Application of Fiber Reinforced Plastic Rods as Prestressing Tendons in Concrete Structures

WA-RD 186.1

Final Report
August 1989

Washington State Department of Transportation
Planning, Research and Public Transportation Division

in cooperation with the
United States Department of Transportation
Federal Highway Administration

Final Report

Research Project GC 8286, Task 25
Fiberglass Tendons

APPLICATION OF FIBER REINFORCED PLASTIC RODS AS PRESTRSSING TENDONS IN CONCRETE STRUCTURES

by

Alan Mattock
Professor of Civil Engineering

Khossrow Babaei
Senior Research Engineer

Washington State Transportation Center (TRAC)
University of Washington, JE-10
The Corbet Building, Suite 204
4507 University Way N.E.
Seattle, Washington 98105

Washington State Department of Transportation
Technical Monitor
C.S. Gloyd
WSDOT Bridge and Structures Engineer

Prepared for

Washington State Transportation Commission
Department of Transportation
and in cooperation with
U.S. Department of Transportation
Federal Highway Administration

August 1989

WASHINGTON STATE DEPARTMENT OF TRANSPORTATION

TECHNICAL REPORT STANDARD TITLE PAGE

1. REPORT NO.	2. GOVERNMENT ACCESSION NO.	3. RECIPIENT'S CATALOG NO.
WA-RD 186.1		

4. TITLE AND SUBTITLE
APPLICATION OF FIBER REINFORCED PLASTIC RODS AS PRESTRESSING TENDONS IN CONCRETE STRUCTURES

5. REPORT DATE
August 1989

6. PERFORMING ORGANIZATION CODE

7. AUTHOR(S)
Alan H. Mattock, University of Washington
Khossrow Babaei, Washington State Transportation Center

8. PERFORMING ORGANIZATION REPORT NO.

9. PERFORMING ORGANIZATION NAME AND ADDRESS
University of Washington, Department of Civil Engineering, and
Washington State Transportation Center (TRAC)
The Corbet Building, Suite 204; 4507 University Way N.E.
Seattle, Washington 98105

10. WORK UNIT NO.

11. CONTRACT OR GRANT NO.
GC8286, Task 25

12. SPONSORING AGENCY NAME AND ADDRESS
Washington State Department of Transportation
Transportation Building, KF-10
Olympia, Washington 98504

13. TYPE OF REPORT AND PERIOD COVERED
Final report

14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES
This study was conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration.

16. ABSTRACT

This study is concerned with the possibility of utilizing fiber reinforced plastic rods as prestressing tendons, in place of traditional steel tendons, in elements of prestressed concrete bridges exposed to corrosive environments.

A survey was made of available information on the behavior characteristics of fiber reinforced plastic tension elements, and in particular those of glass fiber reinforced (GFR) tension elements. Also, an analytical study was made of the flexural behavior of concrete elements prestressed by GFR tendons.

Based on the analytical study and on the survey of available information, an assessment is made of the impact on the design of prestressed concrete members if GFR tendons are used. Some preliminary design recommendations are made, together with proposals for research needed before GFR prestressing tendons should be used in practice.

Four GFR tendons with Con-Tech Systems anchorages were tested, the primary variable being the embedded length of the GFR rods in the anchorages. All the tendons failed by the rods pulling out of the anchorages. For embedded lengths of 385 mm (15.2 in.) or greater, the failure loads were about 90 percent of the advertised tendon strength of 220 ksi, or about 100 percent of the guaranteed tensile strength of 197 ksi (60 kN/rod).

17. KEY WORDS
Prestressed concrete, tendons, glass fiber reinforced tendons, corrosion, anchorage

18. DISTRIBUTION STATEMENT
No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22616

19. SECURITY CLASSIF. (of this report)	20. SECURITY CLASSIF. (of this page)	21. NO. OF PAGES	22. PRICE
None	None	55	

DOT FORM 310-022 (11/86) -1216-

DISCLAIMER

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

SUMMARY

This study is concerned with the possibility of utilizing fiber reinforced plastic rods as prestressing tendons, in place of traditional steel tendons, in elements of prestressed concrete bridges exposed to corrosive environments.

A survey was made of available information on the behavior of fiber reinforced plastic tension elements, and, in particular, of glass fiber reinforced (GFR) tension elements. Also, an analytical study was made of the flexural behavior of concrete elements prestressed by GFR tendons.

On the basis of analytical study and the survey of available information, an assessment was made of the impact of using GFR tendons on the design of prestressed concrete members. Some preliminary design recommendations were made, together with proposals for research needed before GFR prestressing tendons should be used in practice.

Four GFR tendons with Con-Tech Systems anchorages were tested, the primary variable being the embedded length of the GFR rods in the anchorages. All the tendons failed when the rods pulled out of the anchorages. For embedded lengths of 385 mm (15.2 in.) or greater, the failure loads were about 90 percent of the advertised tendon strength of 220 ksi, or about 100 percent of the guaranteed tensile strength of 197 ksi (60 kN/rod).

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and preliminary recommendations are based on a survey of the literature, an analytical study of the flexural behavior of concrete members prestressed with glass fiber reinforced (GFR) rods and with strand, and the physical testing of four GFR tendons consisting of Polystal rods with Con-Tech Systems anchorages.

1. GFR elements, such as the Polystal type P rods produced by Bayer AG (1), have potential for use in prestressed concrete bridge elements subject to aggressive environments. In addition to their resistance to conditions which cause corrosion of steel tendons, their use would lead to smaller prestress losses caused by concrete shortening, because of their low modulus of elasticity (about one-quarter that of steel strand).
2. It is not possible at this time to propose final recommendations for the design of prestressed concrete elements utilizing GFR tendons, as there are outstanding questions regarding their behavior.
3. The following preliminary recommendations are based on present knowledge.
    a. The maximum value of the effective prestress in GFR tendons, after losses have occurred from creep, shrinkage, and elastic shortening of the concrete, should not be more than 0.50 f_{pu} (f_{pu} = tendon ultimate strength).
    b. The maximum value of the initial prestress in GFR tendons should be 0.70 f_{pu}.
    c. Members prestressed by GFR tendons should be designed so that the maximum concrete flexural tensile stress under service load is

zero, or so that a specific factor of safety is provided against flexural cracking under service load.

d. Currently used approximate equations for the stress in prestressing tendons at flexural ultimate should not be applied to GFR tendons, since the equations are empirically based on the observed behavior of members prestressed with steel tendons.

e. In post-tensioned members with unbonded GFR tendons, careful consideration should be given to the value used for tendon ultimate strength (f_{pu}), since GFR tendon strength decreases as the uniformly stressed length increases. The tendon ultimate strength (f_{pu}) could be calculated using the following equation:

$$f_{pu}/(f_{pu})_{12} = [104.8 - 5.212\log_{10}L_f]/100$$

where f_{pu} is the strength of the tendon if the uniformly stressed length is L_f inches, and $(f_{pu})_{12}$ is the tendon strength measured on a specimen of length 12 in. between grips.

f. Care must be taken that the radii of saddles used to deflect GFR tendons and of curved tendon ducts are sufficiently large to prevent excessive flexural or lateral bearing stresses occurring in the tendons. Currently a minimum radius of curvature of 25 ft is recommended.

g. Attention must be paid to space requirements for GFR tendons and their anchorages when member cross-sections are proportioned. These requirements are more demanding for GFR tendons than for steel tendons.

4. Before GFR tendons are used in prestressed concrete bridge structures, research should be carried out concerning the following topics:

a. the values of the strain compatibility factor (F) that should be used when calculating the flexural ultimate strength of bonded, post-tensioned members and of pretensioned, prestressed members (F = [change in strain in tendon]/[change in strain in adjacent concrete at section of maximum moment]),

b. the value of the "transfer length" and of the "development length" of coated GFR tensile elements such as Polystal rods when they are used in pretensioned, prestressed concrete members, and

c. the appropriate approximate expressions for the tendon stress at flexural ultimate when GFR tendons are used.

5. More detailed information should be obtained from Bayer AG concerning the tests carried out to determine the resistance of stressed Polystal rods to aggressive environments. The description of these tests in their literature is incomplete. (2)

6. The Con-Tech Systems anchorage for Polystal rods with an embedded length of 385 mm (15.2 in.) has the potential to satisfy the Post Tensioning Institute's requirements for post tensioning anchorages. (32) However, the actual efficiency of these anchorages, i.e., (strength of anchorage)/(actual tensile strength of tendon), is still in doubt, since the actual tensile strength of the Polystal rods used in these tests is not known. (On the basis of Con-Tech Systems' value of 60 kN (13.5 kips) for the guaranteed tensile strength of a 7.5-mm (0.295 in.) diameter Polystal rod, the efficiency of the Con-Tech Systems' anchorage with an embedded length of 385 mm (15.2 in.) was calculated to be 1.03. However, with the value of 1520 MPa (220 ksi) for tensile strength listed by Bayer and Strabag, the efficiency of this anchorage becomes 0.92, i.e., less than the value of 0.95 required by the Post Tensioning Institute. (2) The strength of the anchorage is a

function of the actual strength of the Polystal rods, since the anchorage failures were all actually inter-laminar shear failures of the rods. Con-Tech Systems should therefore be required to produce a single rod anchorage capable of developing the full tensile strength of a Polystal rod, so that the actual tensile strength of the Polystal rods, and hence the actual efficiency of their anchorage, can be determined.)

7. Con-Tech Systems should also be required to demonstrate that its anchorage behaves satisfactorily under cyclic load, as specified by the Post Tensioning Institute. (32)

INTRODUCTION

RESEARCH OBJECTIVES

The objectives of this research project were as follows:

1. to evaluate the feasibility of using fiber reinforced plastic rods as prestressing reinforcement in concrete bridges in corrosive environments,
2. to assess the impact on prestressed concrete member design philosophy of using this type of prestressing tendon,
3. to evaluate the effectiveness of the Con-Tech Systems post-tensioning anchorage by conducting static tension tests on four tendon assemblies, each consisting of eight 7.5-mm diameter glass fiber reinforced plastic rods anchored in Con-Tech anchorages at each end, and
4. on the basis of the information obtained, to develop design recommendations if possible, and to determine the need for further research.

THE PROBLEM

Traditional steel prestressing tendons are vulnerable to corrosion unless special precautions are taken. Bridge deck slabs in general, and complete bridges in a marine environment, are subject to corrosive conditions. Corrosion of reinforcement of any kind leads to the deterioration of concrete structures. The durability of concrete bridge decks, and of bridges in a marine environment, would therefore be improved if a non-corrosive, or highly corrosion resistant material could be used for prestressing tendons. This would result in reduced maintenance costs and an increased service life for the structure.

Composite rods consisting of high strength organic or inorganic fibers embedded in a resin matrix have been developed. These composites combine high strength with high corrosion resistance. They are potentially very suitable for use as prestressing tendons in corrosive environments. They also offer the additional advantage of reduced prestress losses caused by creep and shrinkage of concrete because of their low modulus of elasticity (from one-quarter to two-thirds that of steel).

Several studies of the possible use of composite rods for prestressing have been made over the past 20 years. Of particular interest is the Bayer company's development in the last decade of the composite rod "Polystal." (1) This rod consists of about 80 percent by weight (65 percent by volume) E-glass fibers embedded in an unsaturated polyester resin. Polystal rods have an ultimate strength of about 220 ksi and a modulus of elasticity of about 7,400 ksi. They behave in an elastic manner up to failure, which occurs at an elongation of about 3 percent. These rods have the potential for use as prestressing elements in both post-tensioned and pretensioned, prestressed concrete.

A joint venture by the Bayer company and Strabag Bau AG, a contracting firm of Cologne, Germany, has produced a post-tensioning system utilizing the Polystal bar. This system has been used in Germany for the construction of two bridges, with a third planned for construction in late 1988.

A Canadian company, Con-Tech Systems Ltd., has entered into an agreement with the Bayer company to use Polystal rods in a post-tensioning system that it is developing. This system is similar to that of Strabag.

The Washington State Department of Transportation (WSDOT) is interested in the possibility of using fiber reinforced rods as prestressing tendons in corrosive environments to improve bridge durability and reduce maintenance costs. Before this can be done, a more thorough evaluation of the physical properties of this type of reinforcement and of the effectiveness of the anchorages is necessary. Also needed is an assessment of the impact on bridge member design philosophy of the unique characteristics of this type of reinforcement.

