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 napthenate (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 100°F 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 75°F 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 2.5x10^-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/m^3. The porosity was affected significantly by the workability of the mix and varied considerably within the range of the variables. The
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

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

(1)

\[
Y = \frac{\sigma}{\sigma_f} \quad \text{and} \quad X = \frac{\varepsilon}{\varepsilon_f}
\]

(2)

where \(\sigma\) and \(\varepsilon\) are the stress and strain; \(\sigma_f\) and \(\varepsilon_f\) 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
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) and the corresponding crack length in four-point bending can be represented as (16)

\[ \text{CMOD} = 4 \sigma a V(\alpha)/E' \]  

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

\[ V(\alpha) = 0.8 - 1.7 \alpha + 2.4 \alpha^2 + 0.66/(1-\alpha)^2 \]  

Hence if during slow growth CMOD could be determined at various loading levels by unloading the specimens, and hence using Equation (3) it would be possible to determine the corresponding crack length. A numerical iterating procedure was used to determine the corresponding crack length \( a \). Crack extension \( D \) is equal to \( (a - a_i) \), where \( a_i \) is the initial crack length. Assuming a beam of cross section bxd with effective crack length \( a \), the stress intensity factor was calculated using the equation for the four-point bending developed by Brown and Srawley (17). The relationship is as follows:

\[ K_1 = 6 M a^{1/2} Y(\alpha/d)/b d^2 \]  

(5a)
where

\[
Y(a/d) = 1.99 -2.47 \frac{(a/d)}{1 + 12.97 \frac{(a/d)^2}{1 - 23.17 \frac{(a/d)^3}{1 + 24.80 \frac{(a/d)^4}{}}}}
\]

(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

\[
K_R = K_0 + \rho (\Delta a)
\]

(6)

where \(K_0\) and \(\rho\) 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 \(\rho\) 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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**TABLE 1 — COMPOSITION OF POLYMER CONCRETE**

<table>
<thead>
<tr>
<th>Constituent material</th>
<th>Relative weight %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Matrix-Polyester Resin</td>
<td>10 - 18</td>
</tr>
<tr>
<td>Initiator-MEK</td>
<td>1.5*</td>
</tr>
<tr>
<td>Promoter-Cobalt Napthenate</td>
<td>0.3*</td>
</tr>
<tr>
<td>Aggregate-Blasting Sand</td>
<td>82 - 90</td>
</tr>
</tbody>
</table>

MEKP-Methyl Ethyl Ketone Peroxide; *by weight of resin
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
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.
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
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 (Cu) of 5.8 and coefficient of concavity (Cc) 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 outside tubes were smaller for the later. This had resulted in almost doubling of the peak strain, whereas the
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-l 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-l 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

$$N_p = A_c f_c + \sum A_s f_y,$$

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 ($\varepsilon_p$) as shown in the Fig. 7. The relationship is given by

$$\varepsilon_p = \varepsilon_c + \frac{p^2}{11.5 + 0.75p^2},$$

where $p = 4 \left( \frac{A_c}{A_0} \right)^2 \left( \frac{f_{ty}}{E_c} \right)$. 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 ($\varepsilon_o/\varepsilon_p$) 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

$$E_p = E_c A_c + \Sigma E_s A_s, \quad A_c + \Sigma A_s$$

where $E_c$, $E_s$, and $A_c$, $A_s$, are the initial young's modulii and the cross-sectional areas of polymer concrete and steel respectively. The average value of the ratio of measured to predicted initial modulii ($E_o/E_p$), 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

$$\frac{N}{N_o} = \frac{S_n \left( \frac{\varepsilon}{\varepsilon_o} \right)}{1 + \left\{ S_n \left( \frac{\varepsilon}{\varepsilon_o} \right) \right\}^m}^{\frac{1}{m}},$$

where $N/N_o$ = normalized load, $\varepsilon/\varepsilon_o$ = normalized strain, $S_n$ = initial slope of the normalized load-strain curve, and
\[ m = \text{curvature parameter.} \]

\[ S_n = \text{defined as the ratio of the initial slope to the secant slope at the peak (N_0/\varepsilon_0) of the total load-strain curve. The curvature parameter (m) was obtained by the least square fit from the experimental normalized load-strain data. There is a relation between the two parameters as shown in Fig. 8 (16, 17). Knowing the normalized slope (S_n), the curvature parameter (m) can be calculated from the relationship given by} \]

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

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

This research was partly supported by a grant (No. 0036521262ARP) from the Texas Higher Education Coordinating Board to the University of Houston under the Texas Advanced Research Program.
REFERENCES


**TABLE 1 — COMPOSITION OF POLYMER CONCRETE**

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<th>Constituent material</th>
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<td>Initiator-MEKPO</td>
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<td>Aggregate-Blasting Sand</td>
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MEKPO-Methyl Ethyl Ketone Peroxide

*by weight of resin
### TABLE 2 — TYPICAL DIMENSIONS AND PROPERTIES OF STEEL TUBES USED IN SANDWICH SPECIMENS

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<tr>
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<th>t_i (mm)</th>
<th>(D_i)/(D_o)</th>
<th>D_o (mm)</th>
<th>t_o (mm)</th>
<th>(D_o)/(t_o)</th>
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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

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N - load, e - 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)

(a) Failure Dominated by Steel Tube

(i) Shear Type  (ii) Hour glass Type

(b) Failure Dominated by Polymer Concrete

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)
Measurement of Shrinkage-Induced Stress in Polymer Concrete Overlays

by Jamal-Aldin H. Zalatimo and David W. Fowler

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 the 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
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 μin./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
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 x 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 μin./in./°F
- Polymer concrete overlays: 10.4 μin./in./°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 x 10^6 psi.
Polymer concrete overlay: 4.5 x 10^6 psi.
At 13°F
Polymer concrete overlay: 5.0 x 10^6 psi.

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:

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</table>

Flexural strength at room temperature:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>6 hours</td>
<td>2,400 psi</td>
</tr>
<tr>
<td>1 day</td>
<td>3,900 psi</td>
</tr>
<tr>
<td>28 days</td>
<td>3,900 psi</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Description</th>
<th>Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overlay - top fiber</td>
<td>1400</td>
</tr>
<tr>
<td>Overlay - interface fiber</td>
<td>1460</td>
</tr>
<tr>
<td>Average across thickness</td>
<td><strong>1430</strong></td>
</tr>
</tbody>
</table>

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

**Beam cut after 5 hours**
- Strain measured upon cutting: 336 μin./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 μin./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 Flackett (5) provided the following equation for the determination of stress relaxation in polymer concrete made with epoxy binders:

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

where

\[
E(t) = \text{relaxation modulus, or the value to be multiplied by the constant } \text{(sustained) strain for evaluating the stress at any time } t.
\]

\[
t = \text{time in hours measured from time at which strain is applied.}
\]
For example, for time $t = 0$, $E(0) = 3.45 \times 10^6$ psi.

Similarly, for $t = 5$, $E(5) = 3.18 \times 10^6$ psi,
$t = 192$, $E(192) = 2.67 \times 10^6$ psi,
$t = \infty$, $E(\infty) = 2.21 \times 10^6$ psi.

The equation is applicable to epoxy polymer concrete having an initial modulus of elasticity of $3.45 \times 10^6$ psi. The value for $t = 5$ hours indicates a drop in the stress level to 92.2 percent of the original value. The stress level drops to 77.4 percent for $t = 192$ hours (8 days), and ultimately to about 64 percent after an infinite number of hours. Since the polymer concrete under consideration has a modulus of $5.0 \times 10^6$ psi at $13^\circ$F, these percentages will only serve as rough approximations. Using these values, after 5 hours the average axial stress induced by the temperature drop is expected to reduce to

$$1430 \times 0.922 = 1320 \text{ psi}$$

and after 8 days to

$$1430 \times 0.774 = 1110 \text{ psi}.$$  

### FINAL THEORETICAL AND EXPERIMENTAL RESULTS

<table>
<thead>
<tr>
<th>Beam cut after</th>
<th>Theoretical</th>
<th>Experimental</th>
<th>Difference % of theoretical</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 hours</td>
<td>1320 psi</td>
<td>1680 psi</td>
<td>27%</td>
</tr>
<tr>
<td>8 days</td>
<td>1110 psi</td>
<td>1250 psi</td>
<td>13%</td>
</tr>
</tbody>
</table>

The assumptions on which the theoretical solutions are based which are not completely satisfied are responsible for part of the difference; experimental inaccuracies are responsible for the other part. Nevertheless, the agreement is reasonably good.
Shrinkage for Different Polymer Concrete Systems

The magnitude of shrinkage displayed by polymer concrete depends on a number of variables. Among the most important are the type of binder and the proportions of different constituents in the mix. In general, epoxies are known to exhibit relatively low shrinkage in comparison to other polymer systems such as polyesters and polyester-urethane hybrid resins. In this section sample results for epoxy and hybrid resin polymer concrete systems are presented. The same mix proportions were used for both polymer concrete systems. Trials with a mortar system utilizing a partially recycled polyester resin are also described.

