

Properties and Uses of Polymers in Concrete

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PREFACE

ACI Committee 548 has sponsored eleven symposia since its organization in 1971, to help fulfill its mission to report information on the use of polymers in concrete. Proceedings of the first five symposia were published by ACI in SP-40, SP-58, SP-69, SP-89, and SP-99, respectively. Papers from the next three symposia held in 1986 and 1989 were published either in *Concrete International* or in *ACI Materials Journal*. The proceedings of the ninth symposium were published in SP-137.

This volume contains the papers that were presented at the 10th and 11th symposia that were held in Minneapolis, Minn., and Tarpon Springs, Fl., in 1993 and 1994, respectively. There were a total of four sessions, the first titled "Polymer Concrete Overlays," the second "Recent Innovations in Polymer Concrete Technology," the third and fourth "Structural Properties of Polymer Concrete, Parts I and II." The sessions were chaired by Jack J. Fontana, Paul D. Krauss, and Al O. Kaeding.

As chair of ACI 548, I would like to express my thanks to these three gentlemen for chairing the sessions and arranging review of the papers. Thanks are due also to the authors who made the proceedings possible, as well as to those committee members who assisted by reviewing the printed papers.

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Chairman, ACI 548

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SP 166-1

Flexural and Fracture Properties of Glass Fiber Reinforced Polyester Polymer Concrete

by C. Vipulanandan and S. Mebarkia

Flexural behavior of a polyester polymer concrete was investigated by varying the polymer and fiber contents. The polymer content was varied up to 18% of the total weight of polymer concrete (PC). The chopped glass fibers were 13 mm long and the fiber content was varied up to 6% (by weight of PC). The fine aggregates were well graded with particle size varying from 0.1 to 5 mm and were mainly quartz. The fine aggregates and glass fibers were also pretreated with a coupling agent (γ -methacryloxypropyltrimethoxy silane, γ -MPS) to improve flexural and fracture properties of PC. In general, addition of fibers increased the flexural strength, failure strain (strain at peak stress) and fracture properties but the flexural modulus of PC remained almost unchanged. Addition of 6% fiber content and silane treatment of aggregates and fibers increased the flexural strength of 18% PC to 41.6 MPa (6,040 psi), almost doubling the strength of unreinforced 18% PC system. Crack resistance curves based on stress intensity factor (K_R -curve) have been developed for the fiber reinforced PC systems. A two-parameter relationship was used to predict the complete flexural stress-strain data. There is good agreement between the predicted and measured stress-strain relationships.

Keywords: Coupling agents; esters; fibers (discrete fibers); flexural strength; fracture properties; polymer concrete

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INTRODUCTION

Polymer concrete is produced using dry aggregates as filler and polymerizing monomers as binder. The composition of PC is determined by its applications, especially loading stress levels and chemical environment. The high-strength, rapid-setting and corrosive resistance makes polymer composite a potential material for structural repairs, and for new constructions which are regularly exposed to strong alkaline environments (1-4).

Polyester polymer is one of the most popular polymer binders used in PC (5). PC exhibits brittle failure (6,7) and therefore improving its post-peak stress-strain behavior is important. Hence developing better PC systems and also characterizing the flexural strength and fracture properties in terms of constituents are essential in aiding the efficient utilization of PC. In order to improve the post-peak behavior and toughness, glass fibers can be added to the PC matrix. Substantial experience and broader knowledge of the optimal compositions, properties and stress-strain relationships of the fiber reinforced PC are necessary with respect to design, production and quality control. The post-peak behavior and the strain softening stress-strain relationship are essential in evaluating the performance of the material for impact, earthquake and fatigue loading.

In this study, the flexural properties of the glass-fiber-reinforced PC are investigated at room temperature. For the PC systems, a well graded blasting sand was used as the filler and polymer content was varied between 10% and 18% by weight of the PC. The glass fibers were added up to 6% by weight of the PC. Also the role of silane coupling agent on the behavior of PC was studied. Relationships have been developed to represent the flexural stress-strain behavior and fracture resistance curve of PC.

RESEARCH SIGNIFICANCE

Polymer concrete is increasingly used in various applications and hence require better characterization of its flexural and fracture properties. Also methods to improve the performance of a polyester based polymer concrete have been investigated.

EXPERIMENTAL PROGRAM

The constituents used in manufacturing the PC are summarized in Table 1. The viscosity of the unsaturated polyester monomer varied between 40 and 50 poise. Polymerization of unsaturated polyester dissolved in styrene (a mix of 65% unsaturated polyester, 35% styrene) is by free radical copolymerization. Cobalt naphthenate (0.3% by weight of resin) was used as the promoter and methyl ethyl ketone peroxide (1.5%) was used as the initiator. The Blasting sand with sub-angular particles had a coefficient of uniformity of 5.8. The sand particles were mainly quartz and had a specific gravity of 2.65. The grain size ranged from 0.1 to 5 mm and was compared to fine aggregate recommended by ASTM C33-85 in Fig. 1. The 13 mm long chopped glass fiber elements have up to about 800 glass strands bonded together. The average diameter of a glass strand was 0.013 mm with a fiber tensile strength of 2,500 MPa (363 ksi), and modulus of 70 GPa (10,160 ksi). The silane coupling agent (γ -methacryloxypropyltrimethoxysilane, γ -MPS) was introduced into the PC by pretreatment of glass fibers and aggregates (8). The aggregates and glass fibers were treated by wetting them with 2% aqueous solution of silane coupling agent. The trimethoxy group undergoes hydrolysis in aqueous solution and hydroxyl groups are then available to form oxane bonds to the sand and glass fiber surface. The treated aggregates and fibers were allowed to dry at 100⁰ C for 24 hours prior to mixing with the resin. PC specimens were compacted in three layers in a teflon lined aluminum mold of dimensions 230 mm x 50 mm x 50 mm. All the flexure and fracture studies were performed on 33 mm thick PC specimens. The PC specimens were first cured at room temperature for a day and at 75⁰ C for an additional day prior to testing (9). A total of 92 beam specimens were tested. The experimental program was divided into four series. The first series included 24 unreinforced beams tested for statistical evaluation of the flexural strength, the second series included 36 beams with and without glass fibers, the third series included 24 beams with silane treated aggregates and fibers and the fourth series included 8 silane treated fiber reinforced notched beams. A diamond saw (2 mm thick) was used to notch the specimens to a maximum depth of 38 mm. During the test the cross head speed of the closed loop servo hydraulic testing machine was maintained constant at 0.05 mm/min. The specimens were tested in four-point bending (in some instances, mentioned on the corresponding plots, three-point bending was used). For the notched beam specimens, the crack mouth opening displacement (CMOD) was measured using knife edges glued to the specimen (10). The deflection of the beams was measured using a LVDT (Linear Variable Differential Transducer) with an accuracy of 2.5×10^{-3} mm (10^{-4} inch). For each test, both the load versus load-point deflection (for all specimens) and the load versus crack mouth opening displacement (for notched specimens) were monitored continuously using X-Y recorders (Fig. 2).

RESULTS AND DISCUSSION

The average density for the unreinforced systems ranged between 1.96 and 2.15 Mg/m³. The porosity was affected significantly by the workability of the mix and varied considerably within the range of the variables. The

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subjective measure of the workability was estimated based on a scale of four with four being the best and zero the worst. Subjective workability assessment was based on the mix flowability, compactability, fiber balling and handling ease or difficulty. Of the unreinforced systems, the 14% PC had good workability and low porosity. For this reason 24 beams were tested for statistical evaluation of the flexural strength and modulus.

Statistical Analysis

Factors such as material variability (polymer, aggregates, promoter, initiator), degree of compaction and curing conditions will affect the properties of PC. Hence it is appropriate to quantify the variation in strength of the material in terms of mean, standard deviation and distribution. The flexural strength of 14% PC are plotted on a normal probability scale in Fig. 3. A total of 24 specimens were tested. Linear variation of the data on the probability plot (coefficient of correlation $R = 0.98$) shows that the data may adequately be represented by a normal distribution. The mean flexural strength was 16.45 MPa (2,390 psi) and the coefficient of variation (C.O.V) was 5%. Figure 4 shows the normal distribution of the flexural modulus ($R = 0.97$). In this case the mean was 11 GPa (1,600 ksi) and the coefficient of variation was 8.5%. Similar distribution was observed for the failure strain and the mean failure strain was 0.15% and the coefficient of variation was 10%.

Effect of Glass Fibers

Several systems have been considered in this study where the polymer content varied from 10% to 18% and the fiber content varied from 0% to 6% (by weight of PC). Enhancement of the unreinforced PC properties may be achieved by addition of glass fibers, treatment with silane coupling agent or both. The general directions of the improved stress-strain curve of the resulting material are shown in Fig. 5. The improvement may be achieved by increasing the (1) strength, (2) failure strain, (3) modulus, or (4) by improving the post-peak relationship.

The first series of specimens were tested in four-point bending to determine the effect of glass fibers on the flexural properties of PC. The following may be concluded from the test results:

(1) Results from a total of 36 specimens are presented in Fig. 6. Within the range of variables investigated, addition of glass fibers improved the flexural strengths of the 14% and 18% PC systems but decreased the strength of the 10% PC system. In the 10% PC system there is inadequate polymer to bind the aggregates and fibers resulting in reduced strength. The flexural strength varied from about 13 MPa (1,890 psi) for the 10% PC with 6% fiber content to about 33 MPa (4,800 psi) for the 18% PC with 6% fiber content. Addition of 6% glass fiber increased the flexural strength of 18% PC by 80%. Flexural stress-strain relationships for the 18% PC system reinforced with glass fibers up to 6% are shown in Fig. 7.

(2) The flexural modulus of 18% PC system was 11 GPa (1,600 ksi) and remained almost unchanged with the increase in fiber content (Fig. 8). The

standard deviation was about 0.94 GPa (136 ksi). The modulus did not increase due to the addition of glass fibers because of the following factors (a) addition of fibers generally resulted in an increase in void ratio due to reduced workability and (b) only up to 6% of fibers were used and fibers partly replaced the aggregates with similar modulus.

(3) Failure strain of 0.17% for the unreinforced 18% PC was increased to 0.6% with the addition of 6% glass fibers. Adding fibers to the PC produced a bridging effect at the crack tip and hence slows the crack propagation which results in an increase of the failure strain (Fig 7).

Stress-strain Relationship

A two-parameter relation (Eq. (1)) was used in predicting the stress-strain relationship of PC (13). This relationship may be written as

$$Y = \frac{X}{q + (1 - q - p) X + p X^{(q+p)/p}} \quad (1)$$

Y and X are defined as follows:

$$Y = \frac{\sigma}{\sigma_f} \text{ and } X = \frac{\epsilon}{\epsilon_f} \quad (2)$$

where σ and ϵ are the stress and strain; σ_f and ϵ_f are the strength and the failure strain and p and q (p and q vary from 0 to 1) are the material parameters to be determined from experimental data. The parameter q is the ratio of secant modulus at peak stress to initial modulus and so the lower the value of parameter q the greater the nonlinear pre-peak behavior of the PC system. This is also reflected by the decreasing value of parameter q with increase in fiber content, for the 18% PC system. The parameter q was equal to 1 for the unreinforced PC and decreased to 0.76 and 0.32 for PC with 4 and 6 wt% fiber contents respectively. The parameter p controls the post-peak curve and is determined by least square fitting the experimental data points. The steeper the descending stress-strain curve, the smaller the p value (p = 0 for brittle material). Figure 9 shows the normalized stress-strain relationships and the predictions agree very well with the experimental results. As shown in Fig. 9 the post-peak curve of the 6% fiber reinforcement descends more rapidly than the 4% fiber system and only the parameter p quantitatively describes this important observation. The parameter p was 0, 0.26 and 0.03 for the 18 wt% PC with 0, 4 and 6 wt% fiber contents respectively.

