.

PULL-OUT AND PUSH-IN BOND STRENGTH

In pretensioned concrete bond in the transfer zone is identical with bond as observed in a push-in test. However in the ultimate load situation bond in the anchorage zone is more analogous with a pull-out condition.

The influence of transverse expansion and contraction on bond has been investigated by comparison of bond in pull-out and push-in tests (fig.3) [1]. These tests were too few to permit firm quantitative conclusions but they indicate significant differences in bond. In the push-in tests arapree bars (nominal: 8 mm) with embedded sand were tensioned to values varying between 23.64 and 26.46 kN. The circular samples of lightweight concrete were cast around these bars with cover 21 to 35 mm. After 3 days wet curing the tensile splitting strength f_{ct} was 1.6 to 2.5 MPa. Then the force F_1 in the upper part of the bar was gradually released. The reaction forces were measured as a check on $(F_2 - F_1)$ as well as the slip of the bar with respect to the l.w. concrete. In the pull-out tests the bars were stressed to a force 2.4 - 6.6 kN. Then the force F_2 in the lower half of the bar was increased. The mean bond stress τ was evaluated taking into account the embedded length of 35 mm, the actual circumference of the bar and the peak value of $(F_2 - F_1)$. The maximum observed values τ_{max} averaged over 35 mm are listed in Table 1. Visible cracking and crack pattern were noted.

As expected the push-in tests showed more cracking than the pull-out tests. In series 3 (the only non cracked push-in test) the average push-in bond stress was 11.07 MPa as compared to 6.74 MPa in the case of the corresponding pull-out test.

TRANSFER LENGTH AND SPLITTING

Transfer length and splitting were further investigated [2] in tests on 300 mm long prisms pretensioned with 5.7 mm Arapree bars at a nominal jacking force of 17 kN. Four types of surface texture were included, varying from perfectly smooth to coarse embedded sand. Prisms had sides of 30, 35, 40 and 45 mm.

In table 2 the occurrence of cracking is summarized. The 40*40 prism showed cracks (fig. 4) after an interval varying between 1 minute to 3 months after testing. Typical strains measured at the concrete surface are given in fig. 5. The considerable spread is attributed to local differences in concrete E-modulus due to aggregate distribution.

The modulus E_{fr} of the fibers is 120 000 MPa and the cross section of 100 000 fibers in the 5.7 mm bar is 11.1 mm².

$$\epsilon_{10} = 17000 / (11.1 \times 120000) = 1.28\%$$

$$\epsilon_{r0} = -\nu \epsilon_{10} = -0.35 \times 1.276\% = -0.45\%$$

$$p_{r1} = 0.0045 \times 4000 / (1 - 0.35) = 28 \text{ MPa}$$

CALCULATIONS

Cracking due to het Hoyer effect has been simulated by Van Gils [2] using the computer program UDEC. The effect of splitting stresses in two dimensional cross-sections of square and circular prisms were investigated. Only the results for a circular cross-section are discussed here, as they differ only slightly from the results for a square cross-section and can be used for comparison with simple design formulas. Three discrete radial cracks were modelled in the simulations, since the tests almost always showed failure due to the formation of three cracks.

Conclusions:

- a] Embedded sand in the surface is essential to transfer of pretension. However, if some sand is embedded the amount and coarseness of the sand has not a dominant influence on the transfer length, which varies between 60 and 120 mm. (In [3] a transfer length of 135 mm in the case of 20*2,6 mm² flat Arapree strips was reported).
- b] Prisms with smooth bars show a very reduced transfer of pretension but they crack nonetheless. Thus showing the predominant influence of the Hoyer effect on cracking.

This is supported further by the facts that (1) three is the smallest number of cracks for which a kinematically possible failure mechanism is created (fig. 7), (2) the number of cracks is limited to three to five by the large aggregates ($d_{max} = 8 \text{ mm}$) and (3) three cracks result in the lowest peak stress and the most brittle post-peak behaviour. Since concrete cracking is not perfectly brittle, a stress-crack-opening relation with linear softening up to $\delta_u = 0.03 \text{ mm}$ was used in the simulations (Fig. 6b, [4]).

Fig. 8 gives the residual pressure in the arapree as a function of the initial pressure. It shows that failure changes from ductile to brittle when the diameter of the concrete prism increases. Failure takes place due to the growth of the radial cracks from the inside to the outside. The maximum residual pressure is found when the crack front reaches the outside of the prism.

6 de Sitter and Vonk

Due to the relatively small size of the prisms, the average opening of the cracks is still small when this happens, which means that the crack stresses have not entered far in the softening branch of fig. 6b and still have a significant value over the total length of the cracks.

The simulations indicate that an elastoplastic model can give good predictions of the maximum allowable initial pressure in the arapree. For $(d_c/d)^2 \gg 1$ the theory of elasticity yields:

$$p_{r,i} = \left(1 + \frac{E_{res}(1+\nu_c)}{E_c(1-\nu_{res})}\right) p_r$$

with $d_c = 2r_o = 2(r_i + c)$

This relation is shown in fig. 8 as the tangent in the origin. According to the theory of plasticity, the maximum residual pressure is:

$$p_r = (d_c - d) f_{ct} / d \quad (5.2)$$

An estimate of the maximum allowable pressure in the arapree is found by calculating the intersection point between the elastic and the plastic line, which gives

$$p_{r,max} = 2c \left(1 + \frac{E_{res}(1+\nu_c)}{E_c(1-\nu_{res})}\right) f_{ct} \quad (5.3)$$

The result for $f_{ct} = 3.4$ MPa, $E_{res} = 4000$ MPa and $E_c = 30\,000$ MPa is shown in fig. 8.

According to an energy approach, failure can only take place when the initial elastic deformation energy in the arapree:

$$E_{ea} = \frac{\pi(1-\nu_{res})}{4 E_{res}} p_{r,i}^2 d^2 \quad (5.4)$$

is at least equal to the energy needed to create the three radial cracks

$$E_{fc} = 3c G_{fc} \quad (5.5)$$

This results in

$$p_{r1max} = \left(\frac{12c E_{res} G_{fc}}{\pi d^2 (1-\nu_{res})} \right)^{1/2} \quad (5.6)$$

The fracture energy G_{fc} can be determined in tests and is equal to $0.5 f_{ct} \delta_u$ in the simulations with the computer program UDEC (see fig. 6b)

In a third approach the minimum expansion Δd of the arapree upon relief of the prestress, necessary to find a crack opening δ_u for the three cracks, can be calculated. Because the concrete pieces move as rigid blocks, as they are stress-free before and after the relief of the prestress, this calculation results in the simple relation.

$$2\Delta r_i = \Delta d = \frac{\delta_u}{\sqrt{3}} \quad (5.7)$$

According to equation 2 this expansion corresponds to an initial pressure in the arapree:

$$p_{rimax} = \frac{E_{res}}{(1-\nu_{res})} \frac{\delta_u}{d\sqrt{3}} \quad (5.8)$$

The results of the three approaches are compared in fig 9 for $d = 5.7$ mm. The fact that the curves for the energy approach and the deformation approach give higher allowable pressures for small diameters of the concrete prism than the elastoplastic approach indicates the ductile character of the failure for those diameters. For larger diameter a more brittle failure is predicted by the models. This is all in accordance with the results of the computer simulations. The preferable design curve is that of the elastoplastic approach because it guarantees a residual stress which is close to the maximum. It will result in the highest shear-stress transfer and thus in the shortest transfer length of the prestress in the anchorage zone. Attention should be given to the required safety factor as the failure of the concrete prisms is brittle for large diameters and a delayed crack growth is observed in the tests of section 4.