COMPOSITE MATERIALS FOR PRESTRESSING

FIBER REINFORCED ELEMENTS

Fiber reinforced plastic elements that use fibers of glass, aramid (abbreviation for polyparaphenylene-terephtalamide), or carbon are being developed for use as concrete reinforcement. (1, 3, 4, 5, 6, 7) (Aramid is more widely known by the trade names Kevlar and Twaron.) Table 1 summarizes the principal properties of some reinforcing elements made from these fibers. (The data were taken from the previously referenced reports.)

At the present time, Polystal is the only fiber reinforced plastic element that has been developed to the point at which it can be used experimentally in an actual prestressed concrete structure. (2, 8) Studies directed toward the utilization of aramid fiber reinforced elements in prestressed concrete structures are in progress at Cornell University, U.S.A. (Kevlar), and Delft University, Holland (Twaron). Carbon fiber reinforced elements are being used experimentally in reinforced concrete members in Japan. (6)

Since WSDOT is interested in the experimental use of fiber reinforced tendons in prestressed concrete bridges in the near future, this report will focus on the possible use of glass fiber reinforced tendons and, in particular, on the possible use of Polystal rods.

GLASS FIBER REINFORCED (GFR) ELEMENTS

Short-Term Loading Characteristics

The short-term loading properties of a 7.5-mm (0.295 in.) diameter Polystal rod with a fiber content by volume of 70 percent, and of its constituent materials, are shown in Table 2, which is taken from reference (9). Note that the longitudinal tensile strength reported corresponds to a "free length" between anchorages of 300 mm (11.8 in.). Rehm and Schlottke have reported that the short-term tensile strength, f_{pu}, of Polystal rods decreases with increasing "free length," L_f. (10) On the basis of 169 tests of specimens

Table 1. Properties of Typical Fiber Reinforced Tensile Elements

Type of fiber	glass	aramid	carbon
Name of fiber	E-glass	Twaron HM	Carbon HS
Name of element	Polystal	Arapree	BRI-TEN HS
Producer of element	Bayer	Erika/HBG	Bridon
Matrix resin	polyester	epoxy	polyester, epoxy or vinylester
Shape of element	round	rectangular	round
Size (mm)	7.5 to 25	1.5 x 20.5	1.7 to 12
Strength (MPa/ksi)	1,520/220	2,800/406	2,400/350
Elastic Modulus (MPa/ksi)	51,000/ 7,400	140,000/ 20,300	150,000/ 21,750
Ultimate strain	0.030	0.020	0.017
Coefft. of Thermal Expansion (10^{-6}/°F)	3.9	-0.90	≈ 0
Density (g/cm^3/lb/ft^3)	2.0/125	1.25/78	1/58/99

Table 2 Short Term Loading Properties of Polystal GFR Elements and of its Constituent Materials (9).

Polystal		
Longitudinal tensile strength[1]	(MPa/ksi)	1,600/230
Transverse normal strength	(MPa/ksi)	140/20
Shear strength	(MPa/ksi)	45/6.5
Elastic modulus	(MPa/ksi)	52,000/7,540
Ultimate tensile strain		0.030
Poisson's ratio		0.28
Density	(g/cm^3/lb/ft^3)	2.1/130
Glass fiber		
Tensile strength	(MPa/ksi)	≅ 2,300/334
Elastic modulus	(MPa/ksi)	≅ 74,000/10,730
Ultimate tensile strain		0.030
Unsaturated polyester resin		
Tensile strength	(MPa/ksi)	75/11
Elastic modulus	(MPa/ksi)	300/44
Ultimate tensile strain		0.040

[1] Based on a specimen length of 300 mm (11.8 in.) between the grips.

with free lengths of from 50 mm (2 in.) to 5000 mm (197 in.), they propose that the relationship between tensile strength (f_{pu}) and free length (L_f) can be expressed as follows:

$$f_{pu}/(f_{pu})_{300}\ (\%) = 112.1 - 5.2122 \log_{10}(L_f)$$

where L_f is measured in millimeters and $(f_{pu})_{300}$ is the tensile strength measured on a specimen with a free length of 300 mm (11.8 in.). They report a coefficient of determination (r^2) of 0.90 for this equation.

The dependence of the tensile strength of Polystal rods on their free length reflects the fact that the tensile strength of a glass fiber-polyester resin composite is essentially proportional to the tensile strength of the glass fibers. This is because the strength of the fibers is many times that of the resin matrix, as may be seen in Table 2. The strength of these fibers shows a statistical variability and a dependence on length as a consequence of the random occurrence of flaws in the glass fibers. (11) The dependence of the tensile strength of glass fiber reinforced composites on the length of the specimen has also been shown by Lifshitz and Rotem and McKee and Sines. (12, 13)

As with other glass fiber reinforced composite elements, the stress-strain relationship for Polystal rods is linear up to failure. Failure does not occur at a single cross section in a glass fiber reinforced element. Failure of groups of fibers occurs at various locations along the length of the specimen. These local tensile failure locations are joined by longitudinal cracks, so that failure occurs over a major part of the specimen. (12) Failure is sudden and violent.

The longitudinal cracks are caused by inter-laminar shear between groups of glass fibers. Inter-laminar shear cracks are also seen when a glass fiber reinforced composite element is bent excessively. After a group of fibers fractures close to the maximum flexural tension face, an inter-laminar shear crack will propagate along the element from the tensile failure crack. The inter-laminar shear strength is small compared to the longitudinal tensile strength because it is a function of the strength of the relatively low strength resin matrix or of the bond between the matrix and the glass fibers. With a view to improving inter-laminar shear strength, coupling agents are used in the manufacture of the composite elements to improve the bond between the resin and the glass fibers.

Franke reports that for temperatures below about 300°C (570°F) the behavior of Polystal rods is better than that of conventional prestressing steel. (5) Rods stressed to half their room temperature tensile strength and subjected to a temperature of 300°C (570°F) for 30 minutes failed when the stress was increased to 85 percent of their room temperature tensile strength. However, if the rods were cooled to room temperature before testing, no loss in tensile strength occurred.

At temperatures above 300°C (570°F) the tensile strength decreased more rapidly and at 500°C (930°F) the tensile strength was about 50 percent of that at room temperature. (5) This is comparable to the behavior of prestressing steels.

Long-Term Loading Characteristics

When a glass fiber reinforced element is subjected to a sustained load, creep will occur. For Polystal rods at 20°C (68°F), if the stress due to the sustained load is greater than approximately 70 percent of the tensile strength in a short-term test, then failure will eventually occur. (3) For example, for a sustained stress of 75 percent of the short-term tensile strength, failure occurred in about 1700 hours. (3) However, when a stress equal to 64 percent of the short-term tensile strength was maintained for 140 days, there was no reduction in the tensile strength of the element when it was then subjected to a short-term test to failure. (9) Rehm and Schlottke have shown that the percent reduction in strength caused by sustained load is independent of the "free length" of the test specimen. (10) Tests of 1/4 in. diameter glass fiber reinforced rods by Wines, et al., resulted in failures after 10 and 19 days under stresses equal, respectively, to 88 and 73 percent of the short-term tensile strength. (14)

Failure under sustained load is known as "creep-rupture" or "stress rupture." The sustained service load stress in a glass fiber reinforced prestressing tendon, i.e., the effective prestress, must be sufficiently low to ensure adequate safety against creep-rupture. Rostásy suggests that this objective can be achieved by limiting the effective prestress to 50 percent of the short-term tensile strength. (3) Somes suggested limiting sustained load stresses to 0.7 of the stress which can be sustained without failure. (15) This corresponds to 50 percent of the short-term tensile strength, as proposed by Rostásy. (3)

When a glass fiber reinforced element is subjected to a sustained strain, then "relaxation" (the converse of creep) will occur, i.e., the stress in the element will decrease with time. Tests at 20°C (68°F) indicate that, for an initial stress level equal to 50 percent of the short-term tensile strength, the stress loss in a Polystal glass fiber reinforced tendon after 100 years would be about 4 percent. (3) Although he does not quote specific values, Franke asserts that creep (and consequently the relaxation) of glass fiber reinforced tendons does not increase significantly as the temperature increases. (5)

Fatigue Behavior

In common with other glass fiber reinforced composites, Polystal rods do not exhibit a fatigue limit stress range below which fatigue failures does not occur. (16) Data reported by Rostásy show that for 7.5-mm (0.295 in.) diameter Polystal rods tested in air at 20°C (68°F) with a minimum stress of 736 MPa (107 ksi), the stress range to cause fatigue failure at 2 million cycles is about 57 MPa (8.3 ksi). (3) This is only about one-third the corresponding stress range for prestressing steel. However, if bridge members prestressed with Polystal tendons are designed not to crack under service load, then the range of stress which would occur in the Polystal tendons would only be about a quarter of that which would occur in comparable steel tendons because of the difference in their moduli of elasticity. Hence, the fatigue strength of Polystal tendons should be satisfactory, providing flexural members are designed to ensure that cracking does not occur under service load.

Environmental Effects

It is necessary to consider the possible adverse effects of the presence of moisture, or of acidic or alkaline environments, on the strength of glass fiber reinforced composites. There is little published information on the influence of environment on the performance of Polystal rods. Strabag-Bau and Bayer report in their literature that satisfactory performance was obtained from Polystal tendons stressed to 50 percent of their short-term tensile strength and surrounded either by cement mortar for periods up to 1 year or by an aqueous cement liquid of pH = 13.0 for periods up to 2 weeks, but no other details are reported. (2)

Rostásy states that "Most fiber reinforced plastic materials are resistant against a series of media highly corrosive for carbon steel, such as: normal and polluted atmosphere, seawater and deicing salt solution, mineral oil, solutions with pH < 10. Strong alkaline solutions, however, lead to a loss of long-term strength . . . . This necessitates the permanent protection of a glass fiber reinforced plastic element by a polymeric sleeve if it is to be embedded in concrete or a cementitious grout." (3) Polystal rods are provided with such a sleeve, a tightly fitting cover of polyamid.

Tests of glass fiber reinforced laminates indicate that water, acid, and alkali solutions can diffuse into them, even when there are no micro-cracks in the resin matrix. (16-24) This occurs more rapidly at elevated temperatures.

Immersion in water at room temperature until maximum water absorption was attained did not cause physical damage to unloaded glass fiber reinforced plastics (GRP) with either polyester or epoxy resin materials. (17, 18) Epoxy based GRP elements exposed to moisture at temperatures of 113°F and above suffered permanent damage in the form of cracks in the epoxy matrix and debonding at the fiber-matrix interface. (18, 19) Although diffusion of moisture into GRP elements at room temperature does not have any significant effect on short-term strength (17, 18, 20), reference (16) indicates that it can result in a reduction in creep rupture strength, particularly if there is cyclic wetting and drying.

Tests have indicated that contact with acids, even at room temperature, can cause a reduction in the creep rupture strength of GRP elements. (21, 22, 23) Acid diffuses through the resin matrix and attacks the glass fibers. Damage is most severe in directly stressed fibers, causing a "stress-corrosion" failure. The time to failure at a given stress level decreases as the acid concentration increases. (21)

Bridge concrete is more commonly exposed to a salt laden environment than to an acidic environment. Previous research has not generally considered the effect of salt on the stress-corrosion fracture of GRP elements. One study did conclude that the creep rupture strength of GRP elements was the same when exposed to distilled water or to water with a low concentration of sodium chloride. (24) Data are not available on the behavior of GRP elements when subjected to salt concentrations of the magnitude that can be present in bridge deck concrete.

Although alkalies can have the same deleterious effects on GRP elements as acids, their effects have not been fully explored. (15) This is because GRP elements have been used much more extensively in acidic environments, e.g., for pipes in chemical plants.

Concrete is a strongly alkaline environment. Tests have indicated that with the use of a polyester resin highly resistant to chemicals and treatment of the glass fibers with an appropriate coupling agent, glass fiber reinforced elements highly resistant to a strong alkaline environment can be produced. (2, 5) Polystal rods are such elements.

The available information indicates that if glass fiber reinforced plastic rods are to be directly embedded in concrete (as in pretensioned, prestressed concrete), then they must be provided with an appropriate polymeric cover. This cover must not be removed with a view to reducing the transfer length, if the long-term integrity of the structure is to be ensured. In post-tensioned, prestressed concrete applications, additional protection against an aggressive environment can be ensured with use of an appropriately formulated resin grout rather than a cement mortar grout. This is done in the post-tensioning system developed by Strabag-Bau and Bayer. (2)

ANCHORAGE OF FIBER REINFORCED ELEMENTS

Fiber reinforced elements such as Polystal rods cannot be anchored with the tapered wedge grip typically used to anchor steel tendons. This is because the transverse strength of the rods is only a small fraction of their longitudinal tensile strength (see Table 2). The combination of the lateral compressive force exerted by the wedges as they are driven home and the locally very high bearing stresses resulting from the serrated inner surface of the wedges crushes the rod. This behavior was reported by several investigators. (9, 14, 15, 25)

Modified friction grips have been used in which the taper of the wedges was greatly reduced and their inner faces made smoother. (15) These grips developed 93 percent of the specified strength of the rod. Another type of friction grip has been developed, in which two 300-mm (11.8 in.) long plates with circular arc grooves cut into their inner faces are clamped onto the rod with special bolts and crushable washers. (9) These limit the transverse stress acting on the rod. The ends of the rod were also wrapped in a lead foil. These grips can develop the strength of the glass fiber reinforced rod, but they are not suitable for practical use in prestressed concrete construction.