Effect of Silane Coupling Agent

Using the silane treated constituents in PC improved the workability for the 10% PC system. This improvement is probably due to the modification of the filler surface which became smoother and therefore increased the flowability of the polymer leading to a better dispersion of the constituents. It has been

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observed that silane treated fillers provide lower viscosities in filled resins than do untreated fillers (14). The PC specimens with treated aggregates (third series with 24 beams) were tested to determine the enhancement in flexural strength due to the silane treatment. The results (average of 2 specimens tested in three-point bending) are shown in Fig. 10. The increase in flexural strength are compared to the untreated PC. The maximum increase was 70%. It is noted that the treated unreinforced PC with the lowest polymer content (10%) has a higher strength than the untreated PC with the highest polymer content (18%). As shown in Fig.10, the increase in flexural strength due to silane treatment is very much dependent on the polymer content. The highest strength for the silane treated fiber reinforced system (18% polymer and 6% fiber) was 41.6 MPa (6,040 psi) . Silane treatment had the highest effect on the 10% PC system showing up to 70% increase in flexural strength and the lowest effect was on the 18% polymer and 6% glass fibers (less than 10% increase). The effect of silane on the increase in flexural modulus of glass fiber reinforced PC is shown in Fig. 11.

Fracture Properties

The performance of PC materials are affected by cracks (15) and their growth during loading. Hence eight silane treated glass fiber reinforced PC beam specimens were tested in four-point bending to quantify the resistance of the material during crack growth. The notch-to-depth ratio varied from 0.26 to 0.80. The data points needed to plot the fracture resistance curves are shown in Fig. 12. Using the concept of linear elastic fracture mechanics (LEFM), the relationship between elastic crack mouth opening displacement (CMOD^e) and the corresponding crack length in four-point bending can be represented as (16)

$$\text{CMOD}^e = 4 \sigma a V(\alpha) / E' \quad (3)$$

where σ is the net stress ($6M/bd^2$); M is the applied pure bending moment; a equal to $(a + H_0) / (d + H_0)$ with H_0 being the clip gage holder thickness; a is the crack length; E' is equal to E (modulus) for plane stress and $E/(1 - \nu^2)^{0.5}$ for plane strain where ν is the Poisson's ratio. An empirical formula with 1% accuracy for any a is used to calculate $V(a)$ and is expressed as

$$V(\alpha) = 0.8 - 1.7 \alpha + 2.4 \alpha^2 + 0.66 / (1 - \alpha)^2 \quad (4)$$

Hence if during slow growth CMOD^e could be determined at various loading levels by unloading the specimens, and hence using Equation (3) it would possible to determine the corresponding crack length. A numerical iterating procedure was used to determine the corresponding crack length a . Crack extension Δa is equal to $(a - a_i)$, where a_i is the initial crack length. Assuming a beam of cross section $b \times d$ with effective crack length a , the stress intensity factor was calculated using the equation for the four-point bending developed by Brown and Srawley¹⁷. The relationship is as follows:

$$K_I = 6 M a^{1/2} Y(a/d) / b d^2 \quad (5a)$$

where

$$Y(a/d) = 1.99 - 2.47 (a/d) + 12.97 (a/d)^2 - 23.17 (a/d)^3 + 24.80 (a/d)^4 \quad (5b)$$

Glass fiber reinforced polyester PC shows substantial amount of nonlinearity up to peak stress and hence resistance curves (R-curves, ASTM E 561-81) are used to characterize the resistance to fracture during slow stable crack extension in such materials.

To construct the K_R curve, effective crack length based on the CMOD method was used. Once the load P and the corresponding effective crack length is known from the test records, the value of K_I was obtained from Equation (5). As shown in Fig. 12, K_R - Da relationship (solid lines) can be best expressed in the following linear form

$$K_R = K_0 + \rho (\Delta a) \quad (6)$$

where K_0 , and ρ are parameters obtained from least square fit of the data. The parameter K_0 for 18% PC (treated blasting sand) with 2, 4, 6% fibers are 1.13, 1.16 and 1.15 MPa $m^{0.5}$ and the values for ρ are 0.20, 0.56 and 0.43 MPa $m^{0.5}/mm$ respectively.

CONCLUSIONS

The influence of glass fibers and silane coupling agent on a polyester based polymer concrete (PC) was investigated at room temperature. Based on the experimental study the following can be concluded:

1. Addition of glass fibers increased the flexural strength and the strain at peak stress but did not change the flexural modulus of the PC systems. The 18% PC with 6% glass fiber had a flexural strength of 33 MPa (4,800 psi), about 80% increase over the unreinforced PC system.
2. Use of silane treated aggregates and fibers further improved the strength of the PC systems. Silane treatment had the greatest effect on the 10% PC system. Silane treatment improved the flexural modulus of various PC systems. Silane treatment doubled the flexural strength (three-point bending) of 18% PC with 6% glass fiber to 41.6 MPa (6,040 psi).
3. The two-parameter relationship effectively predicted the flexural stress-strain relationships for the glass fiber reinforced and unreinforced PC systems.
4. Crack resistance curves based on stress intensity factor (K_R - Da relationship) have been found to be best approximated by linear relationships.

ACKNOWLEDGEMENT

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REFERENCES

1. Murray, A. M., "Epoxy Injection Welds Cracks Back Together," *Concrete Construction*, January 1987, Vol. 32, No. 1, pp. 45-49.
2. Cremaschi, J., "Polymer Concrete Bridge Overlays," *Concrete International*, Vol. 8, No. 5, May 1986, pp. 58-60.
3. Scarpinato, E. J., "Thin Polymer Concrete Bridge Deck Overlays," *Concrete Construction*, Vol. 29, No. 4, August 1984, pp. 711-715.
4. ACI Committee 548, "Polymer Concrete," SP-89, American Concrete Institute, Detroit 1985, 346 pp.
5. ACI Committee 548, "Guide to the Use of Polymer in Concrete," *ACI Journal*, Vol. 83, No. 5, 1986, pp. 798-829.
6. Vipulanandan, C. and Paul, E., "Mechanical Properties of Epoxy and Polyester Polymer Concrete Systems, Report No. UHCE 88-13, 1988, 165 pp.
7. Vipulanandan, C. and Paul, E., "Performance of Epoxy and Polyester Polymer Concrete," *ACI Materials Journal*, Vol. 87, No. 3, May-June 1990, pp. 241-251.
8. Vipulanandan, C. and Dharmarajan, N., "Bonding Agent in Polyester Polymer Concrete," *Lewis H. Tuthill Symposium SP-104-5*, 1987, pp. 89-106.
9. Vipulanandan, C., "Curing and Constitutive Relationships for the Polyester Mortar," *Polymer Engineering and Science*, Vol. 29, No. 22, 1989, pp. 1628-1635.
10. Vipulanandan, C. and Dharmarajan, N., "Influence of Aggregates in the Fracture Properties of Polyester Polymer Concrete," SP-116-11, 1989, pp. 177-192.
11. American Society for Testing and Materials, "Standard Method of Test for Flexural Toughness of Fiber Reinforced Concrete (Using Beam with Third-Point Loading)," *ASTM Standards for Concrete and Mineral Aggregates*, Vol. 04.02, Standard Number C 1018, 1985, pp. 637-644.
12. ACI Committee 544, "Measurement of Properties of Fiber Reinforced Concrete," *ACI Journal*, Vol. 85, No. 6, November-December 1988, pp. 583-593.
13. Vipulanandan, C. and Paul, E., "Performance of Epoxy and Polyester Polymer Concrete," *ACI Materials Journal*, Vol. 87, No. 3, 1990, pp. 241-251.
14. Plueddemann, E. P., *Silane Coupling Agent*, Plenum Press, New York, 1982.

15. Dharmarajan, N. and Vipulanandan, C., " Fracture Toughness of Particle Filled Fiber Reinforced Polyester Composites," Journal of Applied Polymer Science, Vol, 42, 1991, pp. 601-607.
16. Tada, H.; Paris, P. C. and Irwin, G. R., The Stress Analysis of Cracks, Handbook, Del Research Corporation, Hellertown, PA, 1973.
17. Brown, W. F. Jr and Srawley, J. E., " Plane Strain Crack Toughness Testing of High Strength Metallic Materials, " ASTM, STP 410, 1966.
18. International Congress on Polymers in Concrete, American Concrete Institute, San Francisco, California, September 1991.

TABLE 1 — COMPOSITION OF POLYMER CONCRETE

Constituent material	Relative weight %
Matrix-Polyester Resin	10 - 18
Initiator-MEKP	1.5*
Promoter-Cobalt Napthenate	0.3*
Aggregate-Blasting Sand	82 -90

MEKP-Methyl Ethyl Ketone Peroxide; *by weight of resin

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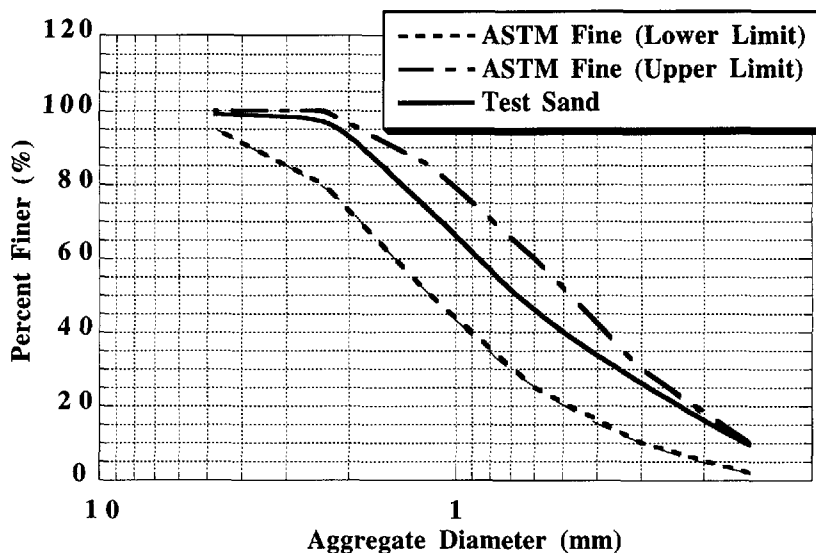


Fig. 1—Particle size distribution of sand compared to ASTM C 33-85 for fine aggregates

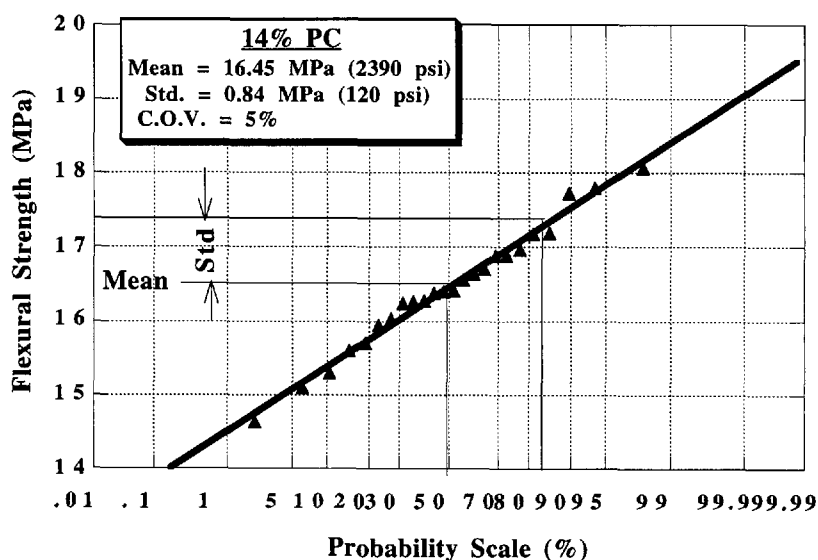


Fig. 3—Variation of flexural strength of 14 wt percent PC on a normal probability plot