A) Epoxy Polymer Concrete (3) - Low Shrinkage System

This epoxy polymer concrete is essentially the same as the system described earlier in the verification of the method. The relative rigidity of epoxies in comparison with other polymer systems adds an interesting aspect to the study as it may translate into less stress relaxation. The temperature rise at peak exotherm followed by subsequent cooling induces thermal stresses in the overlay. The strain and temperature scales in the figures were chosen such that a unit length of change on either scale produces a unit length of change on the other in accordance with the value of the coefficient of thermal expansion. This scale selection allows the direct realization of strain that is due to temperature change rather than to shrinkage. Subtracting an equivalent amount of strain from the given shrinkage strain curve yields the true value of shrinkage strain.

First, a test was run for 72 hours. According to Fig. 3, no measurable residual stresses were present at the time of restraint removal indicating the possibility that a period of 72 hours was too long and allowed complete stress relief. A second test was run over a period of only 9 hours in an attempt to reduce the time available for stress relaxation. Again no residual stresses were indicated as shown in Fig. 4. A further time reduction was tried, taking the total time period down to 7 hours. For a third time no shrinkage stresses were indicated. The results for the 7-hour test are plotted in Fig. 5. No attempt was made to investigate the behavior at shorter time periods since the low stiffness of the polymer concrete at such early stages would not permit the development of
significant stresses. The restrained and unrestrained shrinkage strain curves are plotted together in Fig. 6.

**B) Polyester/Urethane Concrete (6) - High Shrinkage System**

In the neat form, this polyester/urethane system has a tensile strength of 3,600 psi, a tensile modulus of $0.20 \times 10^6$ psi, and elongation at break of about 20 percent. In the concrete form (73°F, 1 day), it has a compressive strength of 10,000 psi, and a flexural strength of 2,700 psi. The free shrinkage plot is shown in Fig. 7. Results for time periods of 9 hours and 3 hours are shown in Figs. 8 and 9. The three curves are plotted together in Fig. 10.

**C) Polyester-Based Mortar - Very High Shrinkage System**

The high unrestrained shrinkage strain for a polyester-based mortar made this trial very interesting. The combined effects of the absence of coarse aggregate and the type of resin used produced the unrestrained shrinkage curve shown in Fig. 11. This mortar has a modulus of elasticity of about $1.50 \times 10^6$ psi after 4 hours of curing and about $1.70 \times 10^6$ after 24 hours. It also has a tensile strength of approximately 500 psi at 4 hours and 1100 psi at 24 hours. Several trials to measure shrinkage-induced strain using the proposed method were unsuccessful since the overlay would typically crack within the 10-in. gage length before cutting was performed (Fig. 12). The first overlay was 1.0-in. thick and cracked after about 90 minutes from the time of mixing. The other two were 0.5-in. thick overlays and cracked after about 3 hours and 45 minutes from the time of mixing. These trials showed that systems having very high unrestrained shrinkage properties are likely to develop problems at an early age. At later stages, shrinkage-related problems are less likely since the strength would be higher and part of the stress would already be relieved through viscoelastic stress relaxation. It should not be assumed, however, that shrinkage-induced stresses in these systems are completely relieved with time since there is not enough evidence to support that. In other words, the possibility of problems continues indefinitely especially if the overlay is subjected to severe combinations of thermal and shrinkage stresses.
Discussion

The experiment in which the composite beam was placed in an environmental chamber indicated clearly that the epoxy overlay experienced a viscoelastic stress reduction through relaxation. This agrees with previous research on stress relaxation of epoxy polymer concrete (5). The experiment also proved that stress was not completely relieved during the 8-day period. This emphasizes the fact that cured polymer concrete is a viscoelastic material as opposed to a viscous material, otherwise the stress would have been completely relieved. Yet the figures plotted for the epoxy polymer concrete overlays indicate that shrinkage-induced stresses were non-existent. This suggests that the polymer concrete mix from which the epoxy overlays were made remained relatively viscous for a long period of time, thus allowing a major part of shrinkage to be "absorbed" without any noticeable stress build-up. Figure 6 supports this hypothesis as it may be seen from the figure that the unrestrained sample began to develop significant strength only after about four to five hours. In addition, the commencement of strength development marks the formation of long, cross-linked molecules. The first molecules to develop in an overlay would become stressed due to restrained shrinkage. Molecules forming at a later stage would initially be unstressed and begin to develop stress as shrinkage proceeds. Furthermore, strength gain continues for a few days while shrinkage almost stops in less than a day, thus the last molecules to form would be virtually unstressed. The result is a combination of molecules stressed to different levels. In other words, the overlay contains highly, moderately, and lowly stressed molecules rather than molecules stressed equally to an average stress level. This concentration of stresses in portion of the total number of molecules increases the rate of stress relaxation. This is a major difference from the thermal case where molecules in the same vicinity would be stressed to an equal level.

1 Strength development for the unrestrained sample was slightly slower than normal due to heat loss to the steel mold, especially since the sample was relatively thin. This was intended for better representation of an overlay placed on a much thicker substrate.
For the polyester/urethane polymer concrete, about 300 µin./in. of residual shrinkage strain was detected when restraint was removed after 3 hours, and about 250 µin./in. when removed after 9 hours. Obviously, some relaxation occurred during the six-hour span, but the stresses were not completely relieved. Explanation for the observed behavior is essentially the same as for the epoxy polymer concrete except for the following:

1. Unrestrained shrinkage strain for the polyester/urethane polymer concrete is much higher than for the epoxy polymer concrete.

2. The polyester/urethane polymer concrete developed strength at an earlier age and a much faster rate. The result is that more residual shrinkage-induced stresses were locked in. Also the range of stress levels among neighboring molecules is smaller, resulting in less stress relaxation.

The results for the two polymer concrete systems are plotted together in Fig. 13. As for the mortar system, the shrinkage-induced stresses were high enough to cause cracking before any measurements could be made; nevertheless, they add an interesting side to the discussion. It can be observed from Fig. 14 that the unrestrained shrinkage strain of the epoxy polymer concrete is almost negligible when compared to the other two systems, and that the mortar system shrinks more than twice as much as the polyester/urethane polymer concrete. Accordingly, systems with high unrestrained shrinkage strain are likely to sustain some shrinkage-induced stresses after curing. The stresses will definitely not be as high as may be indicated by a free shrinkage test, but they may be severe enough to cause problems at an early age when the overlay has not yet developed full strength. Once full strength is developed and part of the stress is relaxed, the possibility of shrinkage-related problems becomes smaller but continues to exist and, in combination with high thermal or other stresses, can cause distress to the overlay.

The previous line of reasoning may be used to explain a related phenomenon. Figure 14 indicates that the polyester based mortar system begins to develop strength almost immediately after mixing, followed by the polyester/urethane polymer concrete after 1 hour then the epoxy concrete after
about 4 hours. It may be suggested that systems that develop strength at a slower rate exhibit a smaller total unrestrained shrinkage strain. In slow curing materials such as many epoxies, the molecules that form initially resist the shrinkage-related deformation accompanying the formation of newer ones. In fast curing materials such as polyesters most of the molecules develop almost together within a short period of time during which most of the shrinkage strain takes place. Therefore the previous notion of older molecules resisting the shrinkage strain accompanying the formation of newer ones is not as prevalent here. It should be noted that in Fig. 14 most of the unrestrained shrinkage strain for the polyester/urethane polymer concrete and the polyester based mortar occur within the first few hours. It is also very interesting to note that after about five hours the curves for the three systems in Fig. 14 are almost identical.

Conclusions

The proposed method provides a helpful tool for predicting the effects of thermally-induced and shrinkage-induced stresses in overlays. In general, slow curing systems with low total unrestrained shrinkage strain appear to be more favorable candidates for overlays, but other properties such as strength, and coefficient of thermal expansion should not be ignored.

Shrinkage-induced stresses may not be encountered with the use of certain systems such as the epoxy polymer concrete considered in this study. Other systems may sustain a residual amount of shrinkage-induced stress indefinitely. The stress may be severe enough by itself to cause distress at an early age, or in combination with other stresses at later stages. It should not be assumed that long-term stresses in overlays are completely relieved with time since polymer concrete is a viscoelastic material not a viscous mixture.
REFERENCES


Fig. 1—DuPont curing shrinkage test

Fig. 2—Curing shrinkage in overlays — schematic representation of proposed method
Fig. 3—Shrinkage strain for epoxy polymer concrete overlay. Restraint removed after 72 hr

Fig. 4—Shrinkage strain for epoxy polymer concrete overlay. Restraint removed after 9 hr
Fig. 5—Shrinkage strain for epoxy polymer concrete overlay. Restraint removed after 7 hr.