An alternative approach is to rely on adhesion between the surface of the rod and some suitable material cast around it. In two instances anchorage was successfully attained by casting concrete blocks around the ends of uncoated glass fiber reinforced rods. (14, 25) However, this would not be suitable for use in prestressed concrete construction because the long-term integrity of the rods could not be guaranteed. Also, the blocks were large and cumbersome.

In the last decade, adhesion type anchorages have been developed in Germany using various types of polymer mortar contained within a steel sleeve. This sleeve was threaded on its inside face. (3, 9) This type of anchorage has been used to anchor tendons of from 1 to 22, 7.5-mm (0.295 in.) Polystal rods. In these anchorages, the polymer mortar protects the glass fiber reinforced rods as well as transferring force to them. This type of anchorage, if made a suitable length, can develop the full strength of a single rod and up to 97 percent of the strength of multiple rod tendons. The anchorage used in the Strabag-Bau system is of this type, as is the anchorage being developed by Con-Tech Systems, Inc. (2) (Details of the Con-Tech anchorage are shown in Figure 5(a).)

Failure of this type of anchorage appears to be an inter-laminar shear failure occurring just below the surface of the rods, rather than a bond failure at the surface of the rods. Anchorage failure is very abrupt.

IMPACT ON DESIGN

The unique physical characteristics of glass fiber reinforced (GFR) tendons impact design in two ways:

1. through the influence of their physical properties on the behavioral characteristics of prestressed concrete elements, and
2. through the effects of such factors as the size and number of GFR tendons required, the minimum curvature of tendons possible, and the size and number of anchorages required on the proportioning of prestressed concrete elements.

IMPACT ON BEHAVIORAL CHARACTERISTICS

Virtually no detailed reports are available on the testing of concrete elements of practical proportions prestressed by GFR tendons. Bayer and Strabag show the load-deflection curve for an 8-m (26.25 ft) span T-beam post-tensioned with Polystal GFR tendons, but no details are given as to the beam cross-section, the prestressing tendons, or the material properties. (2)

Somes reported successful tests of four pretensioned, prestressed concrete model beams of 5-ft span with rectangular sections varying from 3 x 2 in. to 3 x 6 in. (15) Wines, et al., reported successful tests of four pretensioned, prestressed concrete beams of 6-ft span with a rectangular section of 4 x 9 in. (14) In both cases, the prestress force acted at the lower third point and the GFR rods used for the prestressing elements had no external protective coating.

Somes reported that his beams failed in flexure. (15) He calculated the flexural strengths from first principles, satisfying equilibrium and compatibility. He used the measured stress-strain relationship for the GFR rods and the concrete compressive stress distribution characteristics proposed by Hognestad, Hanson, and McHenry. (26) The average ratio of test strength to calculated strength for the four beams was 1.05.

Wines, et al., reported that in all cases failure was initiated by crushing of the concrete in the flexural compression zone. (14) (In two of the beams, this was followed by diagonal tension cracking.) They did not calculate the member strengths from first principles. This has now been done, and it was found that the average ratio of test strength to calculated strength for the four beams is 1.00.

It therefore appears reasonable to use established methods to calculate the behavior of members prestressed with bonded GFR tendons to compare their behavior and strengths with those of members prestressed with bonded high strength steel tendons. This has been done for two classes of member used in bridge construction, in which GFR tendons might be used. The two types of member are the rectangular section (corresponding to bridge decks resisting moments in the transverse direction) and the T-section (corresponding to typical principal bridge members).

The study team decided that it would be reasonable to compare the strength and behavior of members which differ only in the type of tendon used to provide the prestressing force. That is, the prestressing force would be the same in two comparable members, but in one case the force would be provided by GFR tendons and in the other by seven wire 270K prestressing strand. The stress-strain relationships for these two materials are shown in Figure 1.

The stress-strain relationship for the GFR tendon is that of "Polystal" and is as shown in the literature of Bayer and Strabag. (2) The relationship is linear up to failure at 220 ksi (1520 MPa). The modulus of elasticity is 7,400 ksi (51,000 MPa). The strain at failure is 0.030.

The stress-strain relationship for seven wire strand is that of stress-relieved strand, which just meets the requirements of ASTM Standard A416-80, "Standard Specification for Uncoated Seven-Wire Stress-Relieved Steel Strand for Prestressed Concrete."

The effective prestress in the GFR tendons was assumed to be 50 percent of ultimate, i.e., 110 ksi, as recommended by the manufacturer, Bayer AG. The effective

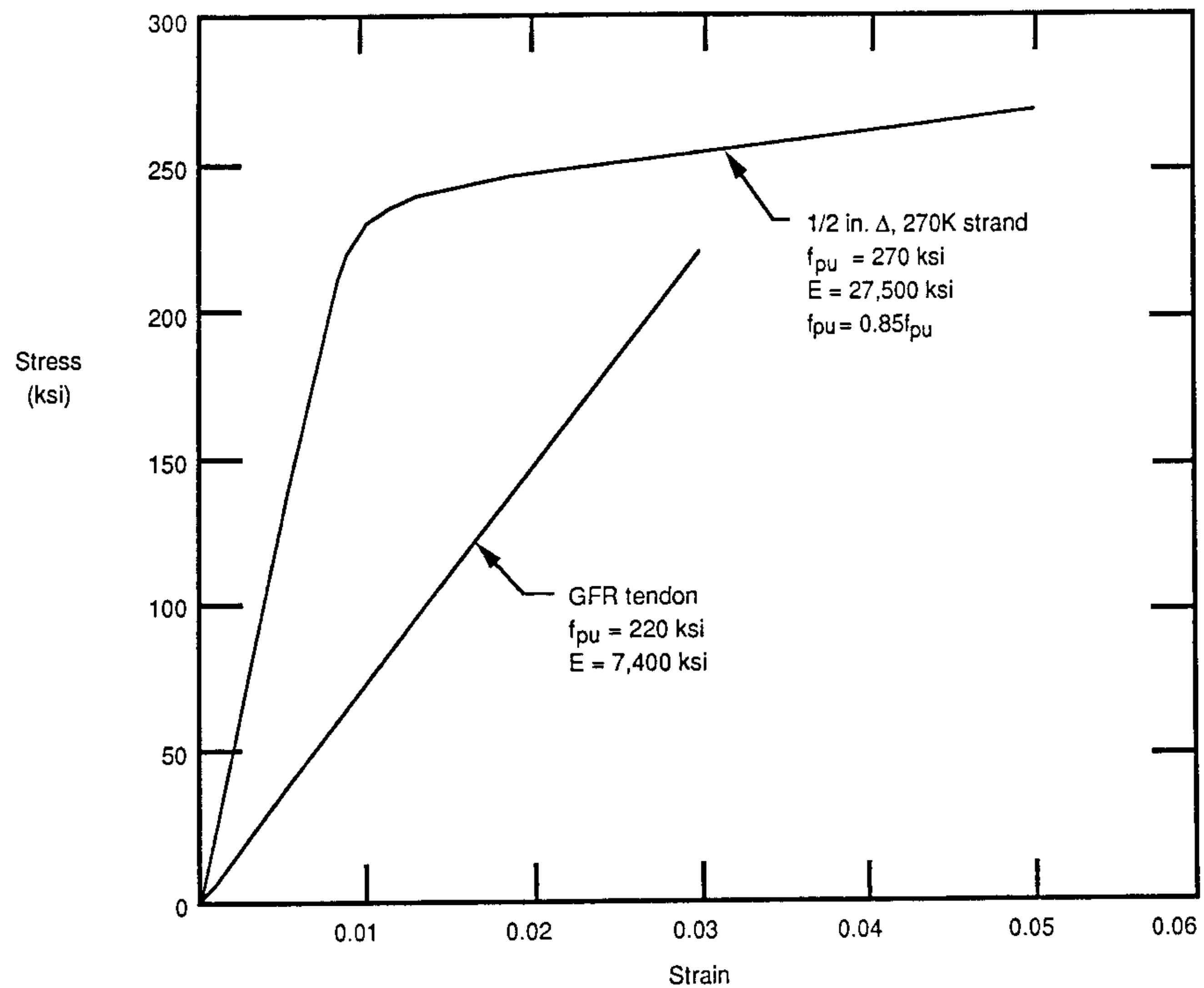

Figure 1. Stress-strain Curves Used in Calculations

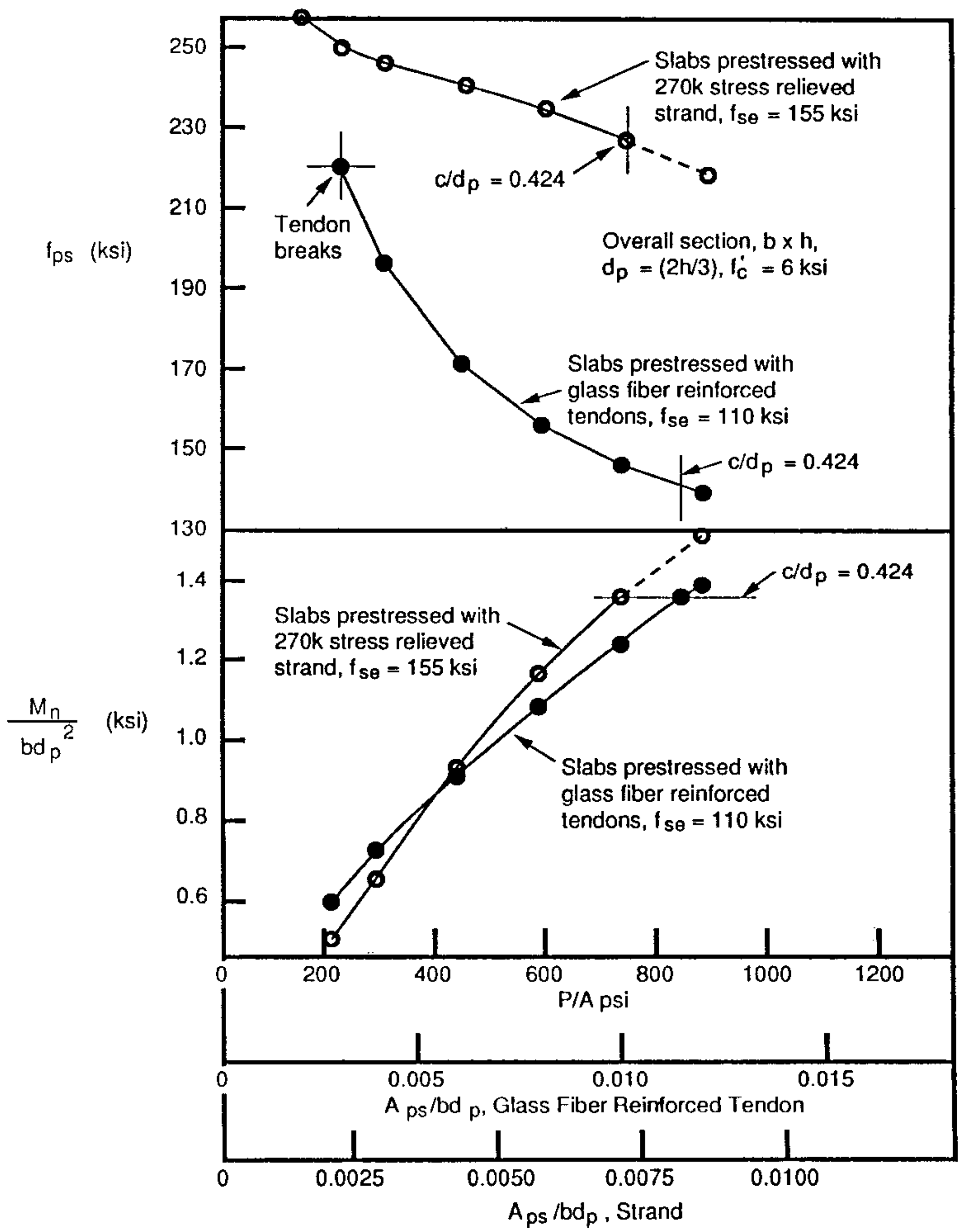

Figure 2. Behavior of Concrete Slabs Prestressed with Strands and with Glass Fiber Reinforced Tendons

prestress in the strand was assumed to be 155 ksi. The concrete cylinder strength was assumed to be 6000 psi.