Fig. 9 gives also a comparison of the model predictions and the test results of section 4. It shows that the test results agree with the elastoplastic model. The deformation approach is found to be very conservative. One additional comparison with test results can be made. Concrete prisms with a square cross-section of 25×25 mm² made of a special high-strength Densit mortar ($f_{ct} = 11$ MPa, $f_{cc} = 180$ MPa, $E_c = 65000$ MPa) were also pretensioned with a force of 17 kN ($p_i = 27.5$ MPa). The elastoplastic model, the energy model and the deformation model predict an allowable initial stress p_i of 41.5, 33.9 and 18.7 MPa, respectively.

8 de Sitter and Vonk

The successful production of the prisms agrees with the first two models. The prisms cracked when they were submerged in water of 60°C. According to the equations in section 2, the rise in temperature increased the initial pressure in the arapree to 42.3 MPa, which is more than the elastoplastic model and the energy model allow.

Conclusions

Three approaches to calculate the critical initial confined radial pressure $p_{r,max}$ have been presented; the deformation criterium, the elasto plastic criterium and the fracture energy criterium. Based on a very limited number of tests and practical experience the following preliminary conclusions are drawn.

- 1) In FRPR pretensioned concrete the confined radial deformations (Hoyer effect) are critical for splitting forces.
- 2) The critical confined pressure can be calculated using the elasto plastic criterium (5.3)
- 3) A safer value for $p_{r,max}$ may be obtained by comparison of the elasto plastic criterium and the fracture energy criterium (5.6) and taking the smallest value.
- 4) Due to interaction with the Hoyer effect there is a significant difference between bond in push-in tests and bond in pull-out tests. Hence transfer lengths should either be directly measured or based on measurements of bond in push-in tests.

NEW DEVELOPMENTS

AKZO and HBG have developed a compressible coating for FRPR (as well as epoxy coated strands) in order to reduce radial stresses due to the Hoyer effect and incompatible temperature expansion coefficients. Short duration tests have shown that this coating solves the problem. There is an influence on bond but transfer lengths remain significantly less than for steel strands. Long duration tests are in progress. If these tests are successful a major obstacle to the use of FRPR in thin pretensioned concrete elements will have been removed. A patent application has been submitted.

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SYMBOLS

c	cover ; mm
d	bar diameter ; mm
d_c	concrete cylinder diameter ; mm
f_{ct}	tensile spl.strength; MPa
p	pressure; MPa
r_i	radius of bar; mm
r_o	outer radius of cylinder; mm
A	area ; mm ²
E	Young's moduljus; MPa
E_{def}	deformation energy; Nmm/mm
F	force ; N
G_f	fracture energy; Nmm/mm
α	thermal expansion; 1/°C
δ	crack opening ; mm
ε	strain
ϕ	angle
ν	Poisson coefficient
τ	shear stress; MPa
σ	normal stress; MPa
ΔT	temp. difference ; °C
Δr_i	radial expansion; mm

indices:

c	concrete
f	fiber
res	resin
r	radial direction
l	longitudinal
t	tangential
i	initial

10 de Sitter and Vonk

TABLE 1 — BOND IN PULL-OUT/PUSH-IN CONDITIONS

series	Pull-Out					Push-in				
	1	2	3	4	5	1	2	3	4	
cover	21	28	35	35	35	21	28	35	35	mm
cracks	yes	no	no	no	no	yes	yes	no	yes	
τ_{max}	8.4	4.77	6.74	7.89	7.20	6.89	6.25	11.07	8.78	MPa
f_{ct}	2.35	2.37	1.61	2.41	2.30	2.10	2.45	1.61	2.41	MPa
F_1	3.94	2.47	5.85	6.59	0	20.4	20.18	13.9	18.1	kN
F_2	11.33	6.67	11.78	13.53	6.34	26.64	25.68	23.64	2582	kN

TABLE 2 — OCCURRENCE OF CRACKING

texture	30 x 30 mm ²	35 x 35 mm ²	40 x 40 mm ²	45x 45 mm ²
coarse	yes	yes	later	no
normal	yes	yes	later	no
fine	yes	yes	no	no
smooth	yes	yes	later	no