Fig. 6—Comparison of restrained and unrestrained shrinkage strain for epoxy polymer concrete.
Fig. 7—Unrestrained shrinkage strain for polyester/urethane polymer concrete system

Fig. 8—Shrinkage strain for polyester/urethane polymer concrete overlay. Restraint removed after 9 hr
Fig. 9—Shrinkage strain for polyester/urethane polymer concrete overlay. Restraint removed after 3 hr

Fig. 10—Comparison of restrained and unrestrained shrinkage strain for polyester/urethane polymer concrete system
Fig. 11—Unrestrained shrinkage strain for polyester based polymer mortar
Fig. 12—Crack in the polyester-based mortar overlay
Fig. 13—Comparison of restrained and unrestrained shrinkage of epoxy and polyester/urethane systems

Fig. 14—Comparison of unrestrained shrinkage strain of the three systems
Shear and Flexure Behavior of Reinforced Polymer Concrete Made with Recycled Plastic Wastes

by K. S. Rebeiz and David W. Fowler

Synopsis: Very little research has been done on the structural behavior of steel-reinforced polymer concrete (PC). In all the previous studies, it was generally assumed that the structural behavior of reinforced PC is similar to the structural behavior of reinforced portland cement concrete because both are composite materials consisting of a binder and inorganic aggregates. However, the design equations developed for steel-reinforced portland cement concrete yield very conservative results when applied to reinforced PC. The objective of this paper is to report on the shear and flexure properties of steel-reinforced PC beams using unsaturated polyester resins based on recycled poly(ethylene terephthalate), PET, plastic waste. The effects of the shear span-to-depth ratio, reinforcement ratio, and compressive strength were investigated with the shear beams, while the effect of reinforcement ratio was investigated with the flexure beams. New design equations were also developed to predict the shear and flexural strength of steel-reinforced PC beams.

Keywords: Plastics, polymers, and resins; polymer concrete; recycling; reinforcing steels; shear properties; structural design; unsaturated polyester; wastes
INTRODUCTION

The development of new composite materials that are stronger and more durable than conventional materials is an essential element in the repair and improvement of the infrastructure. Polymer concrete (PC) is an example of a relatively new high performance composite material that is produced by mixing well graded inorganic aggregates with a resin binder [1]. The most common type of binder used in PC is unsaturated polyester because of its good properties and relatively low cost. PC is very strong, very durable, virtually impermeable, and cures very fast.

Like portland cement concrete, PC has relatively low tensile strength compared to its compressive strength. Therefore, in some applications, it may be necessary to add reinforcing steel in the tensile zone of PC to increase its strength capacity, its ductility, and its toughness. The objective of this paper is to investigate the shear and flexure properties of steel-reinforced PC and to recommend an efficient, yet simple design procedure in this area for steel-reinforced PC beams.

Most of the PC beams were made with unsaturated polyester resin based on recycled PET plastic waste. The PET waste is mainly recovered from beverage bottles that are collected after use in many localities; thus, it is available in fairly large quantities for recycling applications. Previous work has shown that PC using resins with recycled PET is potentially less expensive, yet comparable in mechanical and durability properties to PC using 100% virgin resins [2]. The recycling of PET in PC would also help save energy and alleviate an environmental problem posed by plastic wastes.

This study is therefore not only important from the standpoint that resins based on recycled plastics were used in the production of PC, but also because very few investigations have been done on the shear and flexure properties of reinforced PC using virgin resins.
EXPERIMENTAL

Materials

PET consists of repeating ethylene glycol and terephthalic acid molecules connected together through ester linkages. For the production of unsaturated polyester, the PET molecules are converted into low molecular weight oligomers by glycolysis in the presence of a transesterification catalyst. These oligomers are then reacted with unsaturated dibasic acids or anhydrides to form unsaturated polyester resins [5]. A variety of other chemical additives may also be used during the production process to give the resin some specific properties such as flexibility or rigidity. The percentages of recycled PET used in these resins varied from a low concentration of 15% by weight to a high concentration of 40% by weight. The maximum possible percentage of recycled PET is usually desirable in making unsaturated polyesters because the PET residues do not adversely affect the PC properties and their use helps in reducing the cost of the material [6, 7].

The PC mix design, proportioned by weight and optimized for workability, strength and economy was 10% resin, 45% oven-dried 3/8-in. pea gravel, 35% river oven-dried sand, and 10% type F fly ash. One percent (by weight of resin) of 9% active oxygen methyl ethyl ketone peroxide initiator and 0.1% (by weight of resin) of 12% solution cobalt naphthenate promoter were added to the resin immediately prior to its mixing with inorganic aggregates to start the hardening process. The PC mixing procedure followed the "Polymer Concrete Test Method 1.0" of the Society of Plastics Industry [8]. Mixing was done using a conventional concrete mixer for a period of about three minutes. Curing of the mixture was done at room temperature.

PC Beams Details

Two types of steel-reinforced PC beams were constructed: Shear beams and flexure beams. A typical shear and flexure specimens are shown in Figures 1 and 2, respectively. The reinforcing steels that were used in this study were Grade 60 deformed bars conforming to ASTM Standard A615. The reinforcing steel bars were located at the bottom of the beams (i.e., in the tensile zone) with a cover of 1.5 in. The beams were simply supported and two equal concentrated loads were applied at the third points of the span.

No web reinforcement was included in the shear specimens because it is necessary to determine the shear strength of concrete before the addition of stirrups. Three linear transducers were used to measure the vertical deflection. One transducer was located at the bottom surface at mid-span while the other two were located over the supports at the top of the beam to measure any eventual crushing of the concrete at the bearing surfaces. Three electrical strain gages were installed on the concrete at mid-span (one in the compression zone next to the top fibers and the other two on each each sides of the beam) and one electrical strain gage was attached at the midpoint of each longitudinal reinforcing steel bar. All beams were painted white after the application of the strain gages so that cracking developed during testing could well be seen and marked.
The two outer portions of the flexural beams were over-reinforced with vertical closed stirrups to prevent a shear mode failure and, thus, ensure a flexure failure. Vertical stirrups were #3 Grade 60 deformed steel bars spaced 3 in. apart (high strength wire was used to securely tie the stirrups). Each stirrup was rectangular in shape and was closed by welding its ends together. Three linear transducers, installed at the quarter-points of the clear span, were used to measure vertical deformations. Five electrical strain gages installed in the compression zone of each beam and one electrical strain gage attached at the midpoint of each longitudinal reinforcing steel bar were used to measure the strain deformations.

**Testing Procedure**

All reinforced PC beams were tested seven or nine days after casting, thus giving the PC ample time to reach its final strength. A Tinius-Olson load frame, with a maximum capacity of 120 kips, was used to load the beams to failure at a rate 10 kips per minute. The vertical deflection of the beams was measured using linear transducers. Plexiglass squares were used to provide a smooth surface for the transducers. PC compressive strains and the tensile strains in the steel were measured using electrical strain gages that were applied according to the manufacturer's instructions. Strains and deflections were recorded electronically by the Fluke data acquisition system at a load interval of one kip until failure. Periodically, the load was held constant for about 30 to 40 seconds to mark the PC cracking.

The compressive strength of various reinforced beams were expected to vary under different mixing and curing conditions. Therefore, control specimens of unreinforced PC were mixed, cured, and tested simultaneously with the corresponding reinforced PC beams. Control specimens, which consisted of two 3-in. x 6-in. compression cylinders for each PC beam, were loaded to failure at a rate 10 kips per minute.

**SHEAR BEHAVIOR**

**Shear Properties**

The failure induced by shear in concrete structures is particularly dangerous and should be avoided in the design process. The effect of shear is to induce diagonal tension stresses in a concrete beam. Failure occurs when these stresses exceed the diagonal tensile strength of the material. However, a design procedure using stresses acting on diagonal planes is practically very difficult to establish because of the uncertainties in establishing the diagonal tension strength of the material. The acceptable design procedure in shear for concrete structures is based on the assumption that a shear failure at the critical section occurs on a vertical plane when the shear stress at that section exceeds the concrete's fictitious vertical shear strength (also called nominal shear strength). The shear stress of concrete is therefore defined as the applied shear force divided by the effective area of the cross section (i.e., $\tau = \frac{V}{bd}$).
The shear test results are summarized in Table 1. Two types of shear strength have been measured: shear strength at the occurrence of a first major diagonal crack ($v_d$) and the ultimate shear strength ($v_u$).