The calculations were carried out using a calculator program described elsewhere. (27) This program follows an iterative procedure until equilibrium and compatibility are satisfied simultaneously.

BEHAVIOR OF PRESTRESSED SLABS WITH BONDED TENDONS

The tendon stress at ultimate, f_{ps}, and the nominal moment strength, M_n, were calculated for six pairs of slab sections with average concrete prestress values of from 214 psi to 880 psi. The effective depth of the prestressing tendons, d_p, was assumed to be two-thirds the total depth, h. Hence, the stresses caused by prestress varied from zero at the top face to a maximum at the bottom face.

The results of the calculations are presented in Figure 2. Because of the different effective prestress values for the two types of tendon, the reinforcement ratio A_{ps}/bd_p is different for comparable sections which have the same prestress force (P). The values of the reinforcement ratios for both types of tendon are indicated by the additional scales at the bottom of this figure.

It can be seen that as the reinforcement ratio increases, the tendon stress at flexural ultimate decreases less rapidly in the case of steel strands than in the case of GFR tendons. This is because, for all the cases considered, the prestressed strand is strained into the flatter part of the strand stress-strain curve at flexural ultimate, whereas the GFR tendon stress/strain curve is relatively steep up to failure. The result is that as the reinforcement ratio increases, the flexural strength increases more rapidly for the slabs prestressed with strand than for the slabs prestressed with GFR tendons.

At the minimum reinforcement ratio used in these calculations, the GFR tendons break at flexural ultimate as the concrete crushes. If a smaller area of GFR tendon were used, then failure would occur by breaking of the tendon before the flexural compression zone crushed, i.e., before the maximum concrete compression strain reached the value of 0.003 normally assumed in flexural strength calculations. This results in a loss of ductility in addition to a violent failure.

This type of failure would not occur in a slab prestressed with strand, until a reinforcement ratio corresponding to providing only about one-half the prestress force provided in the GFR tendon slab that first fails by breaking the tendon.

The value of 0.424 for the ratio of the neutral axis depth at ultimate, c, to the effective depth of the prestressed reinforcement, d_p, corresponds to the maximum reinforcement index of $0.36\beta_1$ allowed for prestressed concrete beams in the ACI Code. (28) It can be seen that the maximum moment strength which can be developed if this limit on c/d_p is observed is the same for slabs reinforced with either strands or GFR tendons. However, the area of GFR tendon required to develop this maximum flexural strength is about 1.65 times the area of strand required to develop the same flexural strength.

The approximate equations for the stress in a bonded prestressing tendon at flexural ultimate, f_{pu}, which are contained in the ACI Code (28) and the AASHTO Bridge Specifications (31), were developed to reflect the behavior of steel tendons. Since, as seen in Figure 2, the relationship between tendon stress at flexural ultimate and reinforcement ratio for GFR tendons is different from that for steel strand, it is inappropriate to use the approximate equations to calculate f_{pu} in a bonded GFR tendon.

To further study the influence of tendon type on ductility and deformation behavior, moment-rotation curves were developed for four comparable pairs of slabs having d_p equal to two-thirds the slab thickness. These slabs ranged from a lightly reinforced case in which the GFR tendon breaks at flexural ultimate to about the most heavily reinforced case permitted for the strand prestressed slab, i.e., one in which the reinforcement index approaches $36\beta_1$.

For the uncracked section, the stresses at the top and bottom faces and the gravity load moment acting were calculated for the following conditions:

1. prestress only acting,
2. uniform stress across the section, i.e., zero curvature,
3. zero stress at the bottom face, and
4. at flexural cracking (assumed to occur when the stress at the bottom face became equal to $7.5\sqrt{f'_c}$).

In each of these cases, the corresponding strains were calculated and hence the curvatures. The rotations occurring in a length of member equal to its effective depth (d_p), were obtained by multiplying the curvature by d_p.

The moment-rotation curves for the section after it had been cracked were developed by calculation of the rotation, θ, and the moment, M, acting when the maximum concrete compression strain had successively higher values. These values of rotation and moment were obtained by an iterative process in which the neutral axis depth was adjusted until equilibrium and compatibility were satisfied simultaneously.

To calculate the resultant concrete compression force, the flexural compression zone was divided into up to eight strips, and the compressive force on each strip was calculated. A linear distribution of strain across the section was assumed and use was made of the following equation for the concrete stress/strain curve proposed by Kriz and Lee (29):

$$f_c^2 + A\varepsilon^2 + Bf_c\varepsilon + Cf_c + D\varepsilon = 0$$

where f_c is the concrete stress at strain ε. The values of the coefficients A, B, C, and D vary with the concrete compressive strength. Kriz and Lee showed that this equation can reproduce quite closely the concrete compression stress/strain curves obtained experimentally by Hognestad, Hanson, and McHenry. (26)

The force in the prestressed reinforcement was obtained from the total strain in that reinforcement and the appropriate reinforcement stress/strain curve.

When the tension and resultant compression forces were within 1 percent of one another, equilibrium was considered to have been satisfied, along with compatibility. The curvature of the section was calculated by division of the maximum concrete compression strain by the neutral axis depth. The rotation was obtained by multiplication of the curvature by the effective depth (d_p). The moment acting was obtained by summation of the moments about the neutral axis of the component concrete compression forces and the force in the tendon.

This process was continued until the moment of resistance reached a maximum value. If the tendon did not rupture, the process was continued until the calculated moment of resistance fell without increase in the curvature of the section. The relationships obtained are shown in Figure 3.

The behavior before cracking is essentially the same for both sets of slabs, since the section stiffness is little affected by the properties of the small amount of reinforcement. After cracking, the strand reinforced slabs are significantly stiffer than those prestressed with GFR tendons. This implies a more rapid increase in deflection under moderate overload conditions for the slabs prestressed with GFR tendons than for those prestressed with strand.

The reduced stiffness of the GFR reinforced slab after cracking is due to this reinforcement having a smaller stiffness than that of the strand in the comparable slab. To provide equal prestress forces in the comparable slabs, A_{ps} for the GFR tendon is about 1.4 times that of the strand. However, the modulus of elasticity of the GFR tendon is only about one-quarter that of strand. Hence, the stiffness of the GFR tendon is only 35 percent of that of strand used to provide the same prestressing force. This smaller extensional stiffness of the GFR tendon will also lead to wider cracks for a given load than when strand is used.

Except for the most heavily reinforced slab, the stiffness of the GFR reinforced slabs after the cracks open remains essentially constant up to failure. Unlike the strand reinforced slabs, there is no significant change in stiffness at loads approaching failure to provide warning of impending failure. Failure is also more abrupt.

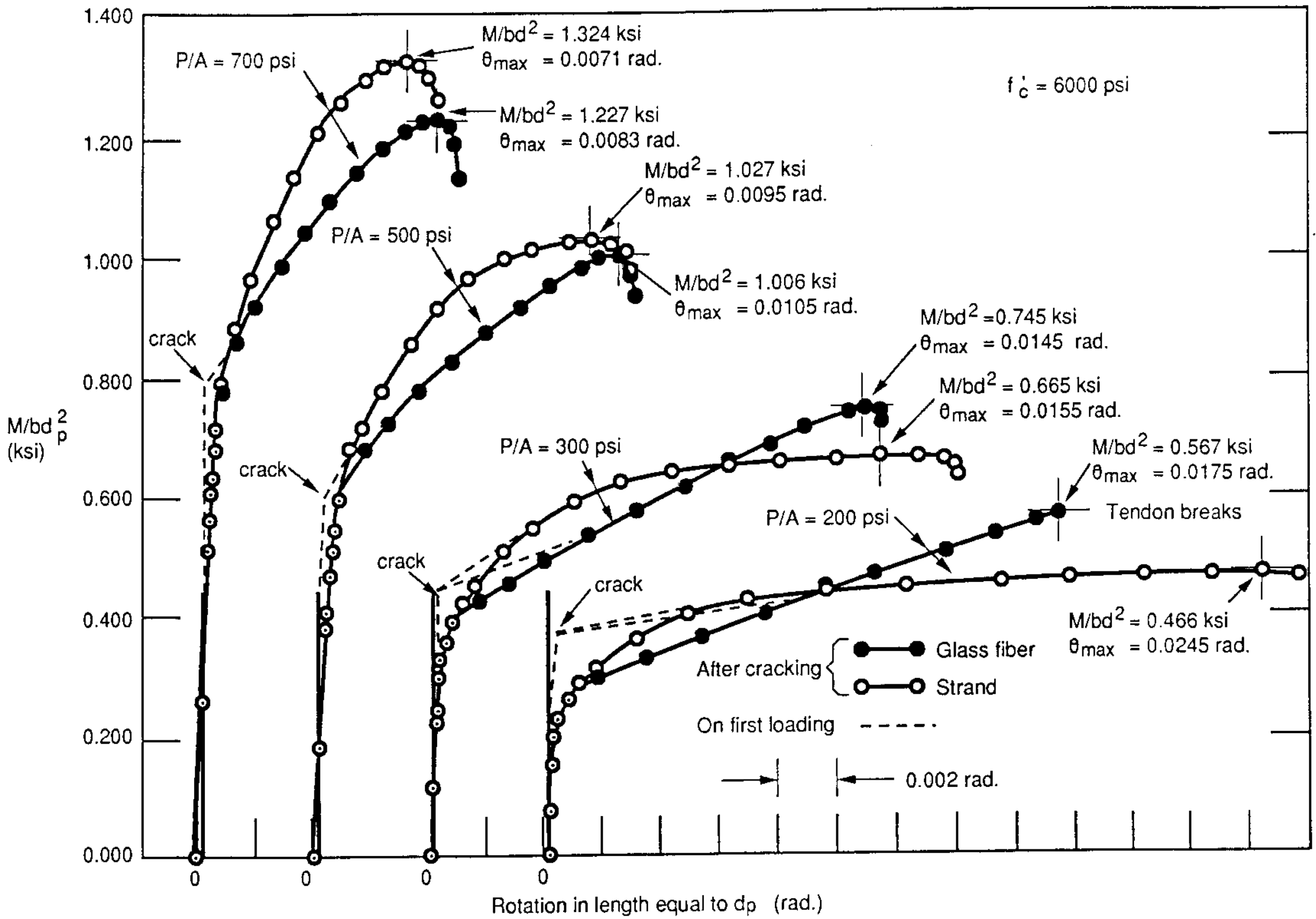

Figure 3. Moment-rotation Curves for Concrete Slabs Prestressed with Strand and with Glass Fiber Reinforced Tendons