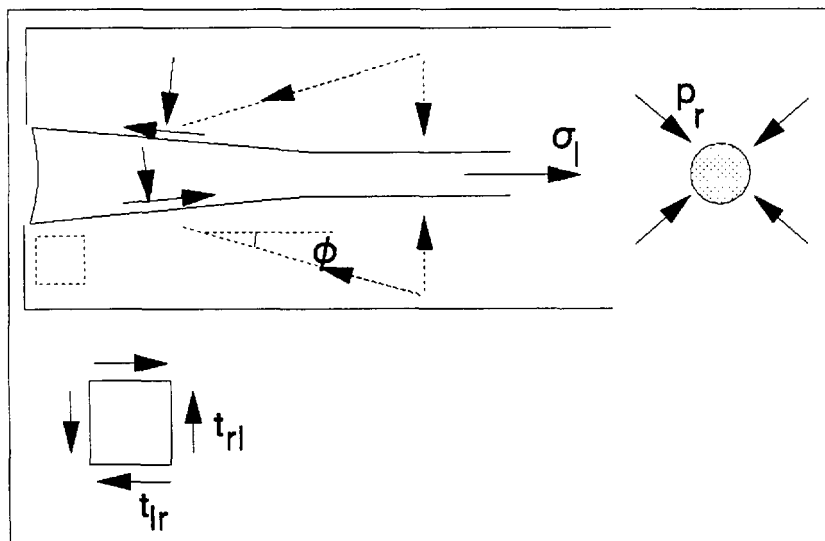


Fig. 1—Hoyer effect



Figure 2

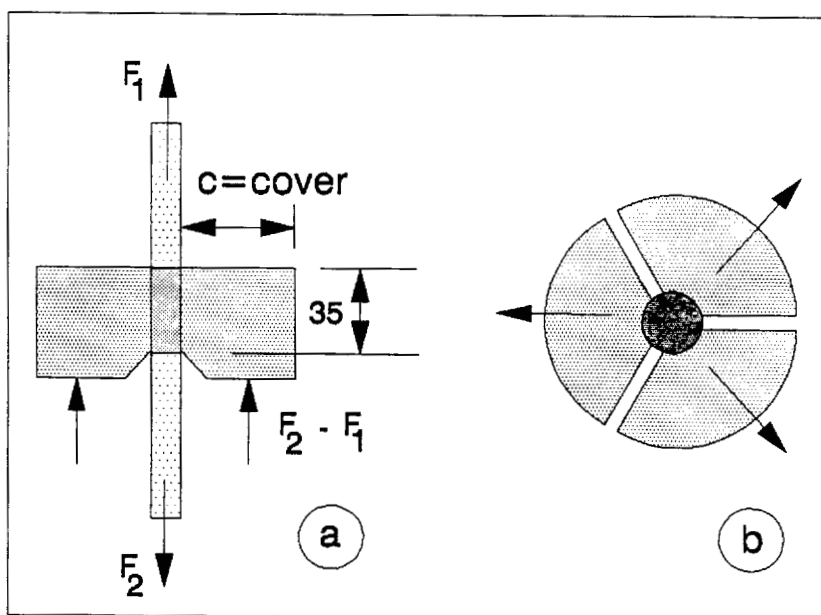


Fig. 3—Push-in and pull-out tests

12 de Sitter and Vonk

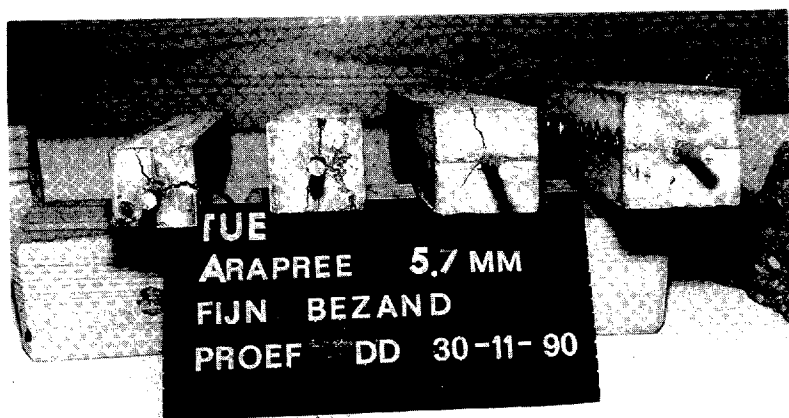


Figure 4

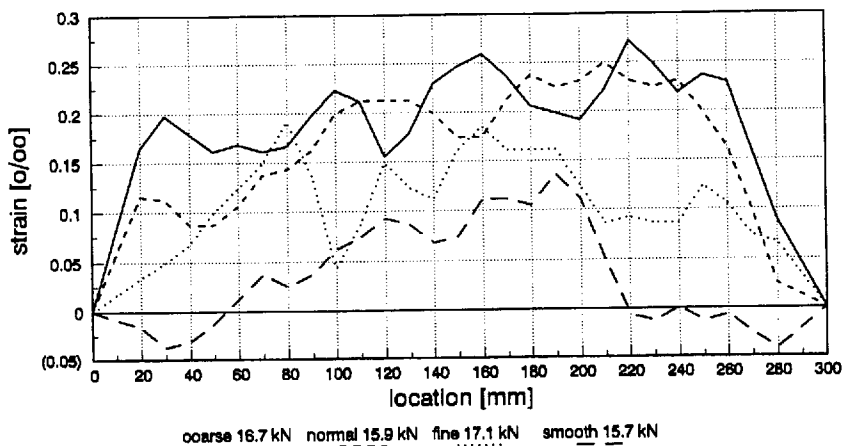


Fig. 5—Compressive strain in 45 x 45 mm² 300 mm prisms
17 kN nominal pretension

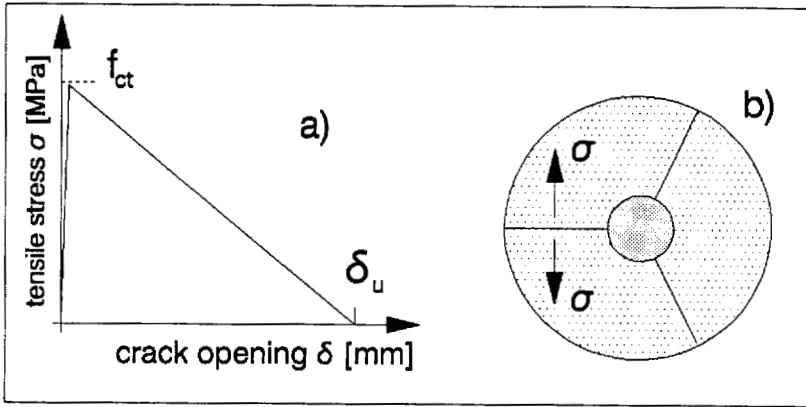
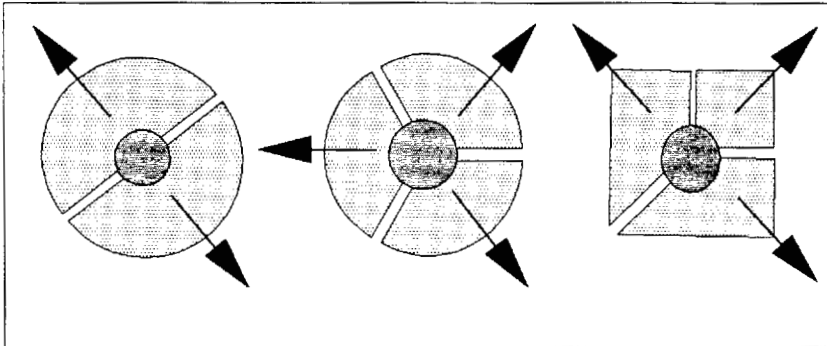


Figure 6

Fig. 7—3 cracks $\approx$ minimum degrees of freedom

14 de Sitter and Vonk

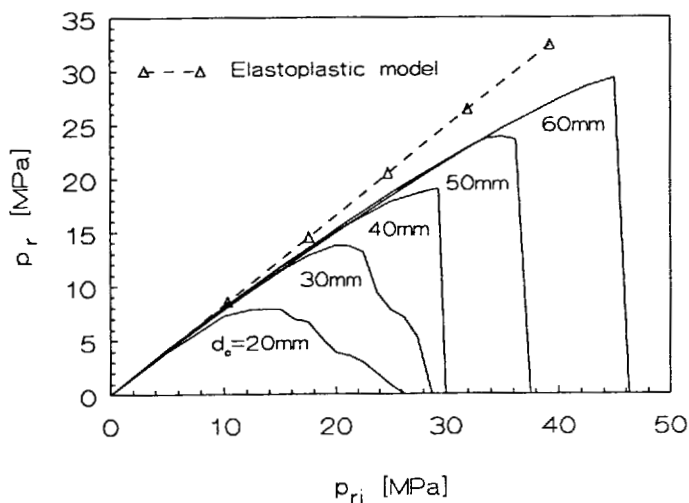


Fig. 8—UDEC calculations

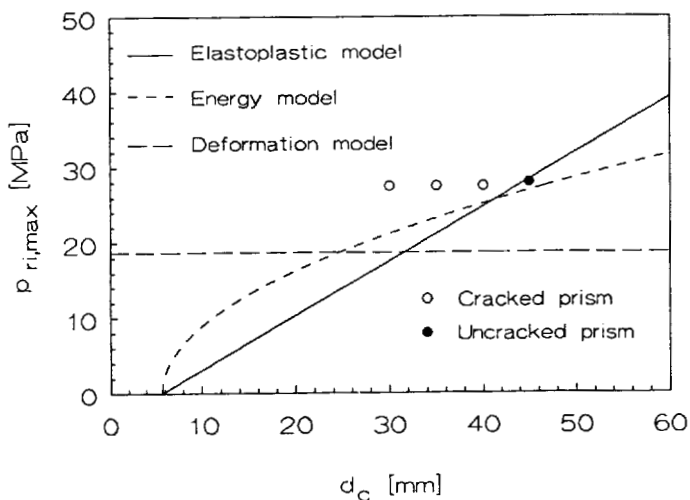


Fig. 9—Model predictions and test results

SP 138-2

Investigation of S-2 Glass/Epoxy Strands in Concrete

by R. Sen, D. Mariscal, and M. Shahawy

Synopsis:

A comprehensive durability study of S-2 glass/epoxy pretensioned beams exposed to wet/dry cycles in 15% salt solution indicated a complete loss of its effectiveness within 3-9 months of exposure. This paper presents results of subsequent follow-up investigations to identify the cause of this deterioration and also to examine practical measures that could be used to prevent its occurrence.

The analysis of the test results suggests that the most likely cause of failure was diffusion of hydroxyl ions from the concrete pore solution through the Shell Epon 9310 resin. This is supported by SEM micrographs of the failed beam.

While these conclusions are only valid for the Shell Epon 9310 resin examined, diffusion is also likely to be a characteristic of other types of resins, e.g. vinylesters or polyesters. This makes long term protection of glass fibers in concrete problematic.

Keywords: Diffusion; durability; epoxy resins; fiber reinforced plastics; glass fibers; pretensioning; rupture; salt water; scanning electron microscope; stresses

16 Sen, Mariscal, and Shahawy

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INTRODUCTION

In recent years, fiberglass composites have been increasingly considered for pretensioning, as reinforcement, as two and three dimensional grids and for strengthening concrete beams (1-4). These applications all share one important characteristic, namely, the fiberglass composite is *directly in contact with concrete*. Because glass fibers are very vulnerable in an alkaline concrete environment (5), an implicit assumption is that the resin system used can provide a protective barrier that assures the integrity of the fiberglass composite. The validity of this hypothesis had not been rigorously investigated until quite recently.

In 1989, the Florida Department of Transportation (FDOT), in cooperation with the US Department of Transportation, sponsored a 26 month study on the feasibility of using fiberglass pretensioned piles in a marine environment (6). As part of this study, tests were undertaken to determine the durability of identical pre-cracked and uncracked S-2 glass/epoxy pretensioned beams subjected to simulated tidal cycles in 15% salt solution (see Figure 1).

In the tests, durability was indirectly assessed from changes in ultimate capacity of the S-2 glass/epoxy pretensioned beams. This indicated a rapid and substantial loss in load carrying capacity with

exposure. For the pre-cracked beams, this reduction occurred after only 3-9 months of exposure (see Figure 2). It took somewhat longer, between 12-18 months, for the uncracked beams (see Figure 3). No commensurate reduction was observed in the unexposed control beams tested over the same 20 month period. Complete details are presented elsewhere (7).

Inspection of Figures 2-3 shows that the recorded failure loads in the exposed S-2 glass/epoxy pretensioned beams approached that for *unreinforced* beams. Since no slip was recorded, this meant that prolonged exposure had led to a *complete destruction of the S-2 glass*.

This paper describes investigations undertaken to determine why the epoxy was unable to protect the S-2 glass fibers and what measures are needed if this problem is to be solved. For simplicity, all subsequent references to 'fiberglass' imply the 3/8 in. (9.5 mm) S-2 glass/epoxy strands used in the testing.

OBJECTIVES OF STUDY

The primary objectives of this investigation were as follows:

1. To identify the most probable cause of deterioration of the fiberglass strands.
2. To corroborate the results of ultimate load tests with scanning electron microscope examination of fiberglass samples.
3. To identify measures that can enhance the durability of glass fiber composites in a concrete environment.

RESEARCH SIGNIFICANCE

Knowledge of the mechanism by which S-2 glass fibers were destroyed is essential for devising appropriate protective measures. Such measures can greatly extend the scope of application of glass fiber composites in concrete. The investigations described may also be adapted to evaluate the durability of other fiber reinforced plastics in alternative applications.

18 Sen, Mariscal, and Shahawy

CAUSE OF FAILURE

Damage to the fiberglass strands could be due to stress corrosion or glass/alkali reaction. The most likely cause of failure may be determined by a careful review of experimental data from the durability tests relating to stress levels in the fiberglass strands and the alkalinity of the concrete used.

Materials

Fiberglass -- The S-2 glass used in the durability tests is a lower cost version of the military grade S glass fiber. It is a magnesia-alumina-silicate glass with mechanical properties superior to E glass, used in demonstration structures constructed in Europe (8). Its chemical composition (9) and mechanical properties (10) are summarized in Tables 1-2.

An epoxy resin, Shell Epon 9310, was selected for the pultrusion of the S-2 glass rods since the amine cured resins are among the best systems for resisting the effects of an alkaline and saline environment. The properties of the Shell Epon 9310 epoxy resin (11) are listed in Table 3.

Seven 0.118 in. (3 mm) diameter S-2 glass/epoxy rods were held together by plastic ties to make up the 3/8 in. (9.5 mm) strands (effective area = 0.0702 in^2 (45.3 mm^2)), used for pretensioning the specimens (12). Two such strands were used in each test specimen.

Concrete -- The concrete used for the durability specimens was an FDOT approved mix using Type III rapid hardening cement, requiring a water/cement ratio of 0.41. Typically, the sodium oxide equivalent of this cement is below 0.6% (see Table 4) making it a *low alkali* cement according to ASTM C 150 classification. In liquid form, its pH value was estimated to be 12.5 (13). This pH is in the 12.5-13.5 range for pore fluid in normal concrete and represents a caustic or highly alkaline solution (14).

Stress Corrosion

Stress corrosion refers to the characteristic property of fiber reinforced plastics in which the failure strength under long term sustained loads in a chemical environment is lower than its short term tensile strength. In air, this phenomenon is referred to as 'stress rupture'.

The ultimate tensile strength, f_{gu} , and elastic modulus, E_{fg} , of the fiberglass strands used in the durability study were 273.5 ksi (1.89 GPa) and 9.16×10^6 psi (63.1 GPa) respectively (12). The target jacking force used for pretensioning each strand was 10,000 lbs (44.5 kN) or $0.52f_{gu}$. Prior to release, the average force in the fiberglass strands was measured to be approximately 8,000 lbs (35.6 kN) or $0.41f_{gu}$.

The effective prestress at the end of the tests can be estimated as $0.38f_{gu}$ assuming losses were 7.1% of the stress before release, i.e. the same as that measured in a parallel creep study using identical fiberglass strands jacked to the same level in specimens having the same dimensions (15).