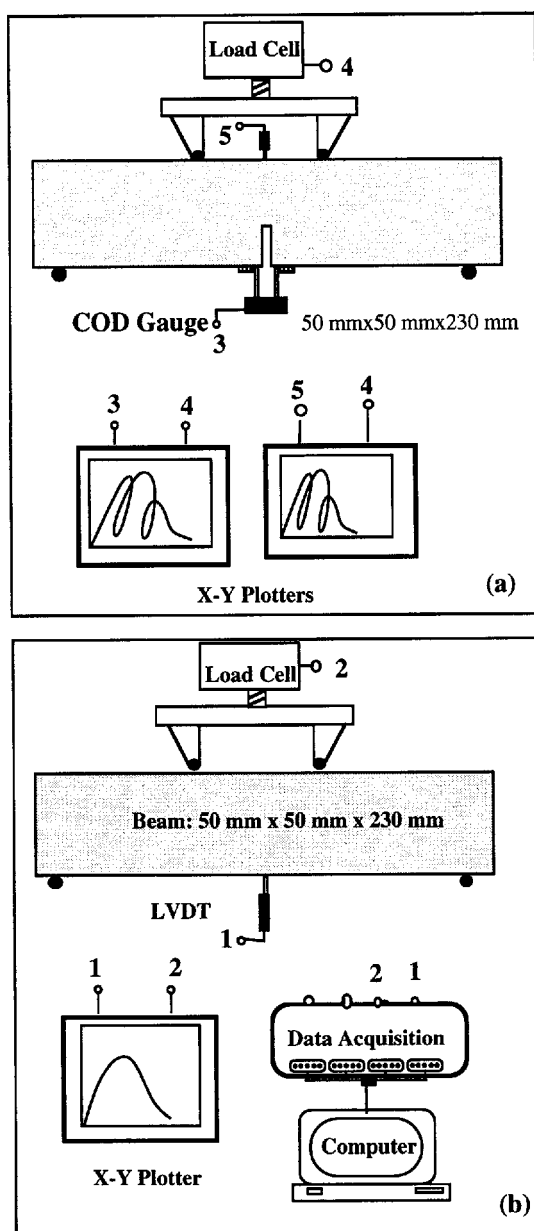


Fig. 2—Testing configurations for flexure and fracture tests

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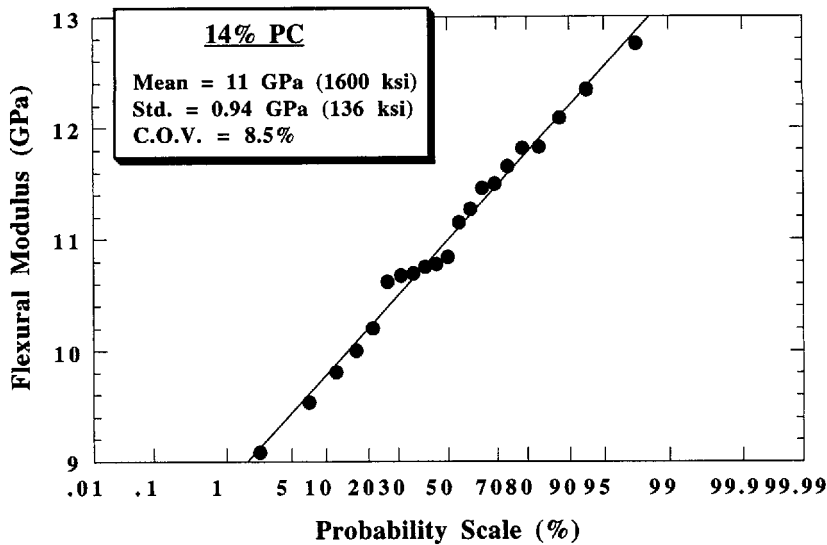


Fig. 4—Variation of flexural modulus of 14 wt percent PC on a normal probability plot

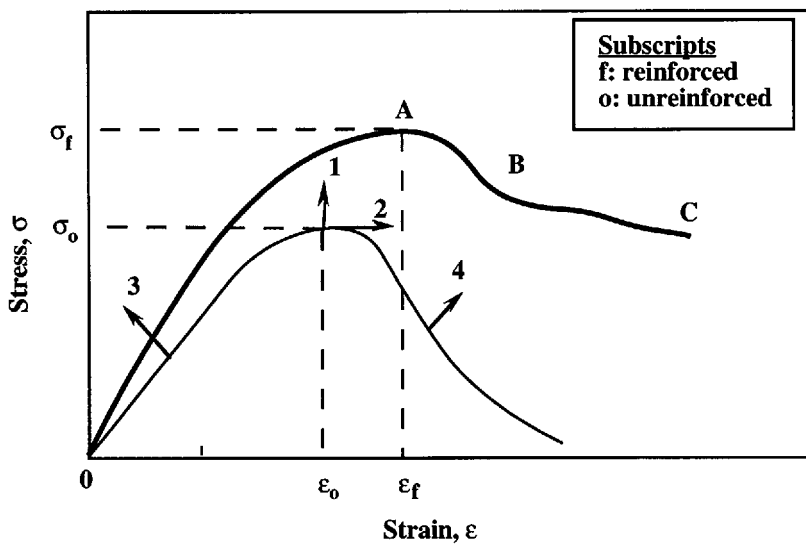


Fig. 5—Typical changes in stress-strain relationships for improved PC systems

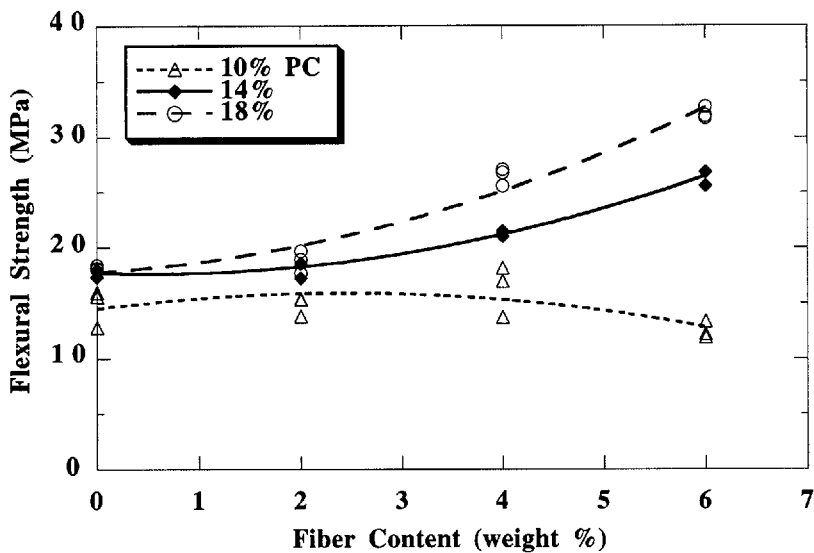


Fig. 6—Effect of glass fibers on the flexural strength of various PC systems

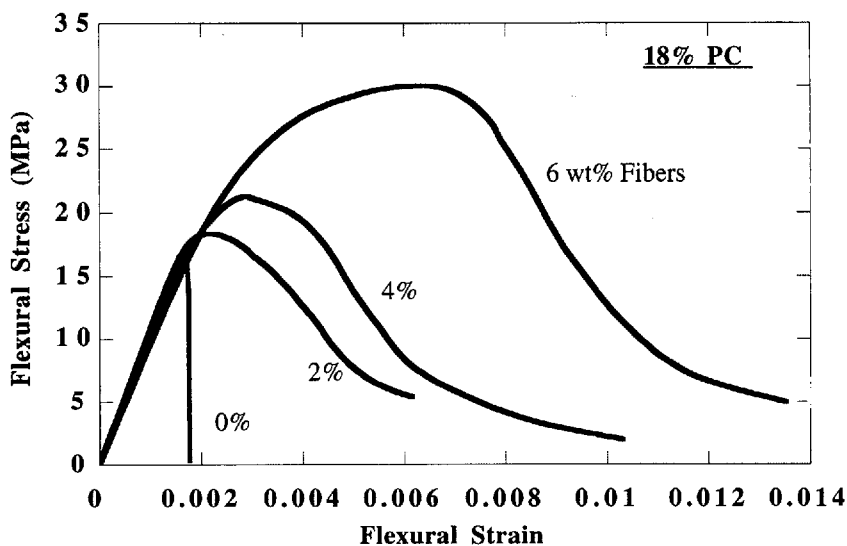


Fig. 7—Effect of glass fibers on flexural stress-strain relationships of 18 wt percent PC

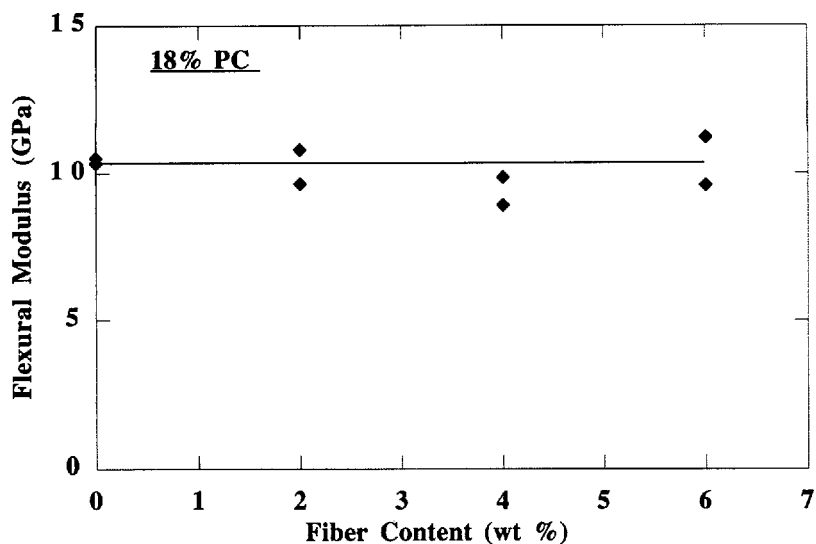


Fig. 8—Flexural modulus of 18 wt percent PC with various fiber contents

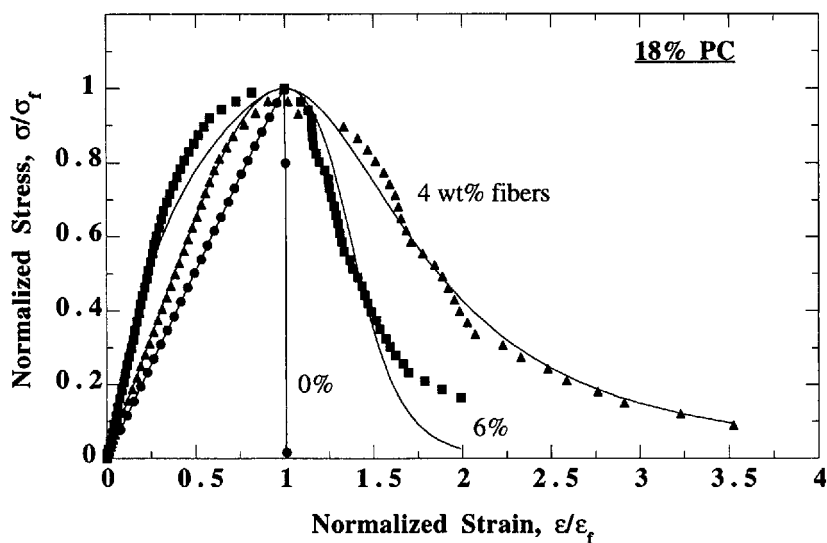


Fig. 9—Predicted (solid line) and measured stress-strain relationships for PC with various fiber contents

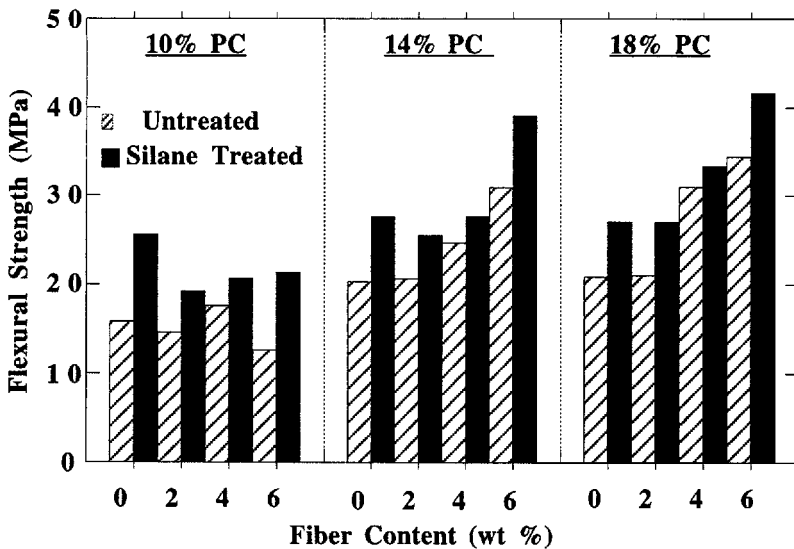


Fig. 10—Flexural strength of glass fiber reinforced PC systems with a silane coupling agent