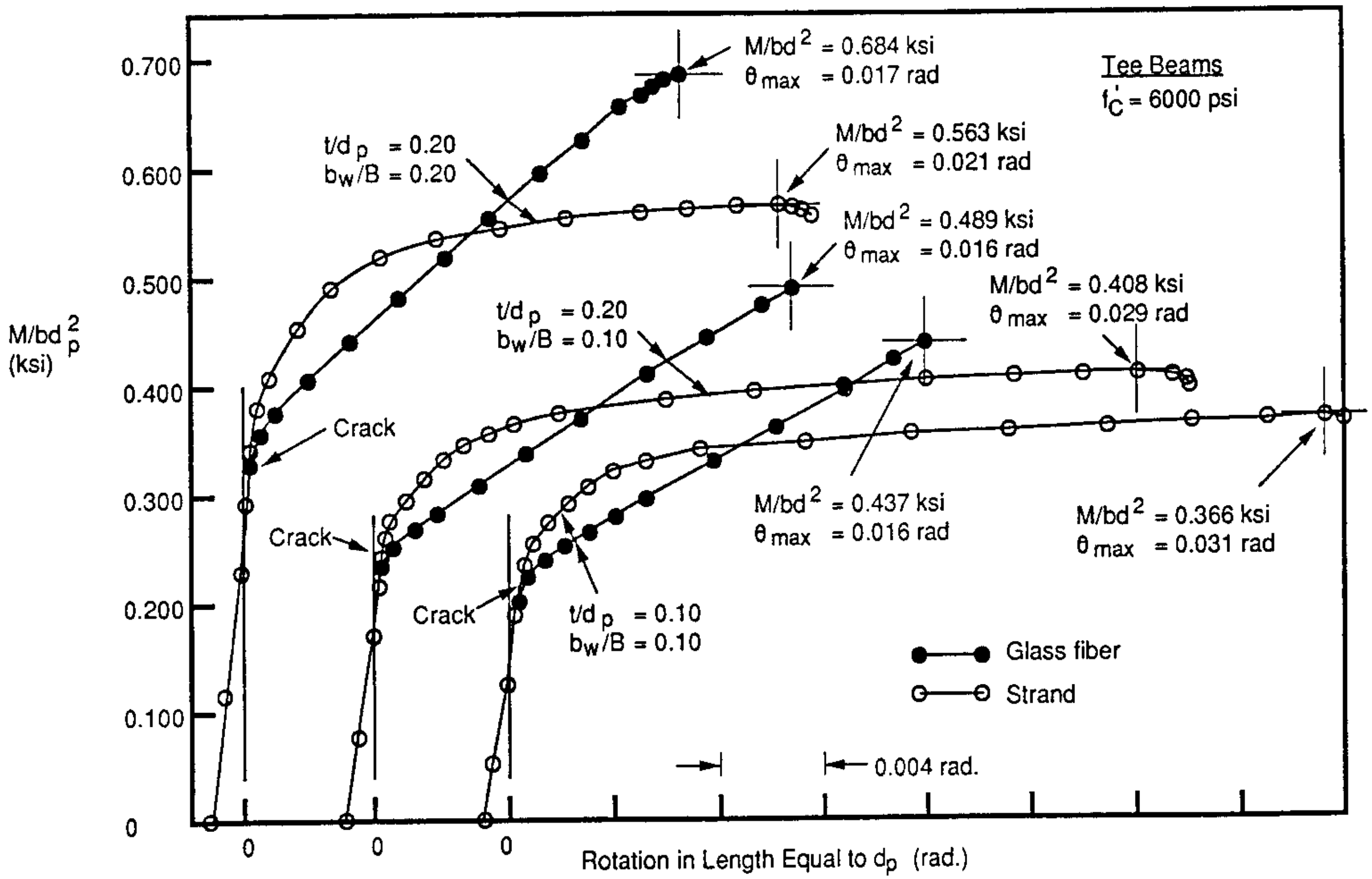

Figure 4. Moment-rotation Curves for Concrete T-section Members Prestressed with Strand and with Glass Fiber Reinforced Tendons

In the most lightly reinforced slab the GFR tendon breaks, while the strand does not. In the more heavily reinforced slabs, the moment of resistance of the GFR reinforced slabs drops off more rapidly after maximum moment than does that of the strand reinforced slabs, indicating a more abrupt or brittle type of failure.

The maximum moments developed by the slabs according to these calculations agreed closely with those calculated in the conventional manner, using the equivalent rectangular stress distribution and assuming a maximum concrete compression strain at failure equal to 0.003. (The moment rotation calculations indicated that the maximum concrete compression strain was within 10 percent of 0.003 for all cases, except that of the most lightly prestressed slab reinforced with GFR tendons. In this case it was 0.0026.)

BEHAVIOR OF PRESTRESSED T-SECTIONS WITH BONDED TENDONS

Moment-rotation curves were calculated for three pairs of prestressed T-sections for which the ratios of flange thickness to effective depth and of web width to flange width were, respectively, 0.1 and 0.1, 0.2 and 0.1, and 0.2 and 0.2, thus covering the range of values likely to be used in practice. The ratio of total depth to effective depth was 1.1. The prestress force in each member was that corresponding to the occurrence of a compressive stress at the bottom face of 2700 psi and zero stress at the top face, under dead load. The bottom face stress corresponded to $0.45f'_c$ for the 6000 psi cylinder strength assumed for the concrete. The only difference between the members of each pair was that in one case the prestress force was provided by GFR tendons with an effective prestress of 110 ksi, while in the other it was provided by strand with an effective prestress of 155 ksi.

The procedure used to develop the moment-rotation curves was the same as already described for the case of the prestressed concrete slabs. In the case of the T-sections, values of moment and rotation were calculated for the situation when the top fiber stress was zero, in addition to the cases previously enumerated for the slab section. The curves developed are shown in Figure 4.

In all cases the GFR tendons are predicted to break at flexural failure, while the stress in the strands reaches from 250 to 258 ksi. Also, in all cases the moment-rotation relationship after opening of the cracks is almost linear for the GFR reinforced sections, with no significant change in behavior approaching failure to provide warning of impending failure.

As in the case of the slabs, the stiffness of the strand reinforced T-section members under moderate overload is significantly greater than that of the GFR reinforced members, for the same reasons.

The strand reinforced T-sections behave in a more ductile manner and provide ample warning of failure by the large increases in deformation as the failure moment is approached. The maximum concrete compression strain in the strand reinforced T-sections is 0.0030 in all three cases, indicating that the concrete crushes at failure.

In the two GFR reinforced T-sections whose b_w/B equals 0.10, the maximum concrete compression strain at failure is only 0.0022, indicating that the tendons break before the concrete develops its full compressive strength. This behavior occurs because the maximum increase in strain in the GFR tendons beyond that caused by prestress is only about 0.015, since the strain caused by prestress is about 0.015. In the case of strand, the strain due to prestress is only about 0.006, and therefore a very much greater increase in strain caused by loading to failure is possible, resulting in greater ductility and a less abrupt failure.

In the case of the section with both t/d_p and b_w/B equal to 0.20, the greater sectional area results in about a 50 percent increase in reinforcement over the intermediate case. This results in the simultaneous breaking of the GFR tendons and crushing of the concrete at a maximum concrete compression strain of 0.0031.

The T-sections prestressed with bonded GFR tendons develop higher moment strengths than those prestressed to the same level using bonded strands. This is because in members where the GFR tendons break at failure, the ratio of the force in the tendons at failure to the effective prestress force is equal to 2.0 for the GFR tendons but is only about 1.65 for the strand (which is not strained to breaking point).

INFLUENCE OF BOND ON BEHAVIOR

The foregoing discussion has been based on the assumption of good bond between the tendons and the concrete, so that the change in strain in the tendons can be assumed to be equal to the change in strain in the adjacent concrete. The tests of Somes (15) and Wines, et al. (14), showed this assumption to be valid for the case of uncoated GFR tendons. The question remains, however, as to whether this assumption is completely valid for GFR tendons, such as Polystal, with a protective coating, . Con-Tech Systems maintains that the assumption is valid because of the rough surface texture of the GFR tendon and of the polyamid coating. However, the polyamid coating is not chemically bonded to the GFR tendon. It is therefore considered advisable that the effectiveness of the bond between Polystal tendons and the surrounding concrete be tested and evaluated before the tendons are used in practice.

If the bond between the GFR tendons and the concrete is imperfect, then use could be made of the concept of a "strain compatibility factor," first suggested by Baker. (30) This is designated as F and is defined as

$$F = \frac{\text{change in strain in tendon}}{\left(\begin{array}{c}\text{change in strain of adjacent concrete} \\ \text{at section of maximum moment}\end{array}\right)}$$

This factor F is then introduced into the compatibility equation when flexural strength is calculated. If the bond is perfect, then F is unity. If some relative slip occurs, then F is less than unity. Baker suggested a value of 0.8 for F in the case of smooth, high strength steel wire used for a prestressing tendon. (30) Because there is no actual chemical bond between the GFR tendon and the polyamid coating in Polystal rods, use of a value of F of about 0.8 might be appropriate.

If F is less than 1.0, then the flexural strength can be less than would be developed with a perfect bond. With a very small amount of reinforcement the GFR tendon would still be broken and the flexural strength would be unaffected. However, as the amount of reinforcement increased, the stress in the tendon would be less than if a perfect bond existed between the tendon and the concrete.

The general shape of the moment-rotation diagrams developed earlier would be unchanged, but their slope after flexural cracking would be somewhat smaller, so that deflections and crack widths after cracking would be greater.

MEMBERS WITH UNBONDED TENDONS

The increase in stress in an unbonded tendon as a prestressed flexural member is loaded to failure is caused by the relative movement of the anchorages of the tendon. The increase in strain in the tendon is equal to the relative movement of the anchorages divided by the length of the tendon. The increase in stress in the tendon therefore depends on its extensional stiffness.

Because the extensional stiffness of a GFR tendon is only about 35 percent of that of a steel tendon designed to provide the same prestressing force, the increase in stress in the GFR tendon for a given deformation of the member would be only about one-third of that which would occur in the steel tendon. The flexural strength of the member reinforced with the unbonded GFR tendon would be correspondingly less than that of a similar member reinforced with an unbonded steel tendon.

The equations for the stress at flexural ultimate in unbonded prestressing tendons, contained in the ACI Code (28) and the AASHTO Bridge Specifications (31), are empirical and are based on the results of tests of members prestressed with steel unbonded tendons. It would therefore be incorrect to use them "as is" for unbonded GFR tendons. In view of the previous discussion, it would appear to be appropriate to reduce the calculated increase in stress by about two-thirds if these empirical equations were applied to beams prestressed with unbonded GFR tendons.

Another characteristic of GFR tendons, of concern when they are used as unbonded tendons, is that their strength decreases as the length of tendon subject to maximum stress increases. It may therefore be appropriate to establish a lower allowable stress at service load for unbonded GFR tendons than for bonded tendons, even though unbonded tendons rarely, if ever, develop their ultimate strength.

Consideration must also be given to the minimum radius of saddles used to deflect the tendons in unbonded construction. Because the GFR tendons behave elastically up to rupture and are relatively rigid in flexure, the minimum radius around which they can be bent is relatively large. If the increase in stress caused by bending around a saddle is limited to 10 ksi, the minimum radius of bend in a 7.5 mm (0.295 in.) diameter rod is 109 in. In addition, consideration must be given to the fact that the transverse strength of the Polystal rod is only about 20 ksi. (9) (Bayer and Strabag in their literature suggested that in bonded post-tensioned construction, bending stresses caused by curvature of bars may be neglected for bend radii in excess of 8 m (26 ft). (2) This limiting radius corresponds to the development of a stress from bending of 3.5 ksi and a bearing stress of 0.08 ksi.)

PRETENSIONED, PRESTRESSED CONCRETE MEMBERS

Con-Tech Systems has suggested that Polystal GFR rods could be used in pretensioned, prestressed concrete members. However, before this can be done, specific information is needed on two facts,

1. the "transfer length" needed to transfer the prestress force from a pretensioned Polystal rod into the concrete surrounding it, both at time of transfer and after a period of time, and
2. the "development length" needed to develop the tensile strength of the Polystal tendon by bond in a pretensioned, prestressed concrete beam.

Expressions for transfer length and development length currently in use are empirically based and relate only to the use of seven-wire prestressing strand.

In addition, tests should be made to determine an appropriate value of the compatibility factor (F) for Polystal rods directly embedded in concrete (as distinct from the case of post-tensioned rods in a duct filled with resin mortar).

FATIGUE BEHAVIOR

Currently, no lower limit range of stress has been established for which a fatigue failure of GFR tendons will not occur. However, the range of stress to cause failure after 2 million cycles of load has been established at about 8.3 ksi. (3) This is significantly lower than the fatigue limit for seven-wire strand. However, this is not considered to be a matter for concern as long as a concrete member prestressed by GFR tendons remains uncracked, since, because of the tendons' low modulus of elasticity, the range of stress in them for a given cycle of loading will only be about one-quarter that which would occur in seven-wire strand. To guard against the possibility of problems with fatigue, it therefore appears to be essential to design members prestressed with GFR tendons with a definite margin of safety against flexural cracking.

LONG-TERM BEHAVIOR

The strength of GFR tendons subjected to sustained load decreases as the length of time under load increases. (3) Tests indicate that the sustained stress which would cause failure after 100 years is about 70 percent of the short-term tensile strength. Rostásy (3) and Bayer and Strabag (2) propose that the service load stress (i.e., the effective prestress) be not more than 50 percent of the specified short-term tensile strength to ensure an adequate safety margin against "stress-rupture" caused by sustained stress.

The loss of prestress caused by stress relaxation after 100 years is about one-third greater for GFR tendons than for "low relaxation" type prestressing steel. (3) This is compensated for by the much lower prestress loss which will occur in GFR tendons as a result of shrinkage and creep of the concrete. The loss in GFR tendons will be lower because the elastic modulus of GFR tendons is about one-quarter that of steel. However, the percentage of loss of prestress force in the GFR tendon that is caused by concrete creep and shrinkage will not be one-quarter of that which would occur in a comparable steel tendon. Because the initial stress level in the GFR tendon is only about 60 percent of that in the steel tendon, the loss of prestress force in the GFR tendon would be about 40 percent of that occurring in the comparable steel tendon.