The diagonal shear strength, $v_d$, is defined as the shear force causing the formation of a major diagonal crack divided by the effective area of the cross section (i.e., $v_d = \frac{V_d}{b d}$). Similarly, the ultimate shear strength, $v_u$, is defined as the shear force causing total and complete shear failure divided by the effective area of the cross section (i.e., $v_u = \frac{V_u}{b d}$). The occurrence of a first diagonal crack in reinforced PC provides a warning that significant tension stresses may lead to a violent and sudden failure in the beam. The magnitude of the difference between the shear strength at the formation of a diagonal crack and the ultimate shear strength depends greatly on the shear span-to-depth ratio, $a/d$, the steel reinforcement ratio, $\rho$, and the compressive strength, $f_c$.

The effects of $a/d$, $\rho$, and $f_c$ on the shear strength of PC are shown in Figures 3 to 5. As shown in Figure 3, an increase in $a/d$ resulted in a decrease in the shear strength of the PC since flexural stresses increased and arch-rib action diminished. It was also observed that the difference between the shear strength at failure and the shear strength at the formation of the first diagonal crack decreased with an increase in $a/d$. Redistribution of stresses, after the formation of a diagonal crack, was easier in short beams than in long beams because of the relatively short distance between the supports and the applied loads.

As shown in Figure 4, an increase in $\rho$ resulted in an increase in the ultimate shear strength of the concrete, while it did not affect the shear strength at the formation of the first diagonal crack. This phenomenon can be explained by the fact that dowel action of the steel really took place after the formation of a diagonal crack. The effect of $f_c$ on the shear strengths is shown in Figure 5. An increase in $f_c$ did not seem to significantly affect the shear strength of the concrete, although it was observed that the maximum ultimate shear strength values corresponded to the maximum $f_c$ value.

Shear Strength Prediction

To predict the shear strength of PC beams, it is useful to know the shear-carrying capacity of concrete beams reinforced in bending only before the addition of shear reinforcements or stirrups. If the shear in the critical section is greater than the "nominal" shear resistance, then stirrups are added to carry the difference. The ACI-ASCE approach, the conventional procedure used in predicting the shear strength of portland cement concrete, was first tried with the PC shear strength data. This approach uses $\frac{v_u}{\sqrt{f_c}}$ as the dependent variable and $\frac{\rho}{a/d\sqrt{f_c}}$ as the independent variable [9]. It should be noted that the shear span, $a$, has been defined for beams loaded with two symmetrically placed concentrated loads and
therefore it is equal to the moment-to-shear ratio. For other loading conditions, the shear span, a, is often substituted with moment-to-shear ratio in the design of reinforced concrete members, even though these two quantities are not equal. Previous work has shown that the ACI-ASCE approach did not yield good results in predicting the shear strength of PC beams mainly because of the difference in behavior between short beams (a/d < 2.5) and long beams (a/d ≥ 2.5) [10].

An alternative method of predicting the shear strength of PC beams was therefore developed using multiple regression analysis. This procedure took into account the difference in behavior between short beams and long beams by using an interpolation function. The interpolation function would increase the shear capacity of PC beams as a/d decreases from 2.5 to 1.0 and would remain constant for values of a/d beyond 2.5. Shear strength at ultimate failure, $v_u$, was considered in this analysis because it is a better reflection of the shear carrying capacity of the concrete and it is a more defined and reliable measure than shear strength at first diagonal cracking, $v_d$. Using multiple regression analysis, the following simple, yet accurate equation proved to give the best results:

$$v_u = [138 - 14 (A_d)^2] \sqrt{\frac{f'_c}{a}} \frac{d \rho}{a} \quad \text{[English units]} \quad (1)$$

$$v_u = [11.5 - 1.17 (A_d)^2] \sqrt{\frac{f'_c}{a}} \frac{d \rho}{a} \quad \text{[SI units]} \quad (1a)$$

Figure 6 compares the theoretical values of the ultimate shear strength obtained from Equation (1) with the observed ultimate shear strength values of PC beams. Most of the data are from PC using unsaturated polyester resins based on recycled PET. Few data are obtained from PC made with virgin unsaturated polyester resins [11]. In this equation, the coefficient of determination ($r^2$) is 0.95. The small scatter of experimental data around the theoretical line (as evidenced by the high coefficients of determination) confirms the fact that the model developed is an excellent predictor of the PC shear strength.

For design purposes, it is desirable to have the theoretical shear strength values slightly overestimate the observed shear values. A reduction factor of 0.75 to Equation (1) will result in the derivation of Equation (2). Figure 7 compares the theoretical values of the ultimate shear strength obtained from Equation (2) with the observed ultimate shear strength values. It can be observed that the use of Equation (2) results in a 100% data exclusion, meaning that all shear values obtained from experiment are larger than the theoretical shear values predicted by Equation (2).

$$v_u = [103 - 10 (A_d)^2] \sqrt{\frac{f'_c}{a}} \frac{d \rho}{a} \quad \text{[English units]} \quad (2)$$

$$v_u = [8.74 - 0.89 (A_d)^2] \sqrt{\frac{f'_c}{a}} \frac{d \rho}{a} \quad \text{[SI units]} \quad (2a)$$
FLEXURAL BEHAVIOR

Flexural Properties

A summary of the test results on the flexural strength of reinforced PC is provided in Table 2. Failure took place when the ultimate compressive strain in the PC reached a value of at least 0.005. Variation of strain over the compression region up to failure for all the flexural beams was found to be almost perfectly linear. The cracking patterns for reinforced flexural PC beams were similar and generally typical to those observed in reinforced portland cement concrete. However, PC exhibited more cracks that were more uniformly spaced than what would be observed with portland cement concrete with the same reinforcement ratio, thus indicating a larger bond strength between PC and steel than between portland cement concrete and steel.

The typical experimental moment-curvature responses for lightly reinforced and heavily reinforced PC beams are shown in Figures 8 and 9, respectively. The moment was computed from the applied loads by statics while the corresponding curvature was calculated from the strain distributions. In the first stage of loading, a linear relationship existed between the moment and curvature. This proportional limit stage ended with the formation of a major vertical flexural crack and the resulting change in slope and decrease in stiffness. In lightly reinforced beams, the reinforcing steel bars yielded significantly before failure took place. Therefore, failure occurred with gradual crushing of the PC in the compression zone. Conversely, in heavily reinforced beam, yielding in the steel took place just before failure happened. As a result, the failure mode in the compression zone of the heavily reinforced beams was sudden and explosive.

The neutral axis depth (measured from the top compression fiber) was determined using a linear distribution of strain for each load increment. Figures 10 and 11 show the plot of neutral axis depth versus loads for lightly reinforced and heavily reinforced beams, respectively. In lightly reinforced beams, the neutral axis depth decreased (i.e., indicating a deeper flexural crack) as failure was about to occur because yielding in the tensile steel occurred before concrete reached its ultimate compressive strain capacity. Conversely, the neutral axis depth raised as failure was about to occur for the heavily reinforced PC beams.

Flexural Strength Prediction

To predict the ultimate flexural strength of the various beams, the equivalent rectangular stress block, adapted from ACI 318, was first used [12]. The values of the stress intensity factor for the PC compressive rectangular stress block, $K_u$, ranged from 0.6 to 1.5. The results of this analysis showed that the ACI procedure is very conservative in predicting the flexural strength of PC beams [12]. One reason may be the fact that a large portion of the PC internal moment at failure is resisted by the tensile stresses in the PC (it should be noted that the flexural strength of PC is at least 6 times higher than the flexural strength of portland cement concrete). Another reason may be the increase in the steel stress beyond the yielding limit (i.e., the stress in the steel does not remain constant beyond the yielding limit as assumed in the computations of the theoretical
moment). It should also be noted that a larger bond strength exists between PC and steel than between portland cement concrete and steel.