The results of stress rupture tests on S glass/epoxy composites conducted over a 10 year period at the Lawrence Livermore Laboratory, CA (16) showed that only 2% of the test specimens failed at sustained stresses of $0.4f_{gu}$ while *none* failed at stresses of $0.35f_{gu}$. Since S glass and S-2 glass have comparable properties, the stress rupture characteristics of the fiberglass strands used in the durability study will also be similar.

While stress corrosion characteristics will be worse than stress rupture ones, the combination of relatively low stress levels in the fiberglass strands coupled with its deterioration within 3-9 months (see Figure 2) suggest that stress corrosion could not have been the principal cause of failure. Further evidence to support this is provided by the results of scanning electron micrographs presented later. This shows the deterioration to be as pronounced in the submerged region (point #1 in Figure 2) located 6 in. (150 mm) from the end as that at mid-span (point #5) even though its sustained stress level was about 40% lower (the transfer length was measured to be about 10 in).

Glass/Alkali Reaction

Concrete has microscopic voids whose size and distribution depend on numerous factors such as water/cement ratio, mode of vibration, or curing (14). Because of the alkalinity of the concrete used, capillary water present in these voids from the wet/dry cycles used in the durability tests would also be highly alkaline.

Silica, the largest constituent of S-2 glass (see Table 1), and alkalis react chemically (17). Hydroxyl ions from the alkaline pore solution can dissolve the basic silicon-oxygen-silicon structural network of S-2 glass as:



20 Sen, Mariscal, and Shahawy

This dissolution of silica is accompanied by a rapid and severe strength loss.

If concrete's pore solution could penetrate the epoxy resin protecting the S-2 glass fibers, the ensuing silica/glass reaction would lead to a dramatic loss in strength over a relatively short period of time as observed in the testing.

Silica/alkali reaction can also explain why the lesser stressed S-2 glass fibers in the submerged end suffered damage. Thus, the most likely cause of premature failure was a breakdown in the epoxy barrier protecting the S-2 glass fibers in concrete.

BREAKDOWN OF EPOXY BARRIER

Ingress of hydroxyl ions from the concrete pore solution through the epoxy resin could be attributed to the following:

1. Damage to the fiberglass strand surface during fabrication of specimens or cracking of the epoxy resin during pretensioning.
2. Diffusion through the epoxy resin.

Resin Damage

If the fiberglass strands were damaged during fabrication, such damage could be expected to occur in a random fashion and not be confined to the critical mid-span section in *all* the specimens. Furthermore, a 'damage theory' cannot explain why the pre-cracked beams failed earlier than the uncracked ones. At best, surface damage could have been a contributory factor.

The strain in the epoxy, corresponding to the maximum jacking load of 10,000 lbs. (44.5 kN) was 1.56% [$10,000/(0.0702)/9.16 \times 10^6$]. This is well below the 4% strain limit for Shell Epon 9310 quoted by its producers (see Table 3). While localized cracking is a possibility, its likely random distribution cannot explain the preferential failure of the pre-cracked specimens.

Diffusion

Since neither damage nor cracking can reconcile the experimental data, the most probable cause of ingress of hydroxyl ions is due to diffusion through the epoxy resin.

Inspection of the properties of the Shell Epon Resin in Table 3 shows that it had a moisture absorption rate of 1.5% of its weight after 14 days immersion in 200°F water. At the 70°F temperature of the laboratory (18), it could be expected to have a smaller, though significant, diffusion rate.

Diffusion can explain why the pre-cracked specimens failed earlier compared to the uncracked specimens shown in Figures 2 and 3. The mid-span location of the cracks, provided a direct passageway for the alkaline pore solution to reach the fiberglass strands at the critical section. It can also explain why the fiberglass strands in the submerged part of the beams experienced more damage - the diffusion was greater here because the capillary voids were never empty.

SCANNING ELECTRON MICROSCOPE EXAMINATION

To corroborate the results of the ultimate load tests and to determine the extent of degradation of glass fibers, scanning electron microscope (SEM) examinations were carried out. In view of limitations of cost, only one fiberglass beam was examined. This uncracked beam, FG-W2, had displayed a classic corrosion failure mode after 18 months exposure to wet/dry cycles.

Representative samples were taken from FG-W2 at locations corresponding to the permanently wet, 'splash zone' and permanently dry zones, i.e. points #1, #5 and #9 in Figures 2-3. To obtain a measure of the damage, if any, a stressed fiberglass strand exposed only to laboratory conditions was used as control. This was one of two strands that were tensioned during the fabrication of the durability specimens on December 28, 1989, but not used subsequently because of failure of the other strand (6).

All samples were professionally prepared for the SEM examination to a half micron final polish (see Figure 4). They were subsequently sputter coated with gold paladium and stored in a dust free vacuum chamber prior to their examination at the Surface Analysis Laboratory, University of South Florida, Tampa. Since only a limited number of

22 Sen, Mariscal, and Shahawy

samples were examined, the findings presented are necessarily *preliminary*. Nevertheless, they provide valuable information on the relative state of identically prepared samples taken from exposed and un-exposed segments of failed beams.

Results

Permanently Dry Location -- Figure 5 compares micrographs for the control (unexposed) specimen with that for the sample taken from the dry end of the beam (point #9). Inspection of this micrograph shows that the relative state of the glass fibers is comparable with little evidence of degradation of the fibers. This corroborates the test results where no significant reduction in ultimate strength was observed for the control beams (see Figures 2 and 3).

Splash Zone -- Figure 6 compares two micrographs for a sample at the splash zone (point #5). Inspection of this micrograph shows that there was much greater deterioration and degradation of the glass fibers located nearer the concrete interface. By contrast, there is less dissolution of the fibers though more cracking at fibers located farther away.

Permanently Wet Location -- Figure 7 presents two micrographs for the sample that was permanently submerged (point # 1). As for the sample taken from the splash zone, there is considerable evidence of deterioration and degradation particularly in those fibers located close to the concrete interface. The extent of degradation for this sample appears to be greater in comparison to the sample at the splash zone (Figure 6).

Discussion of Results

Overall, the micrographs corroborate the results of the ultimate load tests. There is little damage at the dry end (Figure 5) but much greater damage in the submerged (Figure 7) and splash zone locations (Figure 6). The extent of the degradation is greater near the concrete interface than near the middle of the fiber. Such observations are consistent with the conclusion reached earlier that diffusion by the epoxy resin was largely responsible for the destruction of the fiberglass. The damage to the S-2 glass fibers observed is commensurate with silica's dissolution in alkalis.

IMPROVING DURABILITY OF FIBERGLASS IN CONCRETE

Since diffusion of the alkaline pore solution by the epoxy resin was identified as the primary cause of failure of the fiberglass, it might be argued that the use of an impermeable epoxy would be the ideal solution for improving durability. From a practical standpoint, however, such a resin is likely to be prohibitively expensive and may be impossible to use in a pultrusion process. More importantly, the experience of epoxy coating of reinforcement bars suggests that reliance on a single method of protection is unwise in the long run.

In view of this, a comprehensive protective system must be devised. At its core, the alkali resistance of the glass should be increased while simultaneously reducing alkalinity and porosity of the concrete. (The simplest and possibly the most trouble-free solution would be use of non-alkaline cement. Unfortunately, it may not be possible to produce it economically).