EFFECT ON MEMBER PROPORTIONS OF PHYSICAL CHARACTERISTICS

Member proportions are to some extent governed by the space requirements of the tendons necessary to provide the prestress force required. Because the effective prestress stress in GFR tendons is only about 70 percent of that in seven-wire strand, an approximately 40 percent larger area of tendon is needed. In addition, the GFR rods currently available are only 7.5 mm (0.295 in.) in diameter, with a cross-sectional area of 0.0685 in.^{2}, as compared with a cross-section of 0.153 in.^{2} for the commonly used 1/2-in. diameter, seven-wire 270K strand. To provide a given prestress force, the number of GFR rods required would be 3.13 times the number of 1/2-in. diameter strands. The cross-sectional area necessary to accommodate the prestressing tendons is therefore significantly greater when GFR tendons are used in place of strand.

A GFR tendon consisting of 19 # 7.5-mm (0.295 in.) diameter rods requires a duct 55 mm (2.17 in.) in diameter and, at an effective prestress of 110 ksi, provides a prestress force of 143 kips. This is approximately equal to the prestress force provided by six 1/2-in. diameter, 270K seven-wire strands working at an effective prestress of 155 ksi. It would therefore be necessary to provide two 19-rod GFR tendons to replace each 12-strand tendon, taking up considerably more space.

If 7.5-mm (0.295 in.) diameter GFR tendons were used in a pretensioned, prestressed concrete beam at 1-1/2 in. centers, they would provide a prestress force of

3.35 kips for each square inch of concrete surrounding the tendons. If 1/2-in. diameter 270K strands are used at 2-in. centers, they can provide a prestress force of 5.93 kips for each square inch of concrete surrounding the tendons. It therefore appears that for a given prestress force, GFR pretensioned tendons would take up almost twice the cross-sectional area that pretensioned strand would occupy.

It therefore appears that in some cases it might be difficult to fit the GFR tendons into the available space in the tensile zone of a typical I-section member. Also, in some situations the required tendon area could result in a reduced available effective depth, requiring a further increase in tendon area.

As noted earlier in the discussion of unbonded tendons, GFR tendons behave elastically up to rupture and are relatively rigid in flexure. Therefore, the minimum radius around which they can be bent is relatively large. For an ultimate strength of 220 ksi and an effective prestress of 110 ksi, the outer fibers of a 7.5-mm (0.295 in.) GFR rod would reach 220 ksi and commence to fail when the bend radius of the rod was 9.9 in. It is clear therefore that GFR rods could not be deflected in a pretensioning bed in the manner used for strand. Hence it appears that only straight pretensioned GFR tendons could be used and that these would have to be combined with draped, post-tensioned GFR tendons.

The anchorages for GFR post-tensioning tendons are significantly larger than those used with strand post-tensioning tendons to anchor the same force. In particular, the anchorage must extend much farther into the end of the member to accommodate the 450-mm (17.7 in.) long sleeve in which the individual GFR rods are embedded in resin mortar. Further, a length of enlarged duct, equal to the extension of the tendon due to prestress, must be provided beyond the final location of the anchorage sleeve. This is because the tendon with its anchorages must be placed in the form before concrete is cast, since the tendon is stressed by pulling on the anchorage sleeve.

The space taken by the GFR tendon anchorage would be a significant drawback to the use of GFR tendons for the transverse prestressing of bridge decks.

PRINCIPAL IMPACTS ON DESIGN

Based on the foregoing discussion, the following appear to be the principal impacts on design.

1. Because of the fact that the strength of GFR tendons under sustained load decreases with time under load, it is necessary to restrict the effective prestress in GFR tendons to 50 percent of their short-term tensile strength.

2. Members prestressed with GFR tendons fail abruptly and without the warning provided by the more rapid increase in deflection approaching failure that occurs in members prestressed with strand. In particular, failure of typical T-sections will occur by breaking of the tendon. It may therefore be appropriate to require the use of somewhat higher than normal load factors or lower than normal ϕ factors in the design of members prestressed with GFR tendons.

3. It appears desirable to ensure a specific margin of safety against flexural cracking at service loads in members prestressed with GFR tendons for the following reasons:

    a. After cracking, the deflections and crack widths will increase much more rapidly in members prestressed by GFR tendons than in members prestressed by steel tendons.

    b. The variation of tendon stress under service loads is small enough to present no fatigue problem, as long as the member remains uncracked. However, the larger tendon stresses after flexural cracking raise questions about satisfactory fatigue behavior if cracking occurs.

4. There is some uncertainty as to the bond developed between coated GFR tendons (such as Polystal) and the surrounding concrete. Tests are needed to clarify this and, if necessary, to establish a suitable value for the

compatibility factor, F, for use in the calculation of the flexural strength of bonded prestressed members.

5. Currently used equations for the stress at flexural ultimate in bonded and in unbonded tendons should not be used for GFR tendons. This is because the current design equations are empirically based on tests and analytical studies of the behavior of members prestressed by steel tendons. Appropriate corresponding equations should be developed for GFR tendons.

6. In unbonded, post-tensioned members with GFR tendons, careful consideration should be given to the value used for the tendon ultimate strength because of the effect of tendon length on GFR tendon strength. Also, care must be taken to ensure that saddles used to deflect the tendons have large enough radii of curvature that they will not cause excessive flexural or lateral bearing stresses in the tendons.

7. Further experimental study is necessary to determine appropriate values for the "transfer length" and the "development length" of GFR tendons before they are used in pretensioned, prestressed concrete members.

8. If GFR tendons are used in pretensioned, prestressed concrete members, they cannot be "draped" in the same manner as strand because of the excessive bending stresses that would occur in the tendons. Post-tensioned, draped tendons could be used in place of pretensioned, draped tendons, providing their radii of curvature was not less than about 25 ft.

9. Attention must be paid to the space requirements for tendons and for anchorages when member cross-sections are proportioned. These requirements are more demanding for GFR tendons than for steel tendons.

TESTS OF CON-TECH ANCHORAGES

ANCHORAGES TESTED

Four tendon assemblies were provided by Con-Tech Systems Ltd. Each tendon consisted of eight 7.5-mm (0.295 in.) diameter Polystal rods and had an overall length of 10 ft., as required by the Post Tensioning Institute for the qualification of post tensioning anchorages. (32) At each end of the tendon the Polystal rods were embedded in epoxy grout in cylindrical steel anchorages. The details of the anchorages are shown in Figure 5.

The embedment length reported for each anchorage is the length of the Polystal rod, (from which the polyamid coating had been removed), which is embedded in the epoxy grout within the anchorage. A short length of the polyamid coating was also embedded in the epoxy grout, just inside the rubber end seal of the anchorage. This length was discounted when the embedment length was calculated, since over this length there was no direct contact between the epoxy grout and the fiberglass and polyester resin composite rod. In each case a fiberglass sleeve 200 mm (7.87 in.) long had been epoxied to the composite rod, after the polyamid covering was stripped and before embedding the rods in the anchorage.

The details of the anchorages used for tendon #1, shown in Figure 5(a), correspond to those which would be used in practice for post-tensioning. (The internally threaded hole at the end of the anchorage is engaged by a pull-rod, which passes through the center-hole ram used for tensioning the tendon.) Tendon #1 failed prematurely when the Polystal rods pulled out of the anchorages. The anchorages for tendons 2, 3 and 4 were therefore made longer to provide greater embedment length for the Polystal rods. Since these tendons were not actually to be used for post-tensioning, the internally threaded hole in the outer ends of the anchorages was omitted, as is seen in Figure 5(b).

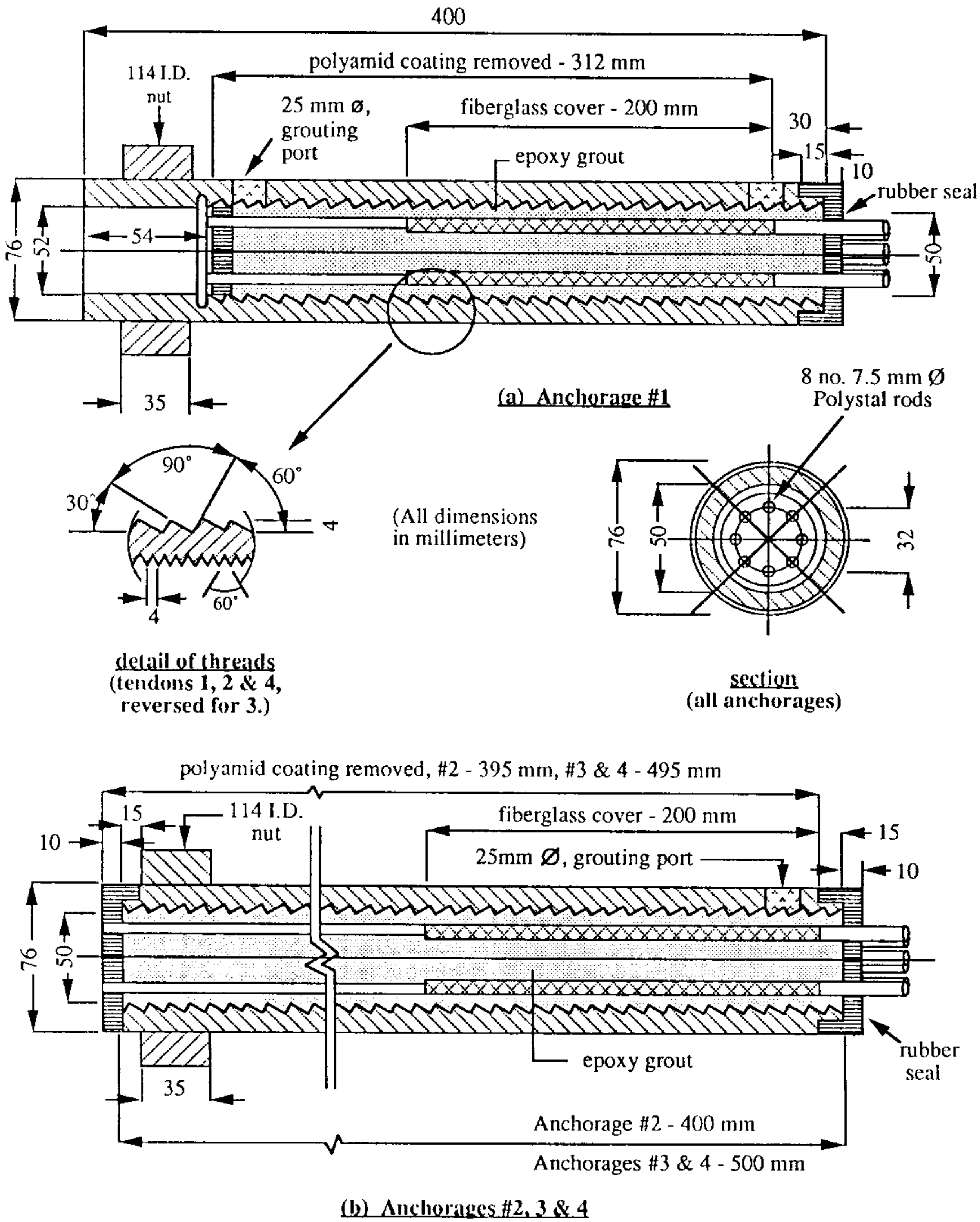

Figure 5. Anchorages Used in Tendon Tests

The embedment lengths provided in the anchorages for tendons 1 through 4 were respectively 300, 385, 485, and 485 mm (11.81, 15.16, 19.09 and 19.09 in.). The difference between anchorages 3 and 4 lay in the orientation of the faces of the internal thread. In #3 the shallow face of the thread provided the reaction to the load from the tendon. In all the other anchorages the steep face of the thread provided the reaction to the load from the tendon.

The Polystal rods used in the tendons tested had a guaranteed strength of 60 kN (13.49 kips) per 7.5-mm (0.295 in.) diameter rod.

TESTING ARRANGEMENTS AND PROCEDURES

The tendons were tested with a 300-kip capacity, Baldwin hydraulic testing machine. Nine-inch diameter, 1-1/2-in. thickness bearing plates, with a central hole 3-3/4 in. in diameter, were attached to the top and bottom faces of the upper and lower cross-heads, respectively. The anchorages were threaded through the holes in the bearing plates and the circular nuts were run onto the ends of the anchorages. In every case, the outer face of the nut was 5/8 in. from the end of the anchorage. The anchorages were carefully centered on the bearing plates before loading commenced.