An alternative method of flexural strength prediction for PC beams using rectangular cross-sections and tensile reinforcement only was tried by using multiple regression analysis. This analysis combined data obtained from PC systems using unsaturated polyester based on recycled PET with data obtained from PC systems using virgin resins to provide with a more general strength prediction equation [13, 14]. The PC systems using virgin resins were unsaturated polyester, methyl methacrylate (MMA), polyesteramide resin (PEAR), epoxy, and vinyl ester. The dependent variable of this analysis is $M_{bd}$ (the ultimate moment strength) whereas the independent variables are $f'_c$ and $\rho_f$ (preliminary analyses results showed that the adoption of the independent variable $f'_c$ yielded better results than the use of the independent variable $f'_c$). Equation 3 was obtained from the analysis using $\rho_f$ and $f'_c$ as independent variables. Figure 12 shows the scatter of data of the observed moment strength values versus the theoretical values as predicted by Equation 3. In this equation, the coefficient of determination ($r^2$) is 0.81. The relatively small scatter of experimental data around the theoretical line confirms the fact that the model developed is a reasonable predictor of the PC flexural strength.

$$
\frac{M_{bd}}{bd^2} = 0.343 \rho_f y + 19.29 \sqrt{f_c} - 90 \quad \text{[English units]} \quad (3)
$$

$$
\frac{M_{bd}}{bd^2} = 0.343 \rho_f y + 1.60 \sqrt{f_c} - 0.63 \quad \text{[SI units]} \quad (3a)
$$

For design purposes, it is desirable to have the observed moment strength overestimate the theoretical moment strength. Equation 4, obtained by modifying Equation 3, is therefore the proposed model for predicting the ultimate moment strength of reinforced PC. Figure 13 shows the scatter of data of the observed moment strength values versus the theoretical values as predicted by Equation 4. As observed in Figure 13, the theoretical model generally slightly overestimates the flexural strength obtained from the test results.

$$
\frac{M_{bd}}{bd^2} = 0.33 \rho_f y + 15 \sqrt{f_c} \quad \text{[English units]} \quad (4)
$$

$$
\frac{M_{bd}}{bd^2} = 0.33 \rho_f y + 1.245 \sqrt{f_c} \quad \text{[SI units]} \quad (4a)
$$

It should be noted that Equations 3 and 4 only apply to reinforced PC with rectangular cross-sections having tensile reinforcement, and with compressive strengths typically ranging from 8000 psi to 14000 psi.
CONCLUSIONS

Very good flexural and shear strengths were obtained with steel-reinforced PC using unsaturated polyester resins based on recycled PET. Flexural failure in reinforced PC beams occurred when the ultimate compressive strain in the concrete reached a value of 0.005 or more. As loads were applied to the beams, variation of strain over the compression region was found to be almost perfectly linear. The steel reinforcement ratio was found to be a very important variable in the ultimate bending strength of the flexural beams. However, an increase in the reinforcement ratio beyond a certain level resulted in flexural failure with virtually no ductility. The shear span-to-depth ratio had the biggest effect on the shear behavior of reinforced beams. The tensile reinforcement ratio did increase the shear strength of reinforced beams but only after the formation of a major diagonal crack. However, an increase in compressive strength did not seem to significantly affect the shear strength of reinforced PC beams. Generally accepted design guidelines in shear and flexure developed for reinforced portland cement concrete were found to be too conservative for use in reinforced PC beams. A new design procedure based on empirically derived equations in flexure and shear was therefore proposed. Future work should concentrate on the use of different cross-sectional shapes and dimensions for the beams, the addition of compression and shear reinforcements, and different loading types.

NOTATION

\[ a = \text{shear span, in. (mm)} \]
\[ \frac{a}{d} = \text{shear span-to-depth ratio} \]
\[ A_d = \text{adjustment factor} = \begin{cases} 
   \text{a/d for } 1.0 < \frac{a}{d} < 2.5 \\
   \text{2.5 for } a/d \geq 2.5 
\end{cases} \]
\[ b = \text{width of PC beam, in. (mm)} \]
\[ d = \text{distance from extreme compression fiber to centroid of tensile steel, in. (mm)} \]
\[ f_c = \text{compressive strength of PC, psi (MPa)} \]
\[ f_y = \text{yield strength of steel, psi (MPa)} \]
\[ M_u = \text{ultimate moment, lbs-in. (KN-m)} \]
\[ \frac{M_u}{bd^2} = \text{ultimate flexural strength (or moment strength), psi (MPa)} \]
\[ \nu = \frac{V_u}{bd} = \text{ultimate nominal shear strength, psi (MPa)} \]
\[ V_u = \text{ultimate shear force capacity, lbs (N)} \]
\[ \rho = \text{Tensile reinforcement ratio, in decimals} \]

REFERENCES

1. ACI Committee 548, 1986, "Guide for the Use of Polymers in Concrete," *ACI 548.1R-86*, American Concrete Institute.


TABLE 1 — SHEAR TEST RESULTS FOR PC USING RECYCLED PET

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<thead>
<tr>
<th>Beam</th>
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<th>2√f'c (psi)</th>
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TABLE 2 — SUMMARY OF FLEXURAL TEST RESULTS

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Fig. 1—Reinforced PC shear beam

Fig. 2—Reinforced PC flexural beam
Fig. 3—Effect of shear span-to-depth ratio, \( \alpha/d \), on shear strength

Fig. 4—Effect of longitudinal steel reinforcement ratio, \( \rho \), on shear strength
Fig. 5—Effect of compressive strength, $f_c'$ on shear strength

Fig. 6—Observed and theoretical shear values using Eq. 1
Fig. 7—Observed and theoretical shear values using Eq. 2

Fig. 8—Experimental moment-curvature responses for lightly-reinforced beams
Fig. 9—Experimental moment-curvature responses for heavily-reinforced beams.

Fig. 10—Neutral axis depth variation moment versus moment for lightly-reinforced beams.
Fig. 11—Neutral axis depth variation versus moment for heavily-reinforced beams

Fig. 12—Observed and theoretical moment strength values using Eq. 3
Fig. 13—Observed and theoretical moment strength values using Eq. 4
Strength Properties of Polymer Concrete Made with Resins Based on Recycled Plastic Waste

by K. S. Rebeiz and David W. Fowler

Synopsis: Recycled polyethylene terephthalate (PET) plastic wastes could be used in the production of unsaturated polyester resins. If specially formulated, these unsaturated polyester resins could in turn be mixed with inorganic aggregates to produce polymer concrete (PC). The results of an extensive research confirm that PC materials using resins based on recycled PET are comparable in strength to conventional PC materials. Resins based on recycled PET could also easily be altered to achieve a relatively wide range in the strength and flexibility properties of the PC depending on the intended use of the material. PC using resins based on recycled PET may be utilized in the repair and overlay of portland cement concrete structures or in the production of various precast products such as utility, transportation, and building components. The recycling of PET in PC would help decrease the cost of PC products, save energy, and alleviate an environmental problem posed by plastic wastes.

Keywords: Modulus of elasticity; plastics, polymers and resins; polyethylene terephthalate; polymer concrete; reinforcing steels; strength; stress-strain relationships; unsaturated polyester; wastes
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D. W. Fowler is the Dean T. U. Taylor Professor in Engineering in the Department of Civil Engineering at the University of Texas at Austin. He is past chairman of American Concrete Institute Committee 548 Polymers in Concrete, and past-president of the International Congress on Polymers in Concrete. He is a member of numerous ACI and ASCE committees.

INTRODUCTION

Polymer concrete (PC) is a relatively new high performance material that has been commercialized since the early sixties. It is produced by mixing a liquid polymer resin with inorganic aggregates. Therefore, PC consists of well-graded inorganic aggregates bonded together by a polymer binder as opposed to the typical hydrated cement binder used in portland cement concrete. The most common type of binder used in PC is unsaturated polyester because of its good properties and relatively low cost.

PC is a strong and durable material. The fast curing time of PC is another important advantage in many construction applications (PC cures in a few minute or hours while portland cement concrete cures in a few days or weeks). PC can be used very effectively in the repair and overlay of damaged cement concrete surfaces in pavements, bridges, floors, and dams. Another effective use of the material is in precast components such as hazardous waste containers, trench lines, floor drains, high voltage insulators, railroad ties, and bases for large machinery.

The high cost of resins used in the production of PC makes the material expensive relative to cement-based materials. Not surprisingly, a recent survey ranked lower cost resins as the most important future need for PC (1). Unsaturated polyesters based on recycled polyethylene terephthalate (PET) plastic waste might be a potentially lower source cost of resins for producing useful PC based-products. The PET wastes are typically found in used beverage bottles, and many states have passed legislation to collect and recycle these bottles. If specially formulated, the unsaturated polyester could be used in the production of PC (2). A main advantage of recycling PET in PC is that the PET materials do not have to be purified, including removal of colors, to the same extent as other PET recycling applications (such as carpets and fiberfill), which should facilitate the recycling operation and minimize its cost. The recycling of PET in PC could also help save energy and allow the long term disposal of the PET waste, an important advantage in recycling applications. The objective of this paper is therefore to report on the strength properties of unreinforced and steel-reinforced PC using unsaturated polyester resins based on recycled PET.
EXPERIMENTAL

Materials

PET consists of repeating ethylene glycol and terephthalic acid molecules connected together through ester linkages. For the production of unsaturated polyester, the PET molecules are converted into low molecular weight oligomers by glycolysis in the presence of a transesterification catalyst. These oligomers are then reacted with unsaturated dibasic acids or anhydrides to form unsaturated polyester resins (2, 3). A variety of other chemicals may also be used during the production process to give the resin some specific properties such as flexibility or rigidity.