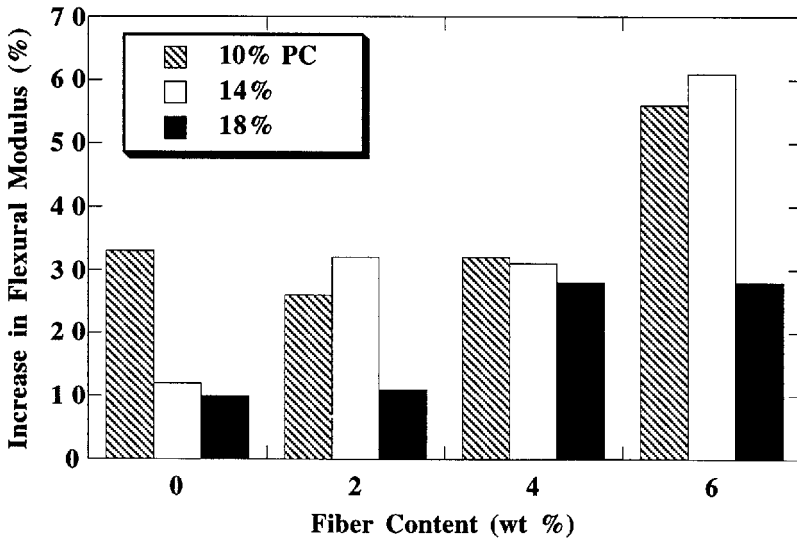


Fig. 11—Improvement in flexural modulus of glass fiber reinforced PC systems with a silane coupling agent

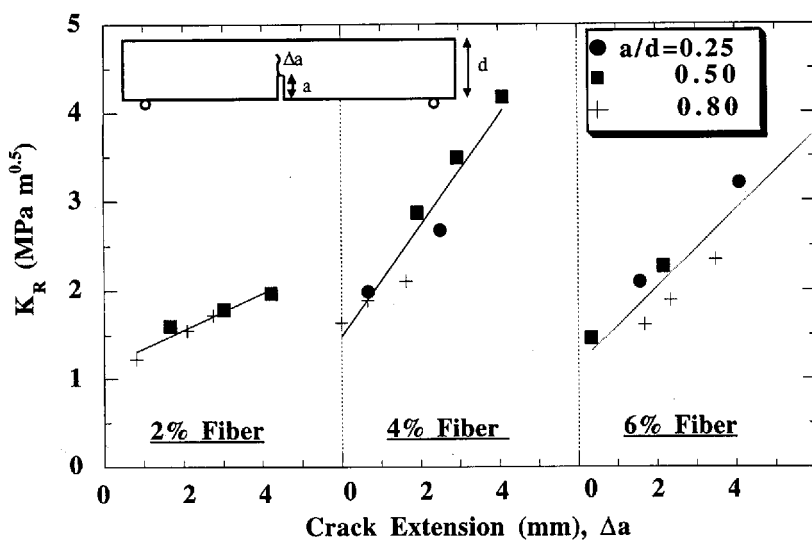


Fig. 12—Fracture resistance relationships for 18 wt percent PC with silane-treated sand and glass fibers

SP 166-2

Performance of Steel-Polymer Concrete Composite Structural Elements

by C. Vipulanandan, S. T. Mau,
Syam K. Mantrala, and S. Wei

There is an interest in developing better performing (high strength and ductility) composite structural elements for construction and repair of onshore and offshore structures. In this study, composite structural elements that consist of filled columns and sandwich columns (two concentric circular steel tubes with polymer concrete sandwiched in-between) were investigated as potential compression members. High strength (480 MPa) and low strength (200 MPa) steel tubes conforming to ASTM A513 type 5 and ASTM A500 Grade B, respectively, were used. The polymer concrete was polyester based with a compressive strength of 60 MPa.

Short composite columns, made of steel tubes of diameter-to-thickness ratio ranging from 16-to-170, were tested under monotonically increasing axial compression. It was observed that the composite columns had compressive strengths of 10-to-30% higher than that of the summation of the individual components. The ductility was much higher than that of the corresponding steel tubes.

Relationships for predicting the initial modulus, and peak load and corresponding strain of the sandwich column have been developed. A simple model was used to predict the load-strain history up to the peak load of the composite elements. The predictions agreed well with the test results.

Keywords: Compression; ductility; models; modulus of elasticity; polymer concrete; sandwich structures; steel construction; strains; strength; tubes

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INTRODUCTION

Concrete filled tubular members (1, 2) or composite members have been used in many engineering applications due to several advantages over the conventional concrete or steel members. Performance of thin structural steel tubes under compression is often limited by stability considerations such as local buckling (3). Filling the tubes with concrete was found to be effective in preventing the premature failure of the tubes due to local buckling. It is also known that portland cement concrete under confinement can withstand higher stresses and strains. Due to these effects the load carrying capacity of the composite member is generally expected to exceed the summation of the load carrying capacities of the steel and concrete when loaded separately. Also the ductility (strain at peak load) of the composite members was high compared to the steel and reinforced concrete.

Polymer Concrete (PC) is a high strength material with rapid setting and good bonding properties compared to cement concrete (4-10). Also compressive properties of PC can be substantially improved if confined (11). But the creep behavior of PC is a concern in accepting it as a potential construction material. By effectively combining PC with steel tubes, a composite can be developed to overcome the shortcomings in steel tubes and PC and to take advantage of their interaction. Since polymer concrete is relatively expensive compared to portland cement concrete, the idea of filling the tube with polymer concrete may not be economically feasible. This has led to the development of a new configuration for composite structural element, which consists of two concentric circular steel tubes with polymer concrete sandwiched in-between. In this study, the mechanical behavior of this new composite was investigated and compared to filled tubes.

RESEARCH SIGNIFICANCE

Present construction industry needs high performance (high strength and ductility) structural elements to meet the requirements of high rise, long span, heavily loaded, onshore and offshore structures, and also to resist seismic loading. This study investigated the behavior of two new steel-polymer concrete composite columns as compression members. Test results show that the steel-polymer concrete composite column has the potential of being developed into a high performance structural element.

MATERIALS AND SPECIMEN PREPARATION

Steel Tubes

High strength steel (480 MPa; mechanical) and low strength steel (200 MPa; structural) tubing conforming to ASTM A513 type 5 and ASTM A500 Grade B, respectively, were used in this investigation. For the tubes selected, the outside diameter-to-thickness ratio (D/t) varied between 36 and 52 for high strength tubes and between 16 and 70 for low strength tubes. The virgin tubes were machined from outside to extend the range of D/t from 16-to-173 with the outside diameter varying from 55-to-114 mm. The height of the tubes tested in compression varied between 191 and 267 mm.

Polymer Concrete

Polyester-based polymer concrete (PC) was used in this study. The composition of the polymer concrete is summarized in Table 1. Earlier studies had shown that PC with 14% polymer had the highest strength and modulus (9, 10). Commercially available well-graded blasting sand with a coefficient of uniformity (C_u) of 5.8 and coefficient of concavity (C_c) of 0.9 was used. In preparing polymer concrete, cobalt naphthenate was first added to the polyester resin, and the solution was mixed for at least two minutes before adding MEKP (methyl ethyl ketone peroxide). After further mixing, the sand was slowly added to the polymer and mixed long enough to obtain a uniform mix.

Cylindrical specimens with a length-to-diameter ratio of 3:1 (length = 160 to 190 mm and diameter = 60 to 65 mm) were used to characterize the polymer concrete. Specimens were cast in steel molds. The inner surface of the mold was greased in order to facilitate the easy removal of the specimens. Polymer concrete was then placed in the mold and compacted in three layers. Specimens were cured at room temperature (about 20°C) for 24 hours followed by 24 hours in an oven at 80°C (5, 6). They were capped with sulfur compound before testing.

Composite Specimens

Composite specimens were made by placing the polymer concrete in the annular space between the two concentric steel tubes in the case of the sandwich configuration (Fig. 1) and inside the tube in the case of filled tube, which by themselves served as molds for polymer concrete. The curing conditions were the same as those adopted for the PC cylinders.

TEST PROCEDURE

All the specimens were loaded in compression using a 400 kip (1.8 MN) capacity Tinius Olsen Universal Testing Machine at a strain rate of 0.0002 in/in/min. Two axial and four lateral strain gages (gage length of 10 mm) were used to measure the axial and lateral (circumferential) strains at the middle of the steel tubes and PC specimens, respectively. In the case of sandwich specimens two axial and four lateral strain gages were glued to both inside and outside tubes. An LVDT (Linear Variable Differential Transformer) and a dial gage were also used to measure the total axial deformation of the specimens. End supporting devices were used for steel tubes and composite specimens to prevent premature failure at the ends. The data was collected using a 486 personal computer. The test setup is shown in Fig. 2, which is essentially the same for all the specimens except for polymer concrete specimens, in which case the end supporting devices were replaced by end bearing blocks.

TEST RESULTS AND DISCUSSION

Steel Tubes

Typical stress-strain relationships of high strength and low strength steel tubes of various diameter-to-thickness ratios (D/t) are shown in Fig. 3. The initial modulus of elasticity, 0.2 % offset yield stress and the corresponding strain for high strength tubes varied from 204-to-221 GPa, 410-to-531 MPa, and 0.40-to-0.45%, respectively, whereas the low strength tubes varied from 169-to-211 GPa, 216-to-302 MPa, and 0.30-to-0.34%, respectively. In Fig. 3 the typical behavior of high strength tubes with D/t of 36 (unmachined) and 64 (machined) are compared. All the unmachined tubes showed plastic buckling, whereas most of the machined tubes buckled well before yielding due to their high D/t values. The buckling strain (strain at peak stress) is reduced from 1.5% to 0.4% due to the increase in D/t ratio. Also the behavior of a low strength steel tube with a D/t of 57 (unmachined) is shown in Fig. 3, which also shows plastic buckling.

Polymer Concrete

The average density and the corresponding coefficient of variation (COV) of fourteen polymer concrete cylinders were 20.4 kN/m^3 and 0.9%, respectively. Typical stress-strain relationship and an average stress-strain relationship are shown in Fig. 4. The average stress-strain relationship was obtained by averaging the stresses of all the specimens at selected strains. The average compressive strength was 58.6 MPa with a COV of 5.4%. The average initial modulus was 18.2 GPa with a COV of 6.2% and the average initial Poisson's ratio was 0.20 with a COV of 3.7%. The average failure strain (strain at the peak load) was 0.75 % with a COV of 12%.

Composite Columns

Filled Column--In order to understand the composite behavior of steel tube and PC, a few filled tube columns were investigated initially. Low strength steel tubes with D/t of 16 and 43 were filled with PC and tested under axial compression. The dimensions are summarized in Table 2. Typical load-strain relationship together with the corresponding component and summation curves is shown in Fig. 5a. The summation curve was obtained by adding the load-strain relationships of the two components, namely the outside tube (OT), and the polymer concrete (PC), at the same strain. In the case of PC the average stress-strain curve (Fig. 4) was used. From Fig. 5a it can be seen that PC filled column behaves in a ductile manner. Even at an axial strain of 3%, the load carrying capacity of the column showed an increasing trend. Also the improvement in the load carrying capacity was about 30% (Table 3) when compared to the summation curve. This shows the potential of steel-polymer concrete composite being used as a compression member. To make this composite less expensive it is necessary to reduce the quantity of PC for a given configuration. This has lead to the development of new steel-polymer concrete-steel sandwich composite, and its behavior is discussed in the following section.