Whereas S-2 glass fibers were used in this study because of their superior mechanical properties, in future applications greater emphasis should be placed on durability. Careful consideration should be given to the use of alkali resistant (AR) glass (19), which is more durable in concrete. The resin system selected should be the least permeable, commensurate with costs. The possibility of using additional protection such as circumferential wrapping with a polymeric yarn (20) should be considered. This will help prevent surface damage and also provide an additional barrier against alkali attack.

The permeability of the concrete should be reduced to increase durability. This may be achieved by using condensed silica fume whose fine particle size allows voids in the cement paste or between the aggregates and cement paste to be filled. The cement with the least alkalinity should be used in construction. As noted earlier, the cement used in this study qualifies as a low alkali cement but its pH of 12.5 proved too high. In addition, procedures successfully used for improving durability of glass fiber reinforced concrete (21), should be carefully investigated since they could well apply for pultruded specimens.

Whatever system of protection is adopted must be carefully evaluated using both accelerated and real time tests. Only when tests provide complete assurance of durability can glass fiber composites be used in applications requiring direct contact with concrete.

24 Sen, Mariscal, and Shahawy

CONCLUSIONS

The following conclusions can be drawn:

1. The primary cause of deterioration of S-2 glass/epoxy strands was diffusion of alkaline pore solution through the Shell Epon 9310 resin (see Table 3). The low stress level (estimated to be $0.38f_{gu}$ at mid-span and $0.23f_{gu}$ at point #1) and the relatively short time needed for deterioration (3-9 months) make stress corrosion improbable. Other possible causes such as surface damage or resin cracking are also implausible since they cannot explain a very important result from the durability testing, namely, that the pre-cracked beams failed much earlier than the uncracked ones. Thus, their contribution, to the destruction of fiberglass, if any, was minor.
2. The results of the scanning electron microscope investigation corroborate the findings from the durability testing. There was little damage to glass fibers in the dry segment (see Figure 5) but much greater damage in the wet or wet/dry segments (Figure 6 and 7). Damage was also greater nearer the concrete interface than away from it.
3. It is highly improbable that the Shell Epon 9310 resin used in the study is the only resin that absorbs moisture by diffusion. Less expensive polyesters and vinylesters can be expected to show comparable diffusion characteristics. In view of this, the durability of *all* glass fiber composites in concrete must be carefully investigated. Several measures are proposed that may eventually lead to a satisfactory solution. Until such time, however, direct contact of glass fiber composites with wet concrete should be avoided.

ACKNOWLEDGEMENTS

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26 Sen, Mariscal, and Shahawy

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NOTATION

H.T.	High tide
L.T.	Low tide
E_{fg}	Elastic modulus of S-2 glass/epoxy strand
f_{fgu}	Ultimate tensile strength of S-2 glass/epoxy strand

Table 1 Approximate Chemical Composition of S-2 and E Glass Fibers (9)

Component (wt%)	S-2 Glass	E Glass
SiO ₂	69	55
CaO	--	19
Al ₂ O ₃	21	14
B ₂ O ₃	--	6
MgO	2.7	0.5
Na ₂ O, K ₂ O	--	1.2
Other ¹	7.3	4.3

¹ Fe₂O₃, Zr₂O₃, TiO₂, ZnO

Table 2 Mechanical Properties for S-2 and E Glass Fibers (10)

Property	ASTM Standard	E Glass	S-2 Glass
Tensile Strength , ksi at 72° F (Single Filament) 1000° F	D2101	500-550 250	665-700 350
Tensile Modulus , ×10 ⁶ psi at 72° F 1000° F	D2343	10-10.5 11.8	12.5-13 12.9
Failure Strain , %	D2343	4.5-4.9	5.4-5.8

1 psi = 6.895 kPa

1 ksi = 6.895 MPa

28 Sen, Mariscal, and Shahawy

Table 3 Properties of Shell Epon® Resin 9310 System (11)

Property	System A ¹	System B
Density, gr/cc	1.18	1.20
Moisture Absorption ² , % wt, 14 days	1.5	--
Tensile Strength, ksi at		
73°F	11	11
300°F	--	3.8
Tensile Modulus, ksi at		
73°F	450	453
300°F	--	205
Failure Strain, % at		
73°F	4	4.0
300°F	--	5.2

1 psi = 6.895 kPa

1 ksi = 6.895 MPa

¹ Curing schedules (System A similar to that used for fiberglass rods)² Sample immersed in 200°F water**Table 4** Chemical Composition of Cement (by weight) (13)

Component	wt %
SiO ₂	20.42
CaO	63.35
Al ₂ O ₃	5.15
Fe ₂ O ₃	4.2
MgO	0.74
SO ₃	3.36
Na ₂ O Equiv.	0.51
Others ¹	2.03

¹ Others includes insoluble resins and L.O.I.