Phthalic anhydride or isophthalic acid is typically used in the formulation of conventional unsaturated polyester. Virgin terephthalic acid is not usually used in the production of unsaturated polyester because it is expensive and possesses a high melting point, which presents difficulties in synthesis. Conversely, recycled PET is effective in incorporating terephthalic functionality into the backbone of a polyester resin (3, 4, 5). Terephthalic-based polyesters exhibit more linear properties than isophthalic or orthophthalic-based polyesters when the polymerization reaction with typical glycols and acids occurs because of the location of the carboxyl groups on the benzene ring of phthalic acid. Terephthalic, isophthalic, and orthophthalic-based polyesters have their carboxyl groups in the para, meta, and ortho position of the benzene ring, respectively. The degree of linearity, in descending order, results from the para, meta, and ortho-based structures. Therefore, the more linear molecular structure of terephthalic-based polyester resins, as compared to isophthalic or orthophthalic based-polyesters, allows the cross-linking reaction to be more accessible when free radical polymerization takes place upon final curing, thus producing a more uniformed and structured cross-linking matrix with a higher degree of strength, stiffness, and toughness. Another advantage of using recycled PET in making unsaturated polyesters, as compared to using virgin materials, is that it takes about 50% shorter processing time to produce a polyester resin with a certain molecular weight and acid number (2).

The experimental unsaturated polyester resins used in this study were supplied by several chemical companies since the PET chemical conversion into unsaturated polyester could not be done at the university laboratories. These resins were prepolymers with high viscosity. They were therefore diluted with styrene to reduce their viscosity and allow their further cure to a solid (polymer) upon the addition of suitable free radical initiators and promoters. The typical styrene content varied between 30 to 40% of the total resin weight, and viscosities were in the range of 100 to 1000 cps.

Mix Design

The PC mix design by weight, optimized for workability, strength, and economy, was 10% resin, 45% pea gravel (3/8-in.), 30% sand (fineness modulus of 3.25), and 15% fly ash (type F). The inorganic aggregates were oven-dried for a minimum of 24 hours at 260 °F to reduce their moisture content to less than 0.5% by weight, thus ensuring good adhesion between the polymer matrix and the aggregates. Fly ash was already obtained dry from the supplier and therefore did not need to be oven-dried. The use of fly ash greatly improved the workability of the fresh mix and the strength of the hardened material. It also helped produce PC
specimens with very smooth surfaces. One percent, by weight of resin, of methyl
ethyl ketone peroxide initiator (9% active oxygen) and 0.1%, by weight of resin, of
cobalt naphthenate promoter (12% solution) were added to the resin immediately
prior to its mixing with inorganic aggregates. The PC mixing procedure followed
the Polymer Concrete Test Method 1.0 of the Society of Plastics Industry (6). The
age at testing of the specimens was three days, unless otherwise specified,
although the specimens could have been tested much sooner.

Testing

The neat resins (cured resins without the use of aggregates) were tested for
t heir tensile and flexural properties according to ASTM procedures (7). There are
no standard tests that are directly applicable to PC. Therefore, ASTM standards
developed for cement based-materials were used as guidelines whenever applicable.
Unreinforced PC specimens were tested for their compressive strength by using 3-
in. x 6-in. cylinders. The specimens were tested in a hydraulic load machine at a
constant loading rate of 10,000 lbs/min. Electrical strain gages were bonded to the
specimens and connected to an automated data acquisition system in a full-bridge
configuration. Unreinforced flexural specimens used 2-in. x 2-in. x 12-in. beams.
A typical unreinforced PC flexural specimen is shown in Figure 1. The beams
were loaded in third-point loading, at a uniform rate of 500 lbs/min.

Rectangular beams, reinforced in tension with longitudinal steel bars, were
tested for their flexural behavior and strength. A typical steel-reinforced PC beam
is shown in Figure 2. The reinforcing steels were Grade 60 deformed bars
conforming to ASTM Standard A615. The beams were simply supported and two
equal concentrated loads were applied at the third points of the span. The shear
span-to-depth ratio was 4.0. The two outer portions of the flexural beams were
over-reinforced with vertical closed stirrups to prevent a shear mode failure.
Vertical stirrups were # 3 Grade 60 deformed steel bars spaced 3 in. apart (high
strength wire was used to securely tie the stirrups). Electrical strain gages were
bonded to the reinforcing steel and to the PC at various levels and connected to a
data acquisition system. The vertical deflection of the beams was measured using
linear transducers. The beams were loaded to failure with strains and deflections
recorded for each load level.

RESULTS AND DISCUSSION

Tensile Strength Properties of the Neat Resin

The typical tensile stress-strain curves for two neat resins based on recycled
PET is shown in Figure 3. It can be observed in the figure that the stress-strain
properties of the two neat resins are quite different. One resin is rigid since it has a
high tensile strength, high tensile modulus, and low elongation at failure.
Conversely, the other resin is flexible as evidenced by the fact that it has a low
tensile strength, low modulus, and high elongation at failure is flexible. These
different properties are very important in the potential use of these resins in PC
applications. Rigid resins are suitable in the production of PC for precast
applications because they produce high modulus structures that deflect and creep
less under the action of loads. Conversely, flexible resins are suitable in the
production of PC for overlay applications to provide compatibility to thermal and
mechanical movements with minimum delamination between the PC overlay and the portland cement concrete substrate.

**Compressive and Flexural Strength Properties of Unreinforced PC**

A typical stress-strain curve in compression for two PC systems, one using a flexible resin and the other using a rigid resin, is shown in Figure 4. It can be seen that the type of resin (rigid or flexible) used in making PC can significantly affect the stress-strain properties of the material. PC made with a flexible resin exhibits lower modulus, higher ductility, and more toughness than the PC made with a rigid resin. The stress-strain behavior and the ultimate compressive strength of PC using resin based on recycled PET are comparable to those obtained with PC using virgin resins (8). It can also be observed that the ultimate compressive stress and strain of PC are much larger than the ones corresponding to portland cement concrete.

The failure of PC in compression was violent. Compression cylinders would shatter violently and the remaining core of the cylinders had either a cone shape or a near vertical failure surface. Flexural beam specimens also failed in a violent manner as a tensile crack developed in the zone of maximum moment near mid-depth. The specimens were broken into almost two identical pieces and the failure surface was near vertical.

**Effect of PET Level on the Strength of Neat Resins and Unreinforced PC**

A D-optimal experimental design was developed to determine the effect of recycled PET as a raw component in unsaturated polyester resins. This study was done on several resins made with the same basic formulation but with the percent PET and glycol type as the only variables. The three different glycols that were used in the digestion process of the recycled PET were ethylene glycol, diethylene glycol, and a combination of ethylene and diethylene glycol. These three different glycols will be referred to as glycols I, 2, and 3 respectively. Unsaturated polyester resins were synthesized with a low PET concentration of 15% by weight and a high PET concentration of 40% by weight of the alkyd portion of the resin. The D-optimal experimental design consisted of eight unsaturated polyester resin preparations with two replicates.

The effects of recycled PET on the tensile and flexural strengths of the neat resins are shown in Figures 5 and 6, respectively. The tensile strength of the neat resin increases with increasing amount of PET for glycols 1 and 2, while it remains almost constant for glycol 3. The tensile strength of resins using glycol 3 is the highest, followed by resins using glycols 1 and 2, respectively. The flexural strength of all neat resins increases with increasing amounts of PET. The flexural strength of resins using glycol 3 is highest, followed by resins using glycol 1, and then resins using glycol 2.

The effects of percent PET and glycol type on the PC compressive and flexural strengths are shown in Figures 7 and 8, respectively. The compressive strength of PC using resins made with glycols 1 and 2 increases significantly with increasing PET, while it increases very slightly for PC using resins made with glycol 3. The compressive strength of PC made with resins using glycol 3 is the highest, followed by PC made with resins using glycol 1, and then PC made with resins using glycol 2. The flexural strength of PC made with resins using glycols 1
and 3 increases with increasing PET, while it decreases for PC made with resins using glycol 2. At a low PET concentration of 15%, all systems are comparable in flexural strength. At a high PET concentration of 40%, the flexural strength of PC made with resins using glycol 1 is the highest, followed by PC made with resins using glycol 3, and then PC made with resins using glycol 2.

**Flexural Strength of Steel-Reinforced PC**

Like portland cement concrete, PC has relatively low tensile strength compared to its compressive strength. Therefore, sudden and brittle failures occur in planes where tensile stresses exceed the tensile strength of the material. Reinforcing steel is therefore added in tensile zones of PC to increase its strength capacity, its ductility, and its toughness.

The effect of the tensile steel reinforcement ratio, $\rho$, on the moment-curvature responses of PC beams using resins based on recycled PET is shown in Figure 9. In the first stage of loading, a linear relationship existed between the moment and curvature. This proportional limit stage ended with the formation of a major vertical flexural crack and the resulting change in slope and decrease in stiffness.