Sandwich Column--Typical load-axial strain curve of a low strength steel-PC sandwich specimen together with the corresponding component and summation curve is shown in Fig. 5b. The summation curve was obtained by adding the load-strain curves of the three components, namely the outside tube (OT), the polymer concrete (PC), and the inside tube (IT), at the same strain. It can be observed from the figure that the peak strain of any of the components was not greater than 0.75%, where as that of the sandwich column was about 1.6%. This showed that the interaction between steel tubes and polymer concrete was responsible for the improved ductility of the composite. About 10% improvement in the load carrying capacity and 200% improvement in the ductility were observed when compared to the summation curve.

Typical dimensions and results of high strength steel-PC sandwich specimens (HSW) are summarized in Tables 2 and 3 respectively. Also the behavior of specimen D1-1 is compared to the summation load-strain relationship in Fig. 5c. Specimens D1-1 and D6-1 were of almost the same area ratio (area of PC/area of steel), but the D/t ratios of the inside and out side tubes were smaller for the later. This had resulted in almost doubling of the peak strain, whereas the

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percentage improvement in the load carrying capacity was same (19%) for both when compared to the summation.

Even though the D/t ratio values of the steel tubes used in specimen A3-1 were smaller than those of D6-1, its peak strain was much smaller and improvement in load carrying capacity was 13%. This could be partly attributed to the very small area ratio in the case of A3-1. This showed that the small quantity of PC was unable to provide enough support to the steel tubes. The buckling strains of the outside and inside steel tubes used in this configuration, when loaded separately, were 1% and 0.4% respectively, and the peak strain of the polymer concrete was 0.75%. The peak strain of the sandwich specimen A3-1 was 1%, which showed that there was not enough support from the polymer concrete to the steel tubes. This can be substantiated by the failure patterns given in Fig. 6a. The failure pattern of A3-1 was almost axisymmetric in shape similar to that of the corresponding empty steel tube showing the domination of the steel in the composite. This also indicates the inadequacy of the amount of PC for composite action in this configuration.

The buckling strains of the outside and inside steel tubes used in sandwich specimen D6-1, when loaded separately, were 1.1% and 0.4% respectively, and the peak strain of the polymer concrete was 0.75%. The peak strain of this sandwich specimen D6-1 was 1.8%. This showed that there was good interaction between steel tubes and polymer concrete which in turn resulted in a peak strain of the composite much higher than that of the individual components. Evidently the failure pattern of D6-1 was close to shear type, similar to that of the polymer concrete, showing that there was enough support for the steel tube from the polymer concrete (Fig. 6b).

Typical dimensions and results of low strength steel-PC sandwich columns (LSW) are summarized in Tables 2 and 3, respectively. The outside steel tube was the same for the specimens H1-1 and J1-1, whereas the D/t of the inside tube was same for both. This configuration was used to study the effect of the area ratio. The area ratio of J1-1 was reduced to less than half that of H1-1. This had resulted in further 5% increase in the improvement of load carrying capacity when compared the summation, whereas it had no significant effect on the peak strain of the composite.

Comparing specimens E1-1 and H1-1, both the specimens had the same inside tube where as the D/t of the outside tube was smaller for the former. The area ratio of E1-1 was about half that of H1-1. This had resulted in further 3% increase in the improvement of load carrying capacity when compared the summation, whereas the peak strain increased by more than 20%. This showed that the D/t of the outside tube had significant effect on the peak strain of the composite. The failure patterns of these sandwich specimens were similar to that of the polymer concrete, either shear type or hour glass type.

The sandwich columns had compressive strength of 10-to-30% higher than that of the summation of the individual components. The peak strain was 100-to-200% more than that of the summation curve, showing the high ductility of the composite due to the interaction between steel tubes and polymer concrete.

Observation of the sandwich specimen by the naked eye during the test did not show any buckling of the outside tube before the peak point. Since the inside tube could not be seen during the test, the strain gage readings of the axial

and the circumferential gages were examined for the buckling. Buckling in the inelastic range can be identified by an abrupt change in strain gage readings. Only in a few cases were this kind of changes observed before the peak point, though there was no effect on the overall load-strain curve of the sandwich specimen. This suggests that the inner tube buckling was restrained by the polymer concrete and the load carrying capacity of the inner tube did not drop substantially. In all other cases the inner tube buckled near the peak. So the drop in load carrying capacity of the sandwich specimen after the peak is possibly due to the initiation of the failure in polymer concrete, which further initiated the buckling of the inner and outer tubes. Since there are many parameters involved in the sandwich configuration, it is difficult to study the effect of the individual parameter on the behavior of the sandwich specimen. Based on the observations from the tests, an attempt was made to combine the most important parameters including the grade of steel to relate the peak strain of the sandwich composite to the constituents.

COMPOSITE MODELS

The performance of the filled and sandwich tubes can be represented by the following simple relationships.

Ultimate Load

It is assumed that all the components in the composite reach their maximum strength together, hence the ultimate load (N_p) of the composite column is given by

$$N_p = A_c f_c + \sum A_s f_y, \quad (1)$$

where A_c = area of the polymer concrete,
 f_c = uniaxial compressive strength of polymer concrete,
 A_s = area of the steel tube, and
 f_y = yield strength of the steel tube.

In the case of a sandwich column the inside tube and the outside tube strengths are added to the contribution of steel. Test loads are compared with the predicted loads in Table 3. The predictions agreed well with the test results. The average value of the ratio of measured peak loads to predicted peak loads (N_o/N_p) was 1.09 with a COV of 5%.

Peak Strain

It was observed from the tests that the main parameters controlling the peak strain were the ratio of inside diameter (D_{oi}) to the thickness (t_o), (D_{oi}/t_o) and f_y/E of the outside steel tube and the ratio of the area of polymer concrete to the area of outside steel tube (A_c/A_o). A parameter (p) which is a function of all

the above mentioned parameters was found to be related to the peak strain (ϵ_p) as shown in the Fig. 7. The relationship is given by

$$\epsilon_p = \epsilon_c + \frac{p^2}{11.5 + 0.75p^2}, \quad (2)$$

$$\text{where } p = 4 \left(\frac{A_c}{A_o} \right) \left(\frac{t_o}{D_{oi}} \right)^2 \left(\frac{E}{f_y} \right).$$

ϵ_c is the peak strain (0.75%) of the polymer concrete under uniaxial compression, and the strains are expressed in percentage. The value of the parameter (p) is equal to zero for PC only, varied between 0.75 and 7.5 for the sandwich specimens, and 45 and 16 for the two filled tubes tested. The strains predicted using Eq. 2 are compared with the test results in Table 3. The average value of the ratio of measured peak strains to predicted peak strains (ϵ_o/ϵ_p) was 1.01 with a COV of 11%.

Initial Modulus

The initial modulus or the equivalent Young's modulus of the composite can be estimated by using the parallel composite model and is given by

$$E_p = \frac{E_c A_c + \Sigma E_s A_s}{A_c + \Sigma A_s}, \quad (3)$$

where E_c , E_s , and A_c , A_s , are the initial young's moduli and the cross-sectional areas of polymer concrete and steel respectively. The average value of the ratio of measured to predicted initial moduli (E_o/E_p), by using Eq. 3, was 0.91 with a COV of 10%.

Load-Strain Relationship up to the Peak Load

The experimental load-strain relationships of the sandwich specimens up to the peak were predicted by using a simple Ramberg-Osgood type model (12-17), in which the two parameters were the normalized initial slope (S_n) and the curvature parameter (m). This relationship is given by

$$\frac{N}{N_o} = \frac{S_n \left(\frac{\epsilon}{\epsilon_o} \right)}{\left[1 + \left\{ S_n \left(\frac{\epsilon}{\epsilon_o} \right) \right\}^m \right]^{\frac{1}{m}}}, \quad (4)$$

where N/N_o = normalized load,
 ϵ/ϵ_o = normalized strain,
 S_n = initial slope of the normalized load-strain curve, and

m = curvature parameter.

S_n is defined as the ratio of the initial slope to the secant slope at the peak (N_o/ϵ_o) of the total load-strain curve. The curvature parameter (m) was obtained by the least square fit from the experimental normalized load-strain data. There is a relation between the two parameters as shown in Fig. 8 (16, 17). Knowing the normalized slope (S_n), the curvature parameter (m) can be calculated from the relationship given by

$$m = \frac{5}{\sqrt{S_n}} \quad (5)$$

Typical normalized and total load-strain relationships predicted using Eqs. 4 and 5 are compared with the corresponding test results in Figs. 9 and 10. The predictions agreed well with the test results up to the peak load.

CONCLUSIONS

Behavior of PC filled column and sandwich column was studied under uniaxial compression. Based on the test results the following conclusions can be drawn.

- (1) The filled composite column had a compressive strength of 5-to-30% higher than that of the summation of the individual components. As high as 3% ductility (strain at peak load) was achieved with this configuration.
- (2) The sandwich composite column had a compressive strength of 10-to-30% higher than that of the summation of the individual components. A 100-to-200% increase in ductility was observed when compared to the summation curve.
- (3) The proposed composite models were able to predict the peak load, peak strain, and initial modulus with a good degree of accuracy.
- (4) The two parameter model was able to predict the load-strain history of the composite column up to the peak.

ACKNOWLEDGMENT

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REFERENCES

1. Gardner, N. J., and Jacobson, E. R., "Structural Behavior of Concrete Filled Steel Tubes," *Proceedings, American Concrete Institute*, V. 64, No. 7, July 1967, pp. 404-413.
2. Knowles, R. B., and Park, R., "Strength of Concrete Filled Steel Tubular Columns," *Journal of the Structural Division*, ASCE, V. 95, No. ST12, Dec. 1969, pp. 2565-2587.
3. Batterman, S. C., "Plastic Buckling of Axially Compressed Cylindrical Shells," *AIAA Journal*, V. 3, No. 2, 1965, pp. 316-325.
4. "Guide to the Use of Polymers in Concrete," *ACI Journal*, V. 83, No. 5, 1986, pp. 798-829.
5. Vipulanandan, C., and Paul, E., "Performance of Epoxy and Polyester polymer Concrete," *ACI Materials Journal*, V. 87, No. 3, May-June 1990, pp. 241-251.
6. Vipulanandan, C., and Paul, E., "Characterization of Polyester Polymer and Polymer Concrete," *Journal of Materials in Civil Engineering*, V. 5, No. 1, Feb. 1993, pp. 62-82.
7. Fowler, D., "Future Trends in Polymer Concrete," *SP 116*, American Concrete Institute, Detroit, Mich., 1990, pp. 129-143.
8. Ohama, Y., "Mix Proportions and Properties of Polyester Resin Concretes," *Polymers in Concrete*, *SP 40*, American Concrete Institute, 1973, pp. 183-294.
9. Vipulanandan, C., Dharmarajan, N., and Ching, E., "Mechanical Behavior of Polymer Concrete Systems," *Materials and Structures*, V. 21, 1988, pp. 268-277.
10. Mebarkia, S., and Vipulanandan, C., "Compressive Behavior of Glass-Fiber-Reinforced Polymer Concrete," *Journal of Materials in Civil Engineering*, V. 4, No. 1, Feb. 1992, pp. 91-105.
11. Wei, S., Mau, S. T., and Vipulanandan, C., "Mechanical Behavior of Confined Polymer Concrete," *Cement and Concrete Research*, V. 22, 1992, pp. 621-630.
12. Ramberg, W., and Osgood, W. R., "Description of Stress-Strain Curves by Three Parameters," *Technical Note No. 902*, National Advisory Committee for Aeronautics, Washington, D. C., 1943.
13. Richard, R. M., and Abbott, B. J., "Versatile Elastic-plastic Stress-Strain Formula," Technical Notes, *Journal of the Engineering Mechanics Division*, ASCE, V. 101, No. EM4, Aug. 1975, pp. 511-515.