In each case, the overall extension of the tendon was measured using a 3-in. travel 0.001 in./division dial gage, as shown in Figure 6(b). A Polystal rod was used to transfer displacement from the top anchorage to the bottom anchorage to provide automatic compensation for elongation or shortening of the tendon caused by temperature changes.

In the tests of tendons #1 and #2, linear variable differential transformers (LVDTs) were used to measure the amount one of the rods pulled out of the top and bottom anchorages, as shown in Figure 6(a). The LVDTs had a range of 0.5 in. and were monitored with a Vishay digital recorder. The displacements could be read to 0.001 in.

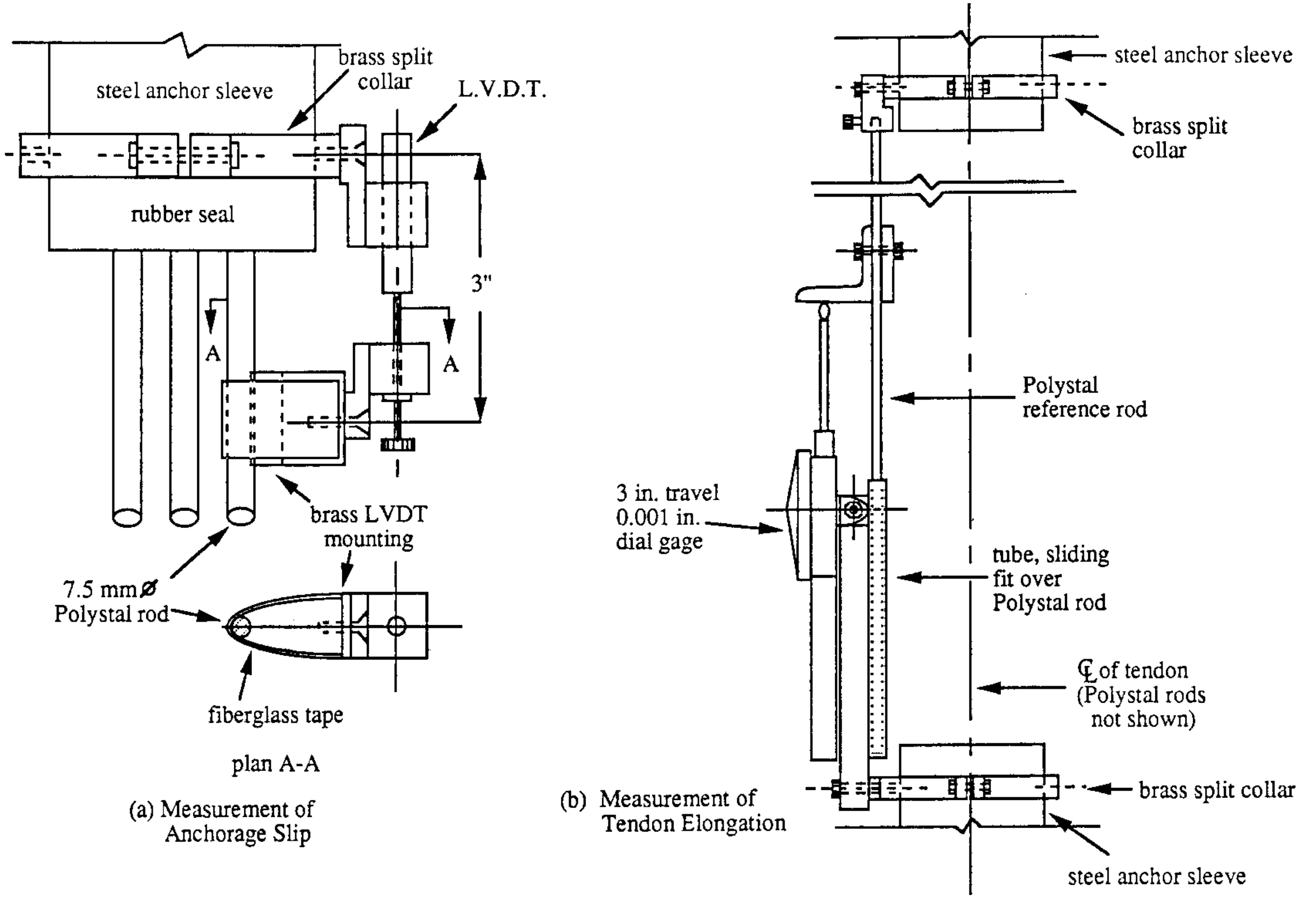

Figure 6. Instrumentation for Tendon Tests

These displacements included the extension of approximately 3 in. of the Polystal rod. The readings at low loads were also affected by initial straightening of the rods. (The Polystal rods had a small initial curvature because they had been coiled earlier.)

Two tests were run on each tendon. In the first test the load was increased by increments of 5 kips to a maximum of 60 kips. This load corresponded to a stress in the Polystal of 109.5 ksi. The load was then decreased to zero by increments of 10 kips.

In the second test the load was increased to 60 kips by increments of 10 kips. Subsequently, the load increments were decreased first to 5 kips, then 2.5 kips and finally to 1 kip, as the tendon was loaded to failure. At each load stage the load was maintained until the hand of the dial gage measuring the overall extension became stationary. Readings were taken on the dial gage and on the LVDTs at each load stage when the load was first attained and after any increase in reading, if an increase occurred.

SPECIMEN BEHAVIOR AND STRENGTH

In all cases there was no visible distress before failure, which occurred with great violence when the Polystal rods pulled out of the anchorages. Some of the rods pulled completely out of the anchorages, while others pulled out about 3 inches. Each rod pulled out of either the top or the bottom anchorage and shattered in compression at its opposite end.

The failure appeared to be instantaneous, a single loud bang being heard. However, it is possible that the failure initiated with the slip of those rods which pulled completely out of the anchorages. When this occurred, load would be transferred instantaneously to the remaining rods, causing them to slip.

The only forewarning of failure occurred in the case of tendon #1. In this case there was a sudden 1/8-in. slip of the instrumented rod at the upper anchorage at the penultimate load stage. This rod pulled out completely at failure.

In all cases the failures were inter-laminar shear failures within the Polystal rods. In no case was a bond failure observed between the epoxy mortar and the surface of the Polystal rod. The failures appear to have initiated close to the start of the polyamid coating, with the fracture of a very thin layer of glass fibers around the perimeter of the rod. Slip had then occurred between the inner surface of this layer of glass fibers and the core of the rod. In some cases additional glass fibers on the surface of the core fractured at points along the embedded length, leaving the rod with a taper of from 0.02 to 0.05 in. over its embedded length after pulling out.

The variation of overall tendon elongation with load is shown in Figure 7 for the four tests. The tendons behaved in an essentially linear fashion up to a load of about 60 kips. At higher loads the behavior became slightly non-linear, so that at ultimate the extension was about 10 percent greater than it would have been if the behavior had remained linear. This non-linearity at higher loads is probably due to increasing anchorage slip, which can be seen in Figure 8, or to deformation of the epoxy mortar within the anchorages.

The slight non-linearity in behavior at very low loads is due to the straightening of the initially slightly curved Polystal rods. Also, typically one or two of the rods were slightly longer than the remaining rods in the tendon. These rods would not straighten and pick up load until the remaining rods had stretched slightly.

The test results are summarized in Table 3. The values of elastic modulus obtained from the measured overall extension of the tendons relate to tendon stresses up to about 110 ksi. They are in reasonable agreement with the value of 51,000 MPa (7400 ksi) quoted by Bayer and Strabag in their product literature. (2)

The failure loads are compared with the total strength of the eight Polystal rods making up the tendon. The value used for the rod strength, 60 kN (13.49 kips) per rod, is the guaranteed minimum strength of the Polystal rods according to Con-Tech Systems Ltd. It can be seen that with 300-mm (11.81 in.) embedment, only 72 percent of the