The cracking patterns for steel-reinforced flexural PC beams were similar and generally typical to those typically observed in steel-reinforced portland cement concrete. However, PC exhibited more cracks that were more uniformly spaced than what would be normally observed with portland cement concrete with the same reinforcement ratio, thus indicating a larger bond strength between PC and steel than between portland cement concrete and steel. Variation of strain over the compression region up to failure for the beams was found to be almost perfectly linear. Failure occurred when the ultimate compressive strain in the PC reached a value of at least 0.005. As the beams failed, the compressive concrete piece separated as a V-shape.

The ultimate flexural strength of steel-reinforced PC using resins based on recycled PET was compared with other steel-reinforced concrete systems as shown in Figure 10. The other concrete systems were portland cement concrete, unsaturated polyester (virgin resin), methyl methacrylate (MMA), polyesteramide resin (PEAR), epoxy, and vinyl ester. It is observed that the flexural strength of PC using resins based on recycled PET is much higher than the one corresponding to portland cement concrete. It should be noted that a large portion of the PC internal moment at failure is resisted by the tensile stresses in the concrete, unlike what would be happening with steel-reinforced portland cement concrete. In addition, a larger bond strength exists between PC and steel than between portland cement concrete and steel. It is also observed in Figure 10 that the flexural strength of steel-reinforced PC using resins based on recycled PET is very comparable to the ones obtained with the other PC systems.

**CONCLUSIONS**

This study has shown that the strength properties of PC materials using resins based on recycled PET are very comparable to those obtained with PC materials using virgin resins. Resins based on recycled PET can also relatively
easily be altered to achieve a wide variety of properties and performances. Resins with high strength and stiffness can be formulated for precast applications, while resins with low modulus, high elongation at break, and good bond strength to Portland cement concrete can be formulated for overlay applications. A higher level of recycled PET in the polyester generally increases the strength of the neat resin and the PC depending on the type of glycol used in formulating the polyester resins.

Field applications and continuous monitoring of PC materials using resins based on recycled PET would really determine the long term behaviors of these materials under field conditions. Special precautions should be taken in cases involving large sustained loads because the viscoelastic nature of the resin binder can result in unreasonably high deformations. Special resin formulations, adequate supports, and/or large safety factors would be advisable in these instances. Trial testing of the materials is also advisable because it would help overcome problems due to erratic cure or batch to batch inconsistencies of the resins. For applications requiring very high strength, a special mix using strong aggregates and special reinforcements should be formulated.

REFERENCES

Fig. 1—Unreinforced PC flexural specimen
Fig. 2—Steel reinforced PC beam

Fig. 3—Typical stress-strain curves in tension for polyester resins based on recycled PET
Fig. 4—Typical stress-strain curves in compression for PC using resins with recycled PET

Fig. 5—Effect of level of PET on tensile strength of neat resins
Fig. 6—Effect of level of PET on flexural strength of neat resins

Fig. 7—Effect of level of PET on PC compressive strength
Fig. 8—Effect of level of PET on PC flexural strength

Fig. 9—Typical moment-curvature responses for steel-reinforced PC beams
Fig. 10—Ultimate moment strength of various steel-reinforced PC systems
Synopsis: An analytical procedure was developed by which thermally-induced stresses in polymer concrete overlays can be quantified. The distribution and the magnitude of thermally-induced stresses can be determined using the proposed procedure. The main variables which influence the thermal-stress development are found to be the thickness ratio and the modular ratio of the polymer concrete to the portland cement concrete, the difference in the coefficients of thermal expansion and the temperature change. The relationships between the variables and the thermal-stresses are determined and presented. Analyses reveal that thermally-induced interface stresses decrease as the elastic modulus and the thickness of the overlay decrease for the thin polymer concrete overlays. The analyses assumed isothermal conditions.

Keywords: Axial stress; beams (supports); concretes; portland cements; resurfacing; shear stress; stresses; thermal stresses
INTRODUCTION

During the last decade, many states in the US and several provinces in Canada have reported the application of thin polymer concrete (PC) overlays on existing portland cement concrete (PCC) bridge decks. A PC overlay is one of the most promising methods of rehabilitation for existing PCC bridge decks because of its high strength, excellent durability and good bond to the PCC. A PC overlay has an advantage over the thicker PCC overlays when live load capacity or clear height considerations are critical since the PC overlays are installed in very thin layers. Curing shrinkage and the coefficient of thermal expansion ($\alpha$) of the PC are usually higher than for the PCC. It is not unusual for the $\alpha$ of the PC to be twice as high as that of the PCC. Despite the successful performance of many PC overlays, several delamination failures of the PC overlays from the PCC subbase have been reported. Delamination failure usually begins at the boundaries. Studies by Sprinkel and Al-Negheimish suggest that the major contributors to the delamination are stresses which are generated near the boundaries of the pavement by the high curing shrinkage and the high $\alpha$ of the PC.
This paper will quantify the thermally-induced stresses in the PC-PCC composite beam. A narrow strip of a PC-PCC composite slab was modeled as an elastic two-layered composite beam in plane stress condition. The thermally-induced internal stresses were found analytically. The distributions of thermally-induced stresses were investigated and the variables which influence the stress development were found. Since overlays are more accurately represented by plane strain, the analyses for the composite beam previously derived were modified to yield a plane strain solution. The findings were used to predict thermally-induced stresses in actual polymer concrete overlays. Analyses presented in the present study include the effect of temperature change only.

THEORETICAL BACKGROUND

Timoshenko\textsuperscript{7} first acknowledged the end-bound nature of the interface shear stresses ($\tau$) in his pioneering study of the composite laminates subjected to a uniform temperature change. Using the elementary beam theory, he obtained the axial stress ($\sigma_x$) distributions of bi-metal thermostats. Hess\textsuperscript{4} made one of the first major contributions to the analysis of end stress concentration by deriving the end stresses for a two-layer laminated strip using an eigenfunction series expansion. Grimado,\textsuperscript{3} using the elementary beam theory, attempted a solution by treating the adhesive layer as the third layer. One of the most comprehensive analyses of the composite member subjected to uniform temperature change was performed by Chen et al. Chen et al.\textsuperscript{2}, like Grimado, treated the adhesive layer as the third layer and formulated a two-dimensional plane stress problem by considering the compatibility conditions between composite layers and the stress-free boundary conditions. The variational energy method was then used to obtain a set of governing differential equations that satisfy all stress-free boundary conditions of the composite beam subjected to a uniform temperature change. Chen et al.'s method is extended in this study to the analysis of thermally-induced stresses in PC overlays.

Al-Negheimish\textsuperscript{1} experimentally studied the end stresses in a PC-PCC composite beam under thermal cyclic loading and also analytically studied the end stresses using a finite element model.
ASSUMPTIONS AND ANALYSIS OF BEAM MODELS

In many PC overlays, the PC is overlaid directly on existing PCC without the help of an adhesive material since the PC bonds well to the PCC. Analytically an adhesive layer with a very small thickness needs to be introduced to analyze the composite beam using Chen's method. A master PC-PCC composite beam model used in the analyses of thermal stresses is shown in Fig. 1. The following assumptions are introduced in the analyses of composite beams:

1. the stress-strain relationship in the PC and the PCC is linearly elastic;
2. the effect of the very thin adhesive layer is negligible; and
3. the PC-PCC composite beam is subjected to a uniform temperature change and the difference in $\alpha$ as remains constant during temperature change.

Tables 1 and 2 summarize the thicknesses and the elastic moduli of the PC and the PCC, respectively, used in the analyses while the analysis parameters are defined as follows:

\[ m = \text{the thickness ratio of the PC to PCC} \ (h_1/h_2) \]
\[ n = \text{the modular ratio of the PC to PCC} \ (E_1/E_2) \]

The elastic modulus, $\alpha$ and Poisson's ratio ($\nu$) of the thin adhesive layer were taken as same as the PC.