14. Desai, C. S., and Wu, T. H., "A General Function for Stress-Strain Curves," *Proceedings, ASCE, 2nd International Conference on Numerical Methods in Geomechanics*, Blacksburg, Virginia, June 1976, pp. 306-315.
15. Desai, C. S., "Constitutive Laws for Engineering Materials with Emphasis on Geologic Materials," *Prentice-Hall, Inc.*, Englewood Cliffs, NJ, 1984.
16. Rao, B. N., "A Simplified Procedure for Stress Analysis in the Elasto-Plastic Range with the Aid of Strain Gages," *Experimental Techniques*, May/June 1991, pp.38-39.
17. Schilling, C. G., "Buckling Strength of Circular Tubes," *Journal of the Structural Division*, ASCE, V. 91, No. ST5, Oct. 1965, pp. 325-348.

TABLE 1 — COMPOSITION OF POLYMER CONCRETE

Constituent material	Relative weight %
Matrix-Polyester Resin	14
Initiator-MEKPO	1.5*
Promoter-Cobalt Napthenate	0.3*
Aggregate-Blasting Sand	86

MEKPO-Methyl Ethyl Ketone Peroxide

*by weight of resin

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TABLE 2 — TYPICAL DIMENSIONS AND PROPERTIES OF STEEL TUBES USED IN SANDWICH SPECIMENS

ID	D _i mm	t _i mm	(D/t) _i	D _o mm	t _o mm	(D/t) _o	t _c mm	A _i mm ²	A _o mm ²	A _c mm ²	A _c /A _s	(f _y /E) _i MPa/GPa	(f _y /E) _o MPa/GPa
Filled Tubes (FT)													
1	—	—	—	60.2	3.84	16	—	—	680	2171	3.19	—	302/201
2	—	—	—	55.1	1.27	43	—	—	215	2171	10.10	—	302/201
Sandwich Columns Made of High Strength Steel Tubes (HSW)													
A3-1	62.0	1.00	62	76.3	1.78	43	5.38	191	417	1139	1.87	470/212	486/211
D1-1	80.3	0.55	146	99.7	0.59	169	9.11	138	183	2555	7.96	474/214	409/193
D6-1	61.5	0.56	118	101.7	1.61	63	18.49	106	505	4645	7.60	432/188	409/193
Sandwich Columns Made of Low Strength Steel Tubes (LSW)													
E1-1	63.5	1.16	55	88.8	1.55	57	11.14	227	425	2842	4.36	216/197	286/209
H1-1	63.5	1.12	57	114.3	1.64	70	23.72	223	580	6501	8.10	216/197	262/195
J1-1	88.9	1.56	57	114.3	1.64	70	11.07	428	580	3474	3.45	286/209	262/195

D - outside diameter, t - thickness, A - area, f_y - 0.2% offset stress, and E - initial modulus.

Subscripts: i - inside tube, o - outside tube, c - concrete, s - total steel

TABLE 3 — COMPARISON BETWEEN TEST RESULTS AND PREDICTIONS

ID	N _o kN	N _o /N _s	N _o /N _p	ε _o (%)	ε _o /ε _s	ε _o /ε _p	E _o GPa	E _o /E _p
Filled Tubes (FT)								
1	408	1.28	1.23	—	—	—	21.8	1.20
2	202	1.06	1.05	2.33	3.43	1.14	65.5	1.06
Sandwich Columns Made of High Strength Steel Tubes (HSW)								
A3-1	395	1.13	1.10	0.99	2.47	0.82	91.0	1.10
D1-1	283	1.19	0.98	0.91	3.25	1.11	32.1	0.83
D6-1	644	1.19	1.23	1.80	1.80	1.18	28.3	0.74
Sandwich Columns Made of Low Strength Steel Tubes (LSW)								
E1-1	357	1.12	1.06	1.94	3.23	1.13	49.4	0.93
H1-1	598	1.09	1.03	1.57	3.02	0.88	33.7	0.89
J1-1	524	1.14	1.10	1.58	3.04	1.12	60.5	1.02

N - load, ε - strain, E - initial modulus.

Subscripts: o - test, s - summation, p - predicted.

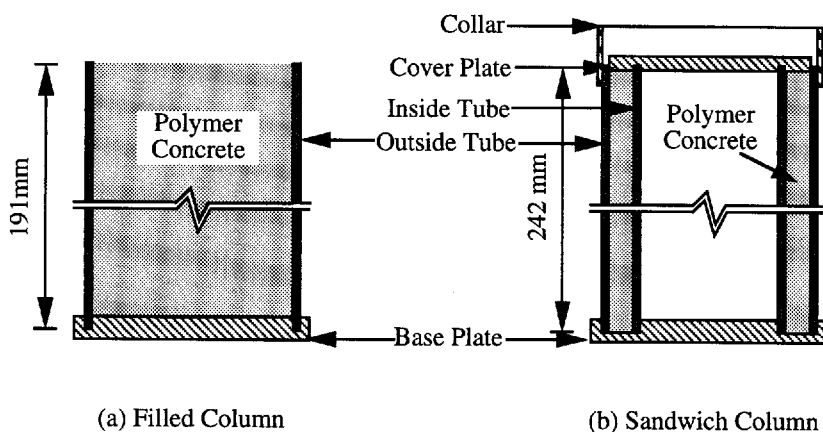


Fig. 1—Casting of composite specimen

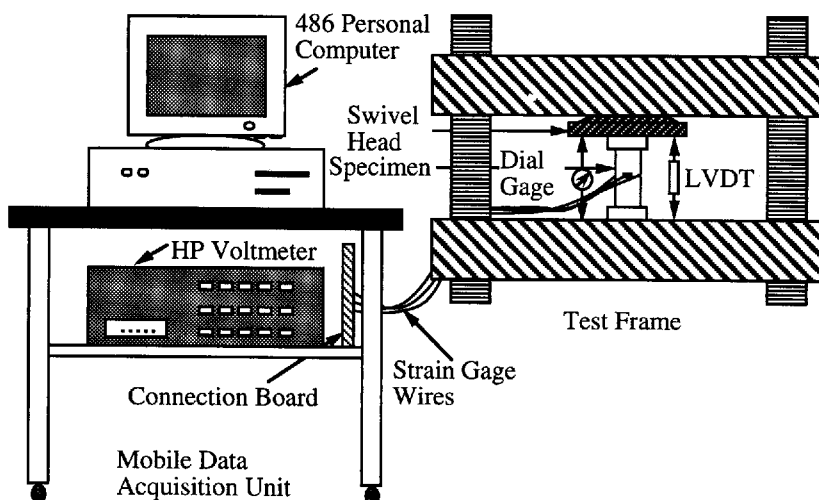


Fig. 2—Testing setup

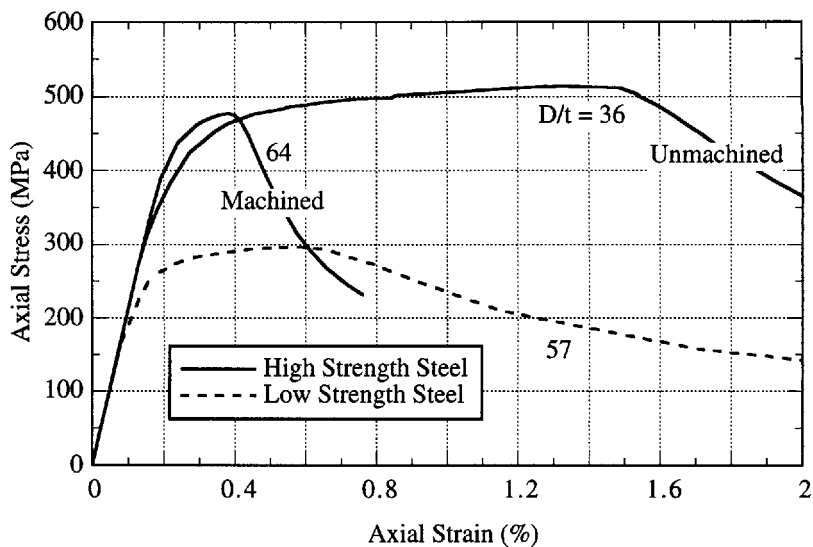


Fig. 3—Typical stress-strain relationships for low- and high-strength steel tubes under axial compression

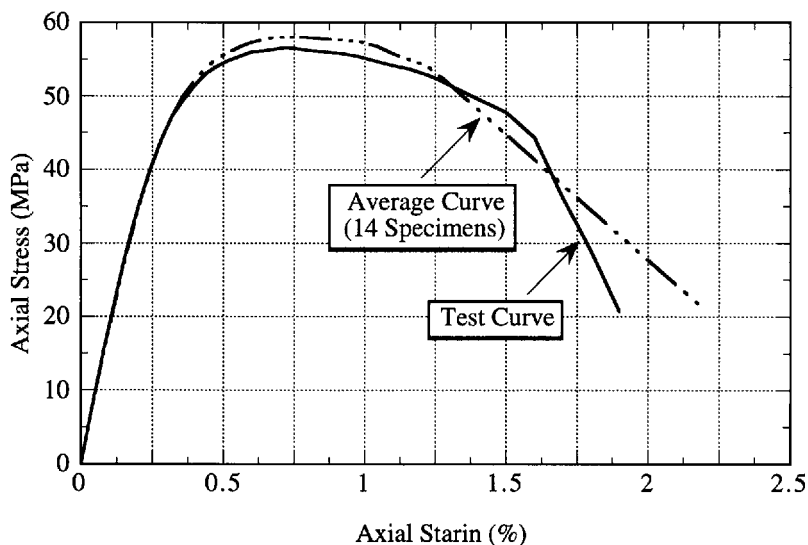


Fig. 4—Typical stress-strain relationship for polymer concrete along with an average curve (14 specimens)

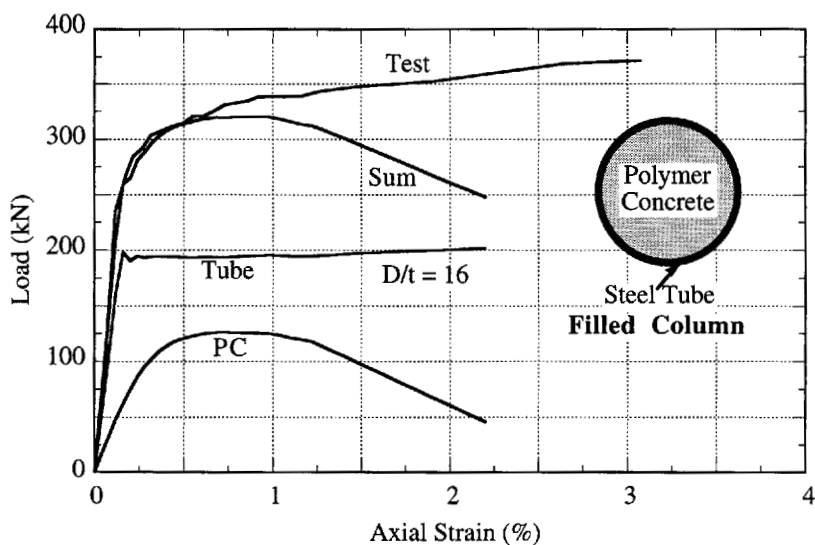


Fig. 5a—Test, summation, and component load-strain relationships for filled column (FT 1)

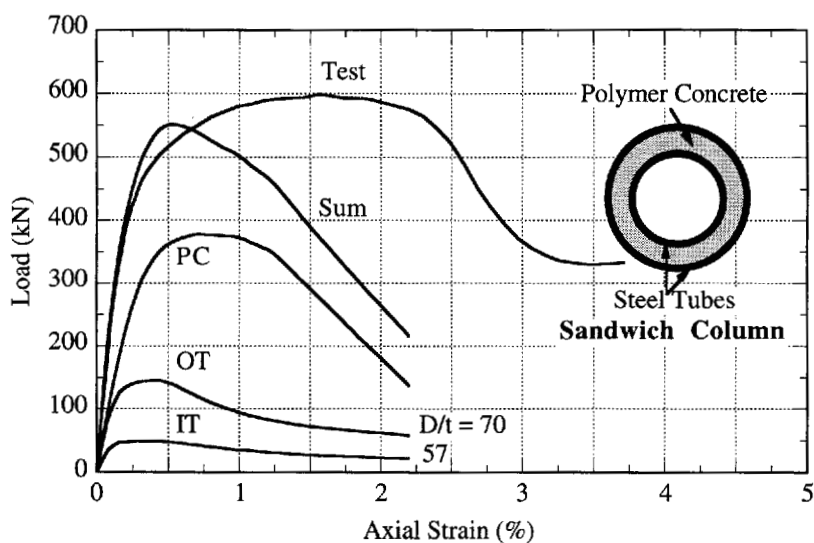


Fig. 5b—Test, summation, and component load-strain relationships for sandwich column (LSW H1-1)