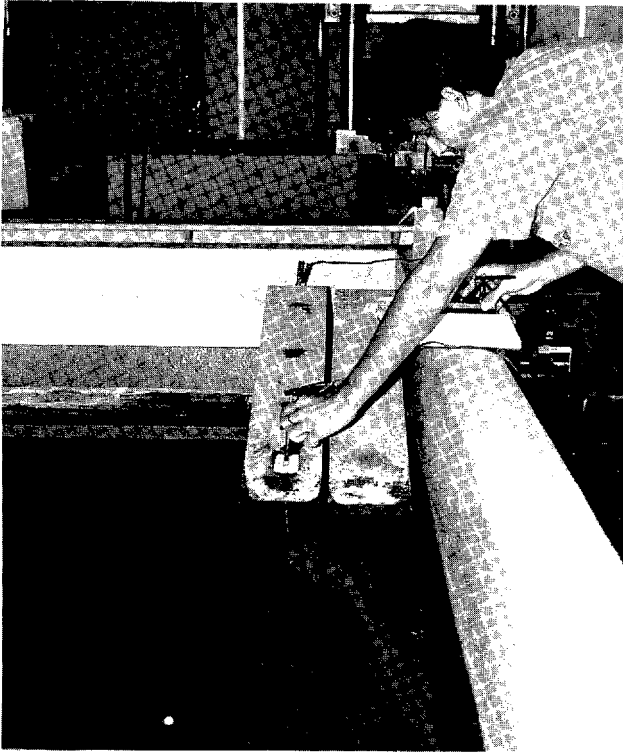


Fig. 1—Tidal simulation set up

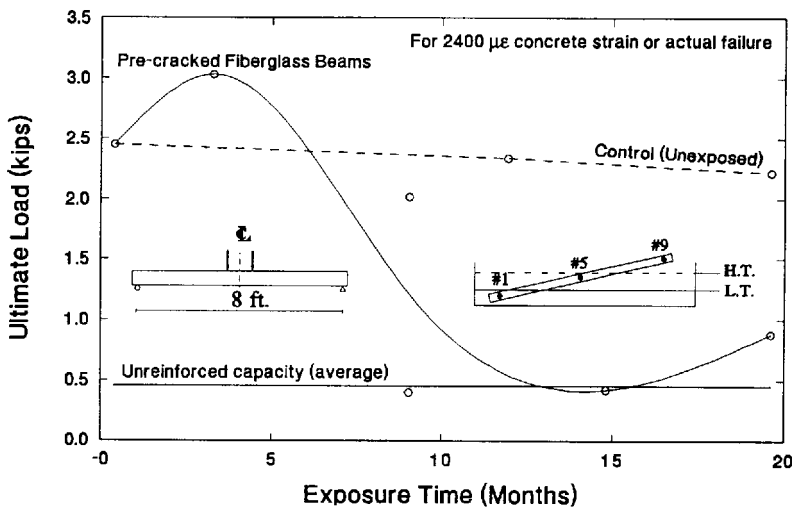


Fig. 2—Ultimate strength change with exposure for precracked fiberglass beams

30 Sen, Mariscal, and Shahawy

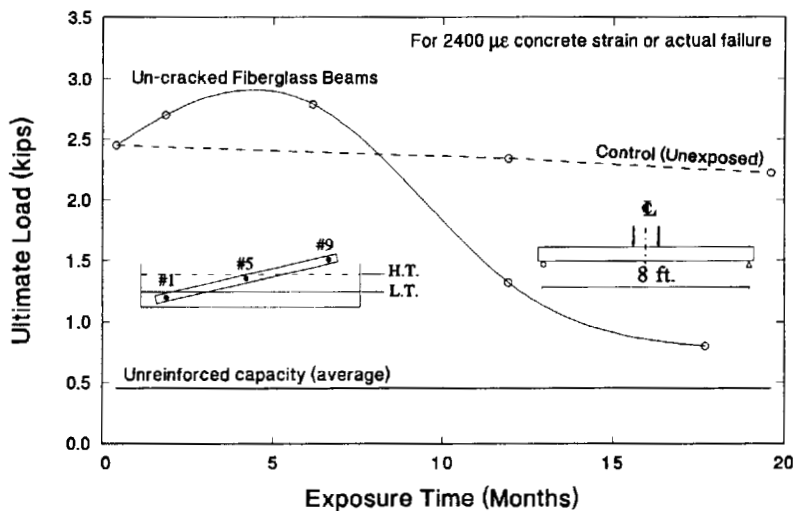


Fig. 3—Ultimate strength change with exposure for uncracked fiberglass beams



Fig. 4—SEM specimens – control, #1, #5, #9

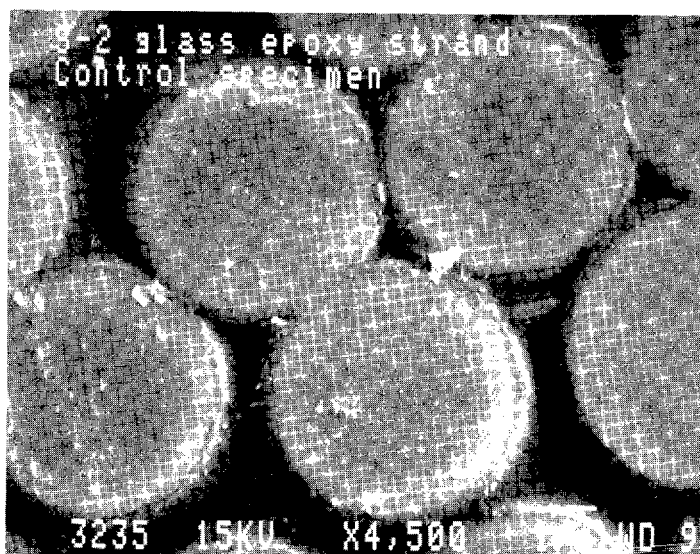


Fig. 5—Unexposed control specimen

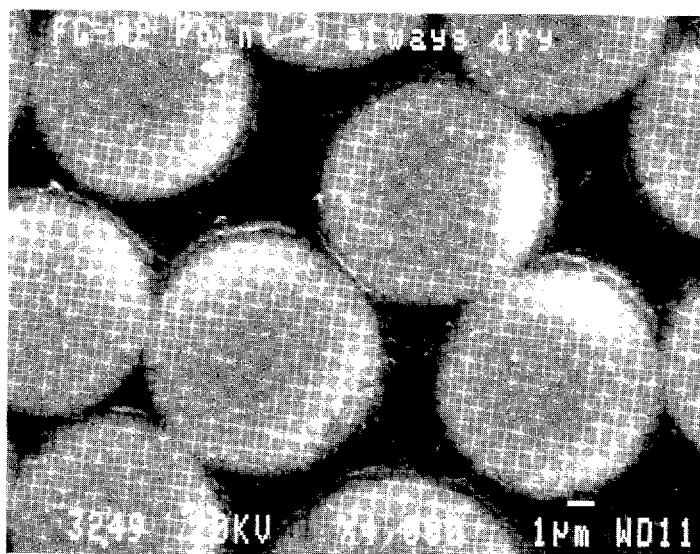


Fig. 5—Always dry specimen

32 Sen, Mariscal, and Shahawy

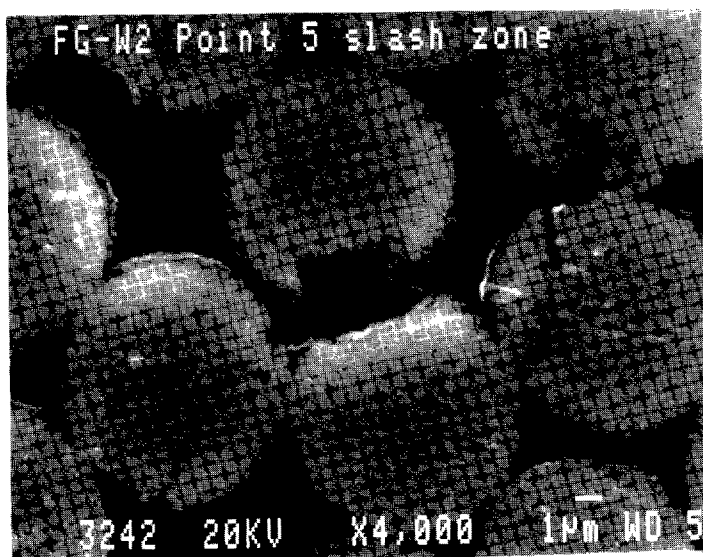


Fig. 6—Splash zone specimen away from concrete interface

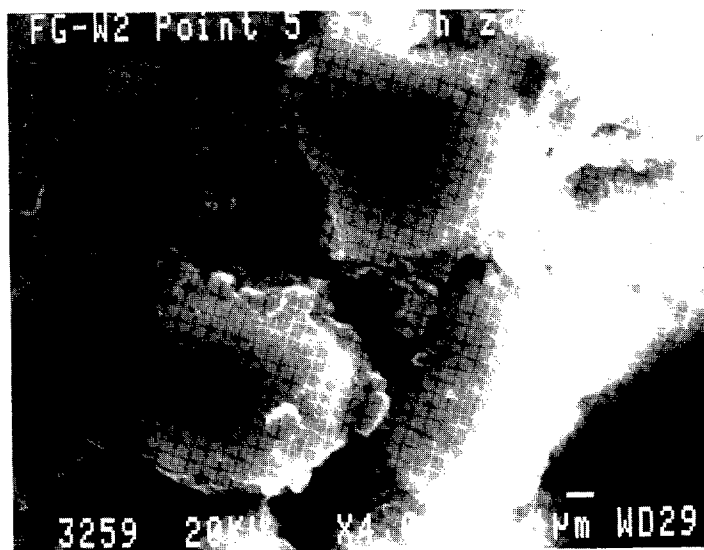


Fig. 6—Splash zone specimen near concrete interface

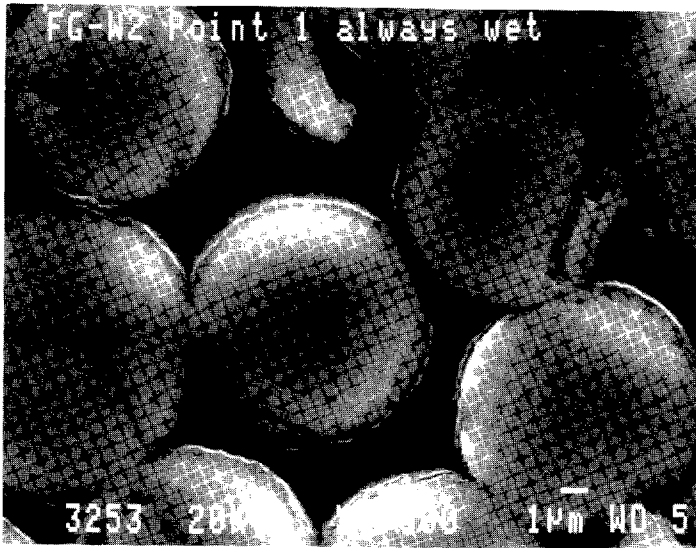


Fig. 7—Submerged specimen away from concrete interface

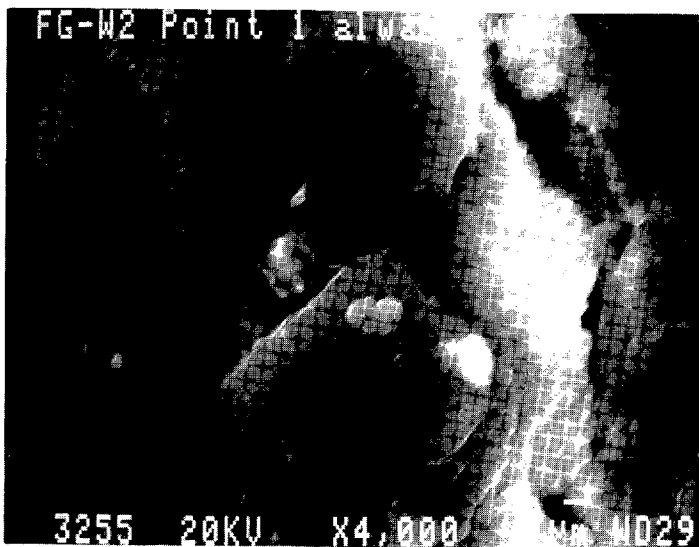


Fig. 7—Submerged specimen near concrete interface

Characteristics of Aramid FRP Rods

by K. Mukae, S. Kumagai, H. Nakai,
and H. Asai

Synopsis: As part of their research into the use of aramid (brand name:Technora®) FRP rods as tensioning material for prestressed concrete or as concrete reinforcement the authors conducted tension test, creep test and fatigue test on the material. In the assessment of the strength characteristics of the material, the anchoring method has a serious influence on the evaluation. In these tests,we appraise the aramid FRP rods with a bond type anchoring method which has a high performance.

The tests showed that the tensile strength of Technora® aramid FRP rods has a normal distribution, with values equal to or above those obtained for steel reinforcement bars. The creep failure curve showed a logarithmic relationship with the time axis; the experimental equation is show here. From the results of a fatigue test with 2 million loading cycles, it was ascertained that the fatigue strength of the material is adequate for practical use.

Keywords: Anchorage (structural); bond (concrete to reinforcement); creep properties; fatigue tests; fiber reinforced plastics; prestressed concrete; tensile strength; tests

36 Mukae et al

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1. INTRODUCTION

Research is in progress in the construction industry into the use of FRP (fiber reinforced plastics) rods with high tensile strength as tendons for prestressed concrete or as reinforcement material for concrete. In comparison to steel FRP rods are highly resistant to corrosion in a saline environment. For this reason, they are expected to prove very useful in harsh environments such as marine environments.