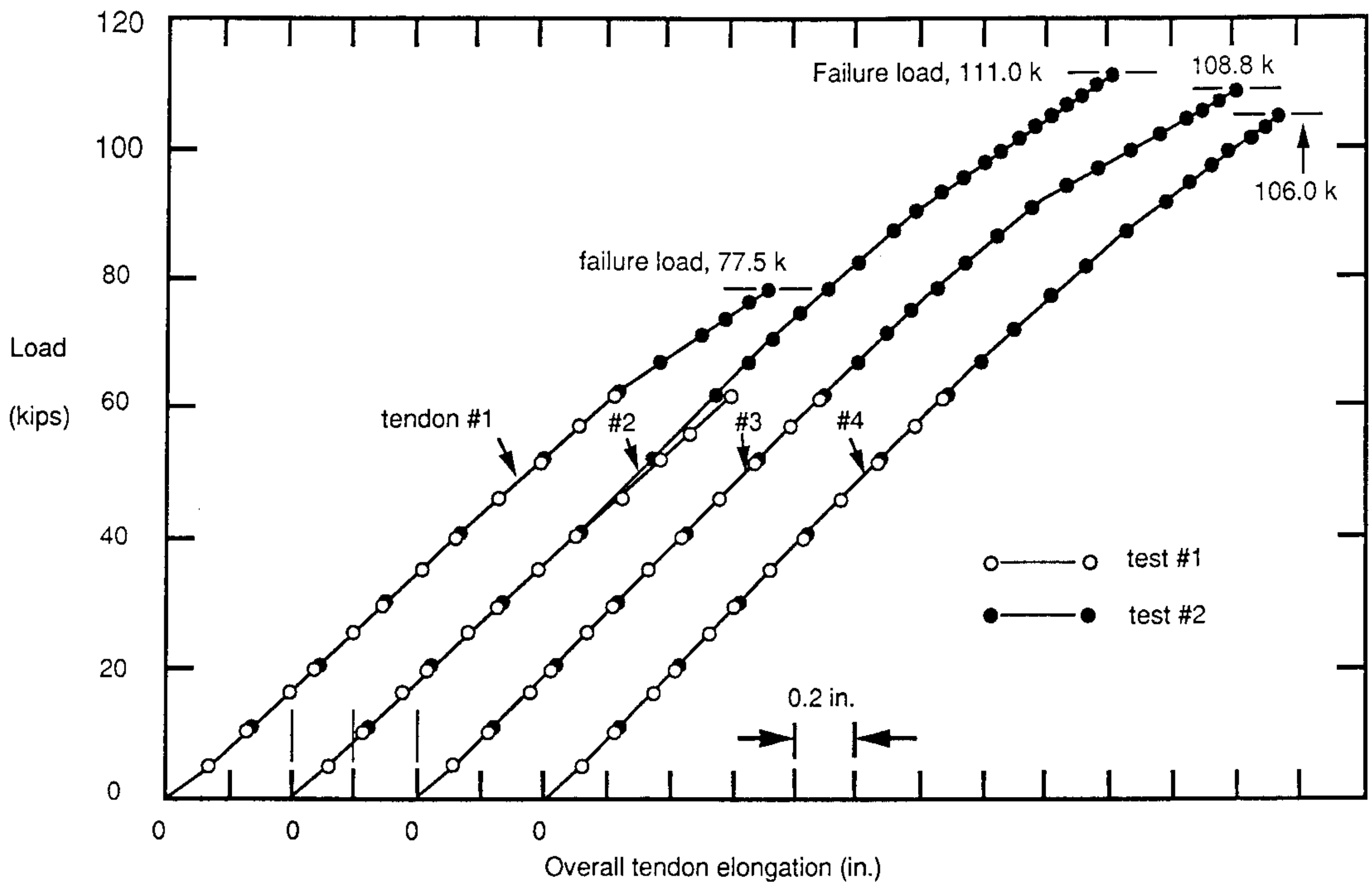

Figure 7. Overall Tendon Elongation with Load

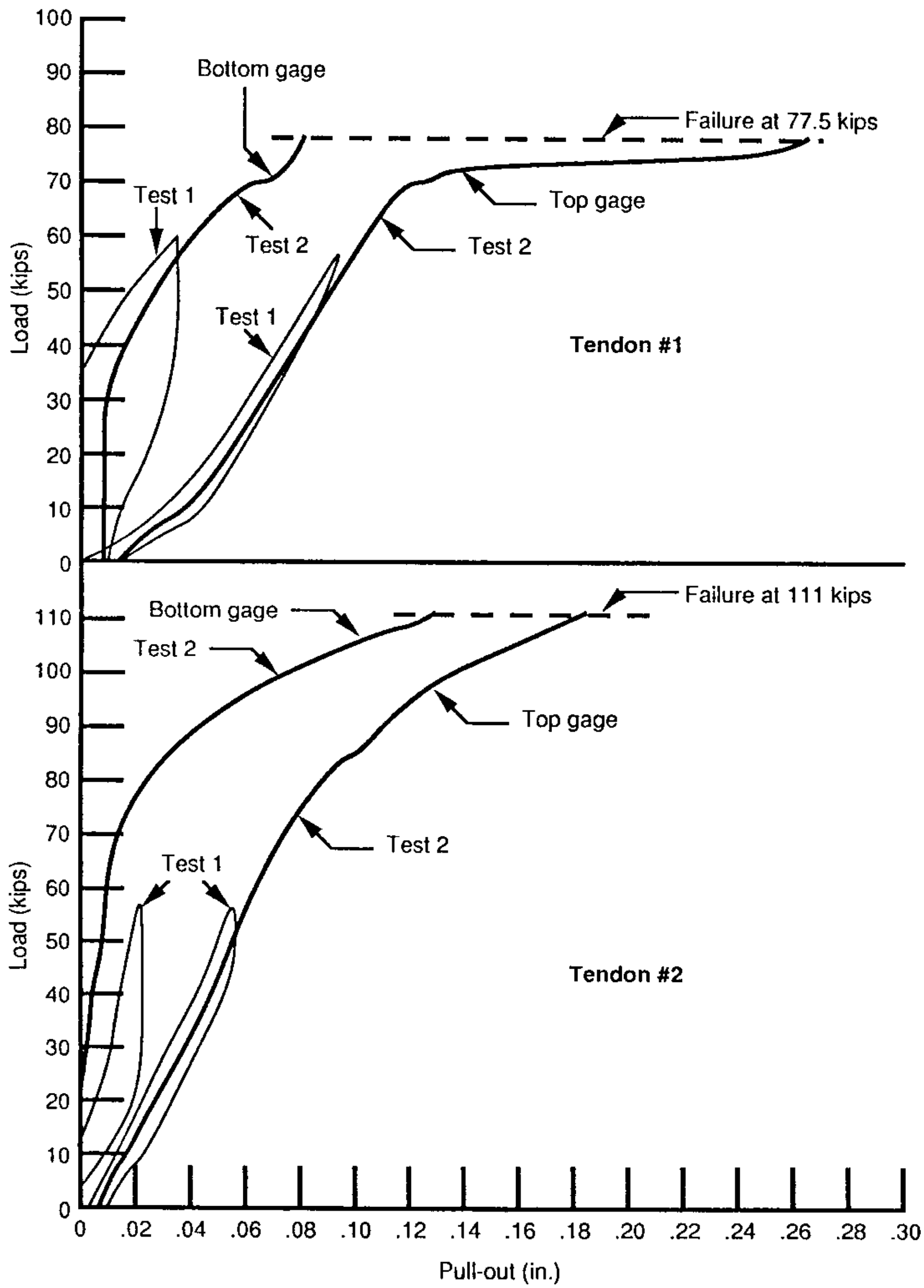

Figure 8. Rod Pull-out from Anchorages

Table 3. Summary of Test Results

Tendon No.	Elastic Modulus[a] (ksi)		Failure Load P_u (kips)	$\frac{P_u}{\text{(Tendon Strength)}}$ [b]	Anchorage Embedment Length (mm/in.)
		E		E 2	
1	7289	7484	77.5	0.72	300/11.81
2	7267	7365	111.0	1.03	385/15.16
3	7142	7245	108.8	1.01	485/19.09
4	7174	7279	106.0	0.98	485/19.09

a E_1 - based on length between inner ends of metal anchorage sleeves.

E_2 - based on length of rod covered by polyamid coating.

b Tendon strength (107.9 kips) based on guaranteed minimum strength of type of Polystal rods used [60 kN (13.49 kips) per 7.5 mm diameter rod.]

tendon strength was developed. However, with 385 or 485 mm (15.16 or 19.09 in.) the anchorage strength was approximately equal to the guaranteed minimum strength of these Polystal rods.

In these tests the rods were never broken in tension, so their actual tensile strength is not known. The tensile strength quoted by Bayer and Strabag in their literature is 1520 MPa (220 ksi). (2) If this strength is used as a basis for comparison, the ratios of anchorage strength to tendon strength become 0.64, 0.92, 0.90, and 0.88 for tendons 1, 2, 3, and 4, respectively. All these values are less than the minimum value of 0.95 required by the Post Tensioning Institute for qualification of a post tensioning anchorage. (32)

The failure of these anchorages occurred within the Polystal rods. The strengths developed are therefore probably dependent upon the actual strength of the Polystal rods. All that is known about these rods is that their tensile strength is in excess of the 13.88 kips per rod (202.6 ksi) developed in the test of tendon #2. If rods with an actual strength of 220 ksi and with an embedment length of at least 385 mm (15.16 in.) were used in this type of anchorage, it is possible that an anchorage strength of 0.95 of the tendon strength would be developed. However this would have to be verified by tests.

SUPPLEMENTAL TEST OF A POLYSTAL ROD

To provide information on the actual properties of the Polystal rods used in these tests, a single rod was tested. This rod had the same overall length as the tendons tested. It was anchored at each end by embedment in epoxy mortar in a 1-in. diameter steel tube. An embedment length of 300 mm (11.81 in.) was provided.

The single rod was first subjected to three cycles of loading and unloading by 0.8 kip increments to a maximum load of 7.2 kips, corresponding to a maximum stress of 105 ksi. Measurements were made of the extension of the rod with a standard, 2-inch gage length extensometer. Small areas of the polyamid coating were removed to permit the extensometer to be clamped directly onto the glass fiber-resin composite. The readings obtained at each load increment were within 0.0002 in./in. for the three cycles of loading. The load strain relationship was completely linear. On the basis of the averages of the three sets of readings, the elastic modulus was calculated to be 7747 ksi.

The rod was then loaded to failure after the extensometer was removed. Failure occurred when the rod pulled out of the bottom anchorage at a load of 12.95 kips, corresponding to a stress of 189 ksi. The failure was an inter-laminar shear failure in the rod of the same type as occurred in the tendons.

REFERENCES

1. "Polystal, New high-performance composite materials," brochure by Bayer AG, SN Group, D-5090 Leverkusen, Federal Republic of Germany.

2. "Heavy Duty Composite Bar HLV (made of Polystal) for Prestressing Tendons and Soil Anchors," brochure by Strabag Bau-AG, D-5000 Cologne, and Bayer AG, D-5090 Leverkusen, Federal Republic of Germany.

3. Rostásy, F.S., "New Materials for Prestressing," preprint of paper for Sept. 1988 FIP Symposium in Israel.

4. Gerritse, A. and Schûrhoff, H.J., "Prestressing with Aramid Tendons, a Non-Corrosive Alternative to Steel in Prestressed Concrete," Proceedings, 10th Congress of the FIP, New Delhi, India, 1986, pp. 35-44.

5. Franke, L., "Behaviour and Design of High-Quality Glass-Fiber Composite Rods as Reinforcement for Prestressed Concrete Members," in "Plastics in material and structural engineering" — Elsevier, 1982, pp. 171-174. (Proceedings of the ICP/RILEM/IBK International Symposium, Prague, 1981.)

6. Fujisaki, T., Matsuzaki, Y., Sekijima, K., and Okamura, H., "New Material for Reinforced Concrete in Place of Reinforcing Steel Bars," Proceedings, IABSE Symposium, Versailles, France, 1987, pp. 413-466.

7. Madatian, S.A. and Mikhailov, K.V., "Main Trends in Development of Efficient Reinforcement for Ordinary and Prestressed Concrete Structures," Proceedings, 10th Congress of the FIP, New Delhi, India, 1986, pp. 245-255.

8. Koenig, G., Oetes, A., Geigerich, G. and Miesseler, H.J., "Monitoring of the Structural Integrity of the Ulenbergstrasse Bridge in Dusseldorf." Report to Bayer AG, published by Mobay Corp. (a Bayer company), Speciality Products Division, Pittsburgh, PA 15205-9741, 18 pp. (undated).

9. Kepp, B., "Zum Tragverhalten von Verankerungen für hochfeste Stäbe aus Glasfaserverbundwerkstoff als Bewehrung im Spannbetonbau. Dissertation, Technical University of Braunschweig, 1984, pp. 145.

10. Rehm, G. and Schlottke, B., "Übertragbarkeit von Werkstoffkenwerten bei Glasfaser-Harz-Verbundstäben," Mitteilungen des Institut fur Werkstoffe in Bauwesen, Stuttgart University, 1987/3, 35 pp.

11. Metcalfe, A.G. and Schmitz, G.K., "Effect of Length on the Strength of Glass Fibers," Proceedings of ASTM Annual Meeting, June 1964, pp. 1075-1093.

12. Lifshitz, J.M. and Rotem, A., "Longitudinal tensile failure of unidirectional fibrous composites," Journal of Materials Science, Vol. 7, 1982, pp. 861-869.

13. McKee, R.B. and Sines, G., "A Statistical Model for the Tensile Fracture of Parallel Fiber Composites," Journal of Elastoplastics, Vol. 1, July 1969, pp. 185-199.

14. Wines, J.C., Dietz, R.T., Hawley, J.L. and Hoff, G.C., "Laboratory investigation of plastic-glass fiber reinforcement for reinforced and prestressed concrete," Report 1, Feb. 1966, Report 2, July 1966. Miscellaneous paper No. 6-779, U.S. Army Corps of Engineers.

15. Somes, N.F., "Resin-bonded glass-fibre tendons for prestressed concrete," Magazine of Concrete Research, Vol. 15, No. 45, Nov. 1963, pp. 151-158.

16. Heger, F.J., Chambers, R.E. and Dietz, A.G.H., "Structural Plastics Design Manual." ASCE manual of engineering practice No. 63, 1984.

17. Han, K.S. and Koutsky, J., "Effect of water on the interlaminar fracture behaviour of glass fibre-reinforced polyester composite," Composites, Jan. 1983, pp. 67-70.

18. Dewimille, B. and Bundsell, A.R., "Accelerated ageing of glass-fibre epoxy resin in water," Composites, Jan. 1983, pp. 35-40.

19. Kasturiarachchi, K.A. and Pritchard, G., "Water absorption of glass/epoxy laminates under bending stresses," Composites, July 1983, pp. 244-250.

20. Ishai, O., and Arnon, U., "Instantaneous effect of internal moisture conditions on strength of glass fiber reinforced plastics," "Advanced composite materials—Environmental effects," ASTM publication STP 658, 1978.

21. Hogg, P.G., Hull, D., and Spencer, B., "Stress and strain corrosion of glass reinforced plastics," Composites, July 1981.

22. Hoff, P.G., "Factors affecting the stress corrosion of GRP in acid environments," Composites, Vol. 14, No. 3, July 1983.

23. Jones, F.R., Rock, J.W., and Wheatley, A.R., "Stress corrosion cracking and its implications for the long term durability of E-glass fiber composites," Composites, Vol. 14, No. 3, July 1983.

24. Wyatt, R.C., Norwood, L.S., and Phillips, M.G., "The stress rupture behavior of GRP laminates in aqueous environments," "Composite structures," Applied Science Publishers, London.

25. Maguire, F.J., "Report on further investigation concerning the feasibility of the use of fiberglass tendons in prestressed concrete construction," M.S. thesis, Princeton University, 1960.

26. Hognestad, E., Hanson, N.W., and McHenry, D., "Concrete Stress Distribution in Ultimate Strength Design," Journal of American Concrete Institute, Vol. 52, December 1955, pp. 455-479.

27. Mattock, A.H., "Flexural Strength of Prestressed Concrete Sections by Programmable Calculator," Journal of the Prestressed Concrete Institute, Vol. 24, No. 1, Jan./Feb. 1979, pp. 32-53.

28. "Building Code Requirements for Reinforced Concrete (ACI 318-83)," American Concrete Institute, 1983.

29. Kriz, L.B., and Lee, S.L., "Ultimate Strength of Over-Reinforced Beams," Journal of Engineering Mechanics Division, ASCE, Proceedings Paper 2502, 86, EM3, June 1960, pp. 95-105.

30. Baker, A.L.L., "The Ultimate Load Theory Applied to the Design of Reinforced and Prestressed Concrete Frames," Concrete Publications, London, 1956.

31. "Standard Specifications for Highway Bridges," 13th Ed. 1983 and Interim Specifications to 1988, American Association of State Highway and Transportation Officials.

32. "Post Tensioning Manual," Post Tensioning Institute, Phoenix, AZ, 1976, pp. 138-142.