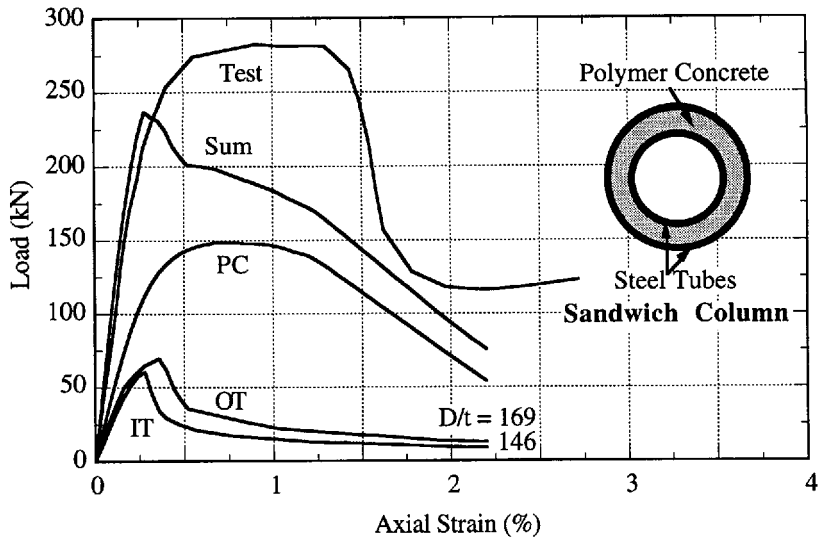


Fig. 5c—Test, summation, and component load-strain relationships for sandwich column (HSW D1-1)

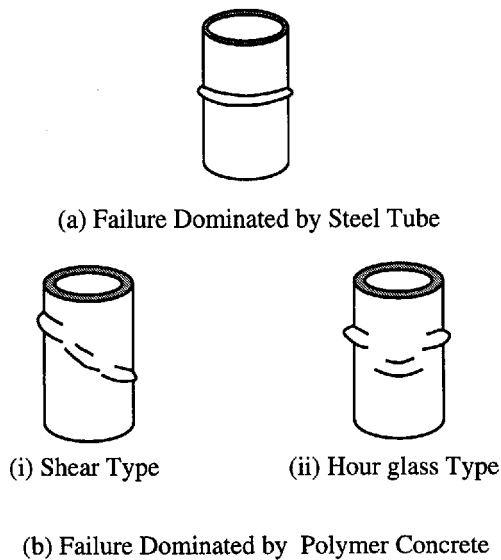


Fig. 6—Typical failure patterns of sandwich specimens: a) failure dominated by steel tube; b) failure dominated by polymer concrete; i) shear type; ii) hour glass type

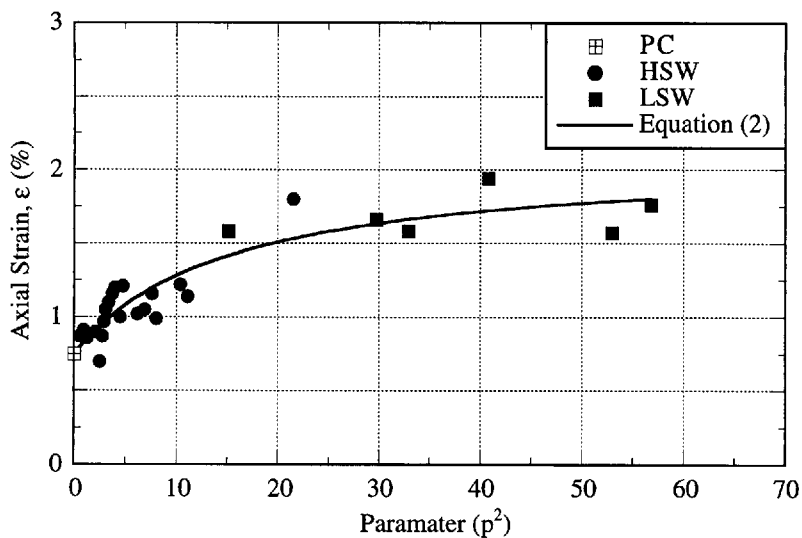


Fig. 7—Axial strain as function of parameter (p) for sandwich column

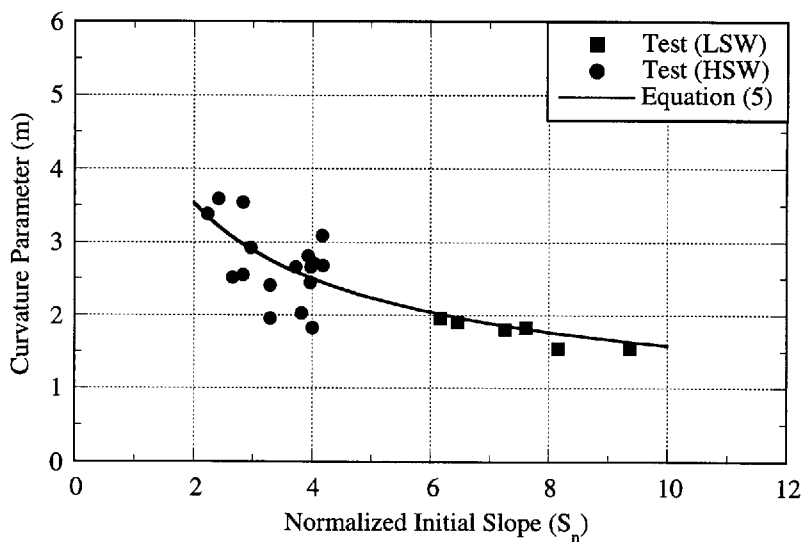


Fig. 8—Curvature parameter as function of normalized initial slope of load-strain relationship

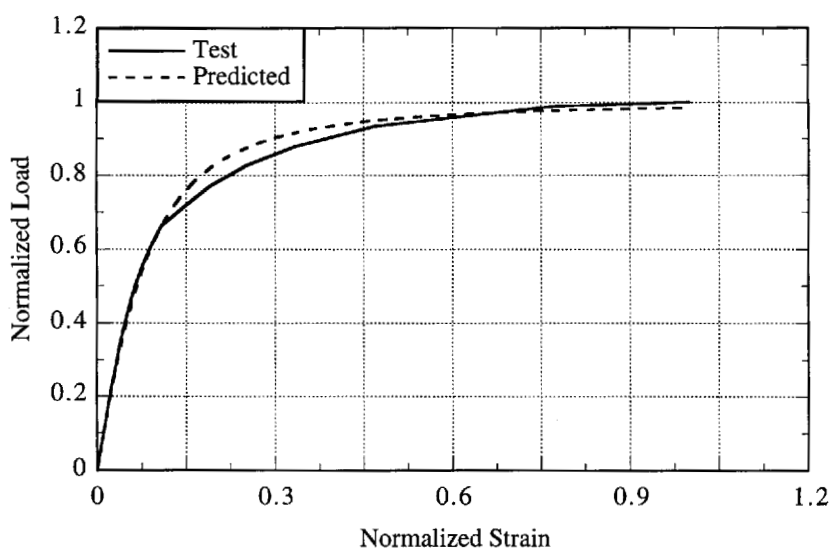


Fig. 9a—Comparison of test and predicted normalized load-strain relationships for sandwich specimen (LSW E1-1)

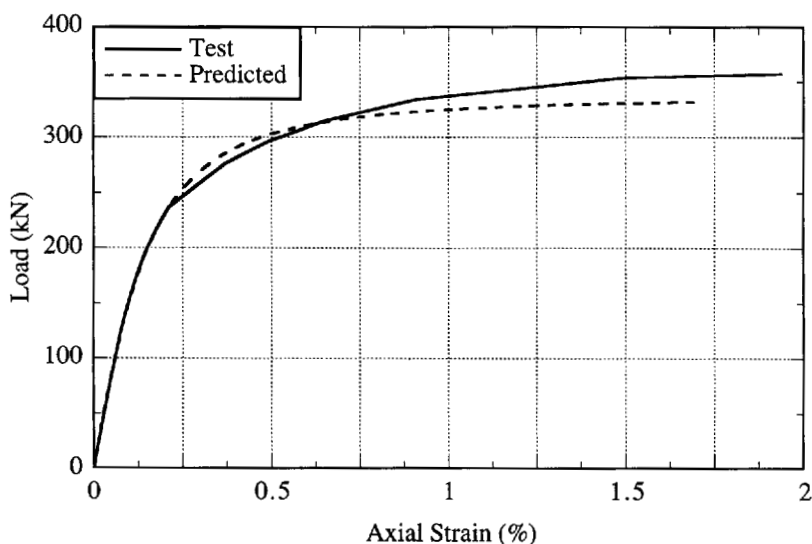


Fig. 9b—Comparison of test and predicted load-strain relationships for sandwich specimen (LSW E1-1)

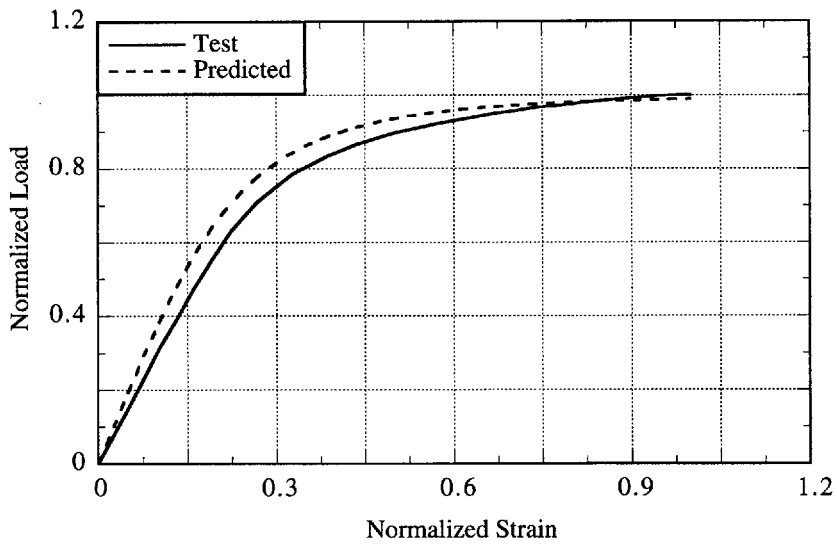


Fig. 10a—Comparison of test and predicted normalized load-strain relationships for sandwich specimen (HSW A1-1)

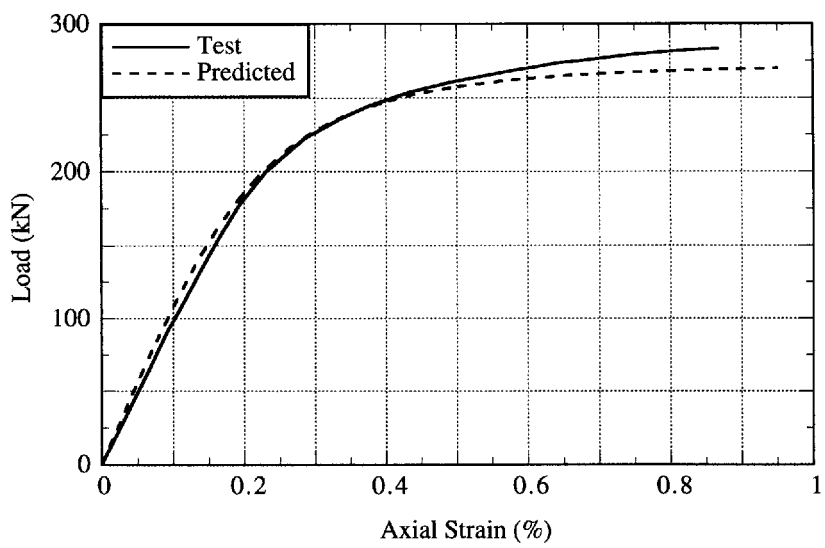


Fig. 10b—Comparison of test and predicted load-strain relationships for sandwich specimen (HSW A1-1)