For FRP rods to be used for tendons in prestressed concrete or for ground anchors, etc., a suitable anchoring method for FRP rods has to be developed. The authors have developed a bond type of anchoring for aramid FRP (AFRP) tendons, and have applied this anchoring with AFRP tendons to actual structures.[1,2]

The present paper describes tension test, creep test and fatigue test carried out on AFRP tendons with a bond type anchoring, and applies the results of these tests to a consideration of tensile stress standards for AFRP tendons.

2. CHARACTERISTICS OF AFRP TENDONS

The AFRP rods used in the present tests were developed in jointly with Teijin Co.,Ltd. The rods consisted of bundles of aramid fibers (brand name:Technora®), which were impregnated with vinyl ester resin and hardened after being formed into rods by the pultrusion method. As shown in Fig.1, the rods consisted of a $\phi 6\text{mm}$ base, around which were wound three types of aramid fiber. First, a winding fiber was wound around the rod to deform its surface, and on top of this were wound a longitudinal securing fiber and a round securing fiber, which were fixed with resin. With these three types of winding fiber, the AFRP rods are assured of a good bond in concrete or mortar.

The rods were anchored using a bond type anchoring. This bond type anchoring device consisted of a single AFRP rod inserted into a 16mm inside diameter steel tube, and fixed by the injection of shrinkage compensating mortar. Water/cement and admixture ratio in the shrinkage compensating mortar was 31%, and the C:S (cement+admixture : fine aggregate) ratio was 1:1. Bolts were added to the outer surface of the steel tube, and these were secured with nuts.

Various anchoring methods for FRP tendons have been discussed by researchers, but a completely practical anchoring method has not yet been found.[3,4] Even with the bond type anchoring method used in the tests discussed here, it was not possible to attain 100% of the theoretical strength of the rods. Therefore, the test results given here are the results of tests of AFRP tendons including their anchoring.

3. TENSION TEST

3.1 Test Method

The device used for tension test is shown in Fig.2. The test sample consisted of a single $\phi 6\text{mm}$ diameter rod of total length 3.8m ,test length 3.0m. A hydraulic jack was used for the loading device. The loading speed was 196 MPa/min., and the sample was brought to breaking point by monotonous loading. The tensile load was measured using a load cell, and the deformation of the tendon was calculated from the values observed using electrical displacement gauges ① & ② in the diagram. These displacement gauges were removed when the tension of the sample had reached around 1400MPa. The test was conducted at room temperature.