SP 166-3

Measurement of Shrinkage-Induced Stress in Polymer Concrete Overlays

by Jamal-Aldin H. Zalatimo
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Synopsis: Shrinkage is a form of dimensional change which, if restrained, can produce stresses similar to those caused by the contraction of a material subjected to a temperature drop. However, a significant portion of total shrinkage takes place during the first few hours after mixing when the polymer concrete mix is still viscous. In addition, shrinkage is typically a one-time occurrence with effects extending over a long period of time. The significance of this difference is associated with a property known as stress relaxation. Research eventually led to the development of a test method for determining shrinkage-induced stresses in overlays. The basic idea behind this method is to accumulate shrinkage-induced stresses in a restrained polymer concrete overlay, to remove the restraint, and to measure the total released strain. To perform the proposed test, the middle region of a portland cement concrete beam is covered with several layers of plastic sheets that act as a bond breaker. Once overlay placement is complete, a DuPont device is positioned within the limits of the unbonded central region. Restraint provided by the substrate through the end regions is then removed by cutting the overlay transversely near one end of the unbonded central region. Test results indicated that shrinkage-induced stresses are not encountered with the use of slow curing systems such as the epoxy concrete considered in this study. As for systems with high unrestrained shrinkage, it was observed that a residual amount of shrinkage-induced stress was sustained. The stress, however, was much lower than the level indicated by the unrestrained shrinkage results.

Keywords: Deformation; epoxy resins; esters; polymer concrete; resurfacing; shrinkage; strains; stresses; stress relaxation; urethanes

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Introduction

Shrinkage is another form of dimensional change which, if restrained, can produce stresses similar to those caused by the contraction of a material subjected to a temperature drop. Portland cement concrete, for example, experiences both plastic and drying shrinkage. Polymer concrete experiences similar shrinkage effects, although due to different chemical or physical processes, following the initial expansion caused by heat build up to peak exothermic temperature. Any form of restraint transforms part or all of the shrinkage strain into stress. Many researchers consider shrinkage-induced stresses to be similar to thermally-induced stresses. This assumption facilitates the use of thermally-induced stress analyses in the solution of shrinkage type problems. For example, Choi (1) includes total strain diagrams that may be used to estimate stresses for a given dimensional change. Obviously, such stresses are superimposed on any existing stresses such as those originating from externally applied loads or thermal effects.

Despite the general similarities, certain differences have prevented shrinkage-related stresses from receiving the same level of attention and analysis as thermally induced stresses. The two major differences are:

1. Shrinkage strain can be determined by monitoring the dimensional change of a non-restrained sample by one of the methods developed for this purpose, an example being the DuPont method described in Fig. 1 (4). However, a significant portion of total shrinkage takes place during the few hours after mixing when the polymer concrete is transforming from a fresh plastic mix to a solid material. No shrinkage-induced stresses develop as long as the polymer binder is still viscous. Such stresses may only begin to develop when the restrained sample, or overlay, begins to develop strength. On the other hand, physical properties such as modulus of elasticity and Poisson's ratio that are necessary for the determination of the exact state of stress in the restrained material are also changing continuously. These two facts significantly complicate the analysis and make the computation of shrinkage-induced stress practically impossible. In comparison, thermally-induced stresses are associated with a cured material having well-defined physical properties.

2. Thermal changes that influence the stress state of an overlay are of two general types: long term, or seasonal, occurring gradually as seasons change, and short term, resulting from daily changes in ambient conditions and sudden climatic changes. In both cases they are of a cyclic nature. On the other hand, shrinkage-related volume changes are typically a one-time occurrence with effects extending over a long period of time. The significance of this difference is associated with a property known as stress relaxation, a manifestation of linear viscoelasticity observed in all polymers at strains below 5,000 $\mu\text{in./in.}$ (2). While the relaxation of stresses resulting from seasonal changes can be estimated relatively simply based on sustained load-deformation tests, the same is not true for the case of shrinkage due to the complicated interaction of shrinkage and changing properties described above.

Accordingly the determination of shrinkage-induced stresses in a bonded overlay from shrinkage information recorded for a companion non-restrained sample is very complicated. The exact state of stress cannot therefore be determined with sufficient certainty. This emphasizes the importance of developing a method by which shrinkage-induced stresses can be studied. Research eventually led to the development of a test method for determining shrinkage-induced stresses in overlays. The method and its use are described in

the next section.

Proposed Method

Basis

The basic idea behind the proposed method is to accumulate shrinkage-induced stresses in a restrained polymer concrete overlay, remove the restraint, and measure the total released strain. Studies have shown that interface shear and normal stresses in portland cement concrete beams with polymer concrete overlays are concentrated in the end region (1). These stresses reach a maximum within a distance from the end equal to the depth of the composite beam, and diminish outside this region (4). Therefore, restraint is provided by allowing the overlay to bond to the substrate within the end regions. The overlay within the interior region where normally no interface stresses exist is prevented from bonding to the substrate by a bond breaker. The restraint provided by the end region transforms shrinkage strain into induced stress. Restraint removal transforms the sustained stress back into shrinkage strain which can be practically measured. Physical properties at the time of restraint removal are determined by testing a companion sample placed under similar conditions. Shrinkage-induced axial stress is then determined from simple stress-strain relationships. This procedure eliminates the need to continuously monitor unrestrained shrinkage or physical properties during the initial curing period. The directness of the approach enables the determination of shrinkage induced stresses with a good level of confidence. Repeating the procedure for different release times provides a comprehensive record of shrinkage-induced stresses for a given overlay system.

Procedure

1. Specimen Preparation: A 6-in. x 6-in. x 36-in. portland cement concrete beam was cast and cured for about one month. Areas extending 8 in. from the ends on one surface of the beam were sandblasted to enhance the bond between the beam and the overlay to be placed at a later stage. The beam

was then dried in an oven for 48 hours at a temperature of about 200 °F. After the beam cooled down to room temperature, the middle region was covered with several layers of plastic sheets. A lubricant such as grease or engine oil was provided between the layers to practically eliminate friction between the overlay and the central region of the beam surface. The sides of the beam were then fitted with cardboard forms in preparation for casting the overlay. Consequently, the polymer concrete system under investigation was mixed thoroughly and applied to the beam surface. The exact time at which mixing was started represented the reference point for time measurement. Screeding and troweling were necessary to ensure uniform thickness and proper compaction of the overlay. The composite beam is described schematically in Fig. 2.

2. Instrumentation: Once overlay placement was complete, a DuPont device was positioned within the limits of the unbonded central region. Simultaneously, a similar device was used to measure free shrinkage strain of a sample of the same mix. To ensure compatibility of the two devices, one run was made with the two devices mounted on the same beam. However, there is no need for this step once compatibility has been established.

In one study, strain values at various locations on the surface of the overlay were monitored by electronic strain gages. Thirty-mm electronic strain gages were placed on the surface while the polymer concrete was still fresh and attached to a strain indicator. The intent was to study the behavior of the entire overlay before and after restraint removal rather than the central region alone equipped with a DuPont device. However, these strain gages were measuring the shrinkage behavior of only the top surface, the outer skin, which may have experienced different strains than the center of the overlay. This was evident from a comparison between the results from the gages and from the DuPont device. Therefore, the electronic strain gages did not provide the desired information and their use was discontinued.

3. Restraint Removal: Restraint provided by the substrate through the end regions was removed by separating the central region from either end. Separation was achieved by cutting the overlay transversely near one end of the unbonded central region (Fig. 2). A high speed masonry saw was

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used to make the cut. At this point in time the residual shrinkage strain was measured by the DuPont device. This represented the second point for time measurement (the first was the initiation of mixing).

Verification of the Method

To verify the practicality of the method and the accuracy of the results two composite beams were fabricated and placed in an environmental chamber set to a temperature of 13°F. The overlays were made with epoxy resin formulated by Texaco Chemical Company (3). On a by weight basis, the overlay mix consisted of 54 percent 1/4 in. trap rock, 27 percent sand, 9 percent white portland cement, and 10 percent epoxy resin. It should be noted that the cement was added as a fine filler only since no water was added to the polymer concrete mix. In the neat form, this epoxy has a tensile strength of 8,400 psi, a tensile modulus of 0.39×10^6 psi, and elongation at break of 5 percent. A preliminary investigation revealed that 5 hours were necessary for the interior temperature of the composite beam to stabilize at the chamber temperature of 13°F. The overlay on one beam was cut after 5 hours, on the other after 8 days. The test had two specific objectives:

1. to check the relationship between calculated and measured stresses for a known set of conditions, and
2. to measure total stress relaxation occurring over the eight-day period.

The test variables and properties of materials were as follows:

Total temperature drop:

(74°F to 13°F)

61°F

Average coefficient of thermal expansion

Portland cement concrete substrate: 5.0 $\mu\text{in./in./}^\circ\text{F}$

Polymer concrete overlays: 10.4 $\mu\text{in./in./}^\circ\text{F}$

Thickness

Portland cement concrete substrate:	6.0 in.
Polymer concrete overlay:	0.5 in.

Modulus of elasticity

Average for the temperature range:

Portland cement concrete substrate:	4.5×10^6 psi.
Polymer concrete overlay:	4.5×10^6 psi.

At 13°F

Polymer concrete overlay:	5.0×10^6 psi.
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Poisson's ratio

Portland cement concrete substrate:	0.20
Polymer concrete overlay:	0.24

Strength properties of overlay epoxy concrete

Compressive strength at room temperature:

6 hours	8,900 psi
1 day	12,600 psi
28 days	14,900 psi

Flexural strength at room temperature:

6 hours	2,400 psi
1 day	3,900 psi
28 days	3,900 psi

Axial stresses in the overlays within the interior region can be determined using one of the methods described in reference 4. The stresses were as follows:

Overlay - top fiber	1390 psi (tension)
Overlay - interface fiber	1450 psi (tension)

Assuming the curvature of the beam results in two "cantilevers" extending away from the center, the stresses have also to be modified to include the self weight

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of the beam. The modified axial stresses were not very different from those calculated originally as can be seen from the following:

Overlay - top fiber	1400 psi (tension)
Overlay - interface fiber	1460 psi (tension)
Average across thickness	1430 psi (tension)

On the other hand, the experimental results were as follows:

Beam cut after 5 hours

Strain measured upon cutting	336 $\mu\text{in./in.}$
Average axial stress	1680 psi.

Strain measured upon cutting the overlay corresponds to the actual axial stress present within the central region, averaged across the thickness of the overlay. Therefore, the stresses can be obtained by multiplying the measured strain by the modulus of elasticity of the overlay material.

Beam cut after 8 days

Strain measured upon cutting	249 $\mu\text{in./in.}$
Average axial stress	1250 psi.

The experimental results show clearly that some stress relaxation had taken place during the 8-day period. Tolbert and Hackett (5) provided the following equation for the determination of stress relaxation in polymer concrete made with epoxy binders:

$$E(t) = (2.21053 + 0.72555 e^{-0.00248t} + 0.22333 e^{-0.01832t} + 0.08844 e^{-0.1353t} + 0.20211 e^{-t} + 0.00225 e^{-7.389t} - 0.000207 e^{-54.598t}) \times 10^6 \text{ psi} \quad (1)$$

where

$E(t)$ = relaxation modulus, or the value to be multiplied by the constant (sustained) strain for evaluating the stress at any time t .

t = time in hours measured from time at which strain is applied.