GOVERNING DIFFERENTIAL EQUATION AND GENERAL SOLUTION

Chen's governing coupled differential equations, written in terms of stress functions $\bar{\sigma}_1$ and $\bar{\sigma}_1^*$, are as follows:
The coefficients $C_1$ through $C_{11}$ are functions of the elastic modulus, thickness, $\alpha$ and Poisson's ratio of the composite section as shown in Appendix. In a typical PC-PCC composite beam where the thickness of the PC is much smaller than that of the PCC, there are eight complex eigenvalues of the characteristic equation. However, depending on the thickness ratio and modular ratios of the two layers, some eigenvalues may become real as shown in Fig. 2. Chen et al. presented a solution to Eq. 1 when eigenvalues are four real and four complex. However, a more general solution to Eq. 1 involves eight complex eigenvalues as shown in Fig. 2. In this case, the general form of the homogeneous solution is as follows:

$$\sigma_{1H} = k_1 \cos(\beta_1 \xi) \cosh(\alpha_1 \xi) + k_2 \sin(\beta_1 \xi) \sinh(\alpha_1 \xi) + k_3 \cos(\beta_2 \xi) \cosh(\alpha_2 \xi) + k_4 \sin(\beta_2 \xi) \sinh(\alpha_2 \xi)$$

$$\tilde{\sigma}_{1H} = \left\{ k_1 \gamma_1 \cos(\delta_1) + k_2 \gamma_1 \sin(\delta_1) \right\} \cos(\beta_1 \xi) \cosh(\alpha_1 \xi) - \left\{ k_1 \gamma_1 \sin(\delta_1) - k_2 \gamma_1 \cos(\delta_1) \right\} \sin(\beta_1 \xi) \sinh(\alpha_1 \xi) + \left\{ k_3 \gamma_2 \cos(\delta_2) + k_4 \gamma_2 \sin(\delta_2) \right\} \cos(\beta_2 \xi) \cosh(\alpha_2 \xi) - \left\{ k_3 \gamma_2 \sin(\delta_2) - k_4 \gamma_2 \cos(\delta_2) \right\} \sin(\beta_2 \xi) \sinh(\alpha_2 \xi)$$

while the eight eigenvalues are

$$\pm(\alpha_1 + i\beta_1), \quad \pm(\alpha_1 - i\beta_1), \quad \pm(\alpha_2 + i\beta_2), \quad \pm(\alpha_2 - i\beta_2).$$

The constants in Eqs. 3 can be determined as follows:

$$\sin(\delta_i) = \sqrt{\frac{1}{f_i^2+1}}$$

$$\cos(\delta_i) = -f_i \sin(\delta_i)$$
\[ \gamma_i = -\frac{a_i}{c_i \cos(\delta_i) - d_i \sin(\delta_i)} \]

while,
\[
\begin{align*}
 a_i &= C_1 (\alpha_i^4 + \beta_i^4 - 6 \alpha_i^2 \beta_i^2) + C_2 (\alpha_i^2 - \beta_i^2) + C_3 \\
 b_i &= 4C_1 (\alpha_i^3 \beta_i - \alpha_i \beta_i^3) + 2C_2 \alpha_i \beta_i \\
 c_i &= C_4 (\alpha_i^4 + \beta_i^4 - 6 \alpha_i^2 \beta_i^2) + C_5 (\alpha_i^2 - \beta_i^2) + C_6 \\
 d_i &= 4C_4 (\alpha_i^3 \beta_i - \alpha_i \beta_i^3) + 2C_5 \alpha_i \beta_i \\
 f_i &= \frac{a_i c_i + b_i d_i}{a_i d_i - b_i c_i} \quad (i = 1, 2)
\end{align*}
\]

When the governing equation is solved and the values of the stress functions are found, they are used to calculate stresses at any desired point in the beam. The particular solution to Eq. 1 and Chen's formula to determine thermally-induced stresses at the interface are shown in Appendix.

RESULTS OF BEAM MODEL ANALYSES

Stress Distribution on Composite Beam Sections

Results are presented for the uniform temperature drops of 100°F (55.6 °C). When a PC-PCC composite beam is subjected to a temperature drop, the PC tries to contract more than the PCC while the composite beam restricts the differential contraction of the PC because the \( \alpha \) of the PC is larger than that of the PCC. Since it is assumed that the two layers cannot slip relative to each other at the interface, the beam is subjected to an internal bending moment and an axial force. The axial stresses (\( \sigma_x \)) that develop remain constant for most of the beam span, but they decrease near the end of the beam and become zero at the end (Fig. 3). The tensile stress is positive while the compressive stress is negative in Fig. 3. By taking advantage of the symmetry, only half the beam is shown in Fig. 3.

The development of interface shear (\( \tau \)) and interface normal stresses (\( \sigma_y \)) is also confined to the narrow region near the end (Fig. 4). The interface shear stress develops over a distance approximately equal to the total depth of the
composite beam. The shear stress increases initially at a slow rate but increases at a faster rate as it approaches the end. The maximum shear stress is reached at a point very near to the end and begins to decay rapidly, becoming zero at the end of the beam. Figure 5 shows the distribution of shear stress over the depth of the composite beam. The maximum shear stress along the cut section always develops at the interface. The maximum shear stress decreases rapidly in the thin PC and decays to zero at the top extreme fiber. In the thicker PCC, shear stress decreases for some distance but again begins to increase and reaches the second peak. The shear stress decays to zero at the bottom extreme fiber.

The interface normal stress ($\sigma_y$) develops along the interface over a similar distance to that of the shear stress (Fig. 4). The tensile interface normal stress (positive stress) develops near the end of the composite beam. Farther along the interface line this tensile stress changes to compressive stress (negative stress). Figure 6 shows the distribution of normal stresses over the depth of the composite beam. The normal stress at the interface is typically of much lower magnitude than the stress which develops below the interface.

**Variables Influencing Stress Development**

Analyses were done for thicknesses of the PC much smaller than those of the PCC to investigate the thermally-induced stresses in a realistic PC-PCC composite overlay. The thicknesses of the PC are 0.12-in. (0.30-cm), 0.48-in. (1.22-cm) and 0.72-in. (1.83-cm), which correspond to thickness ratios of 0.015, 0.06 and 0.09, respectively, in the following analysis. The analysis reveals that the interface shear stress ($\tau$) decreases with decreasing thickness of the overlay for a constant modular ratio, as shown in Fig. 7. The interface normal stress ($\sigma_y$) also decreases when the thickness of the overlay decreases in Fig. 8. Similar analyses were conducted for the different elastic moduli of the PC. The modular ratios of the PC to PCC are 0.5, 0.25 and 0.025, respectively, while the thickness ratio is 0.06. The elastic moduli of the PC are 2,000,000 psi (13,780 MPa), 1,000,000 psi (6,890 MPa) and 100,000 psi (689 MPa), while that of the PCC is 4,000,000 psi (27,560 MPa), in Figs. 9 and 10. Interface shear stress decreases when the modular ratio decreases from 0.5 to
0.025 in Fig. 9. The interface normal stress also decreases as the modular ratio decreases in Fig. 10.

Interface shear and normal stresses developed in PC-PCC composite beams are proportional to the uniform temperature changes. Interface stresses also develop in proportion to the difference of the αs between the two layers. A variable which includes the effect of the above two variables can be defined as follows:

\[ \Delta \varepsilon_T = - \Delta T (\alpha_1 - \alpha_2) \]  

(5)

In Equation 5, \( \Delta \varepsilon_T \) denotes the total strain due to the difference in the αs and the temperature change in the unrestrained PC. The thermally-induced interface stresses have a linear relationship with \( \Delta \varepsilon_T \) as defined in Eq. 5.

THERMAL STRESSES IN POLYMER CONCRETE OVERLAYS

Actual thermally-induced stresses in the polymer concrete bridge deck or the pavement overlays are represented by the plane strain solutions rather than the plane stress solutions obtained for the beams. The parameters of Chen's equation (Eq. 1) can be suitably adjusted to convert plane stress results to plane strain results (ref. 5). The result of the plane strain beam analysis represents the thermally-induced stresses in a typical section of PC-PCC composite pavement. The plane strain PC-PCC models were analyzed with results summarized in Figs. 11, 12 and 13.

Axial stress (\( \sigma_x \)) in the PC always increases as the thickness of the PC decreases (Fig. 11). When the elastic modulus of the PC is much smaller than that of the PCC, interface axial stress increases at a slow rate as the thickness ratio decreases. For example, axial stress increases only slightly as the thickness of the PC decreases when the modular ratio is 0.125. Axial stress increases at a faster rate when the modular ratio is 1.0. The analysis results reveal that when thin PC with a high elastic modulus is overlaid on existing PCC a high level of tensile axial stress develops along the interface as the temperature drops. The maximum shear stress (\( \tau \)) at the interface always decreases as the thickness of
the PC and the elastic modulus of the PC decrease (Fig. 12). Interface shear stress decreases at a slow rate as the thickness ratio decreases when the modular ratio is relatively small. When the PC overlay is relatively thick, the change in the thickness of the overlay has a small effect on the maximum shear stress. When the PC overlay is very thin and the elastic modulus of the PC is high, the maximum shear stress decreases significantly as the thickness of the PC decreases. It can be concluded that a PC overlay, which has a thinner PC and a lower elastic modulus of the PC, develops smaller interface shear stress than the one with a thicker PC and a higher elastic modulus of the PC. The maximum normal stresses ($\sigma_y$) at the interface were found and plotted from similar analyses of axial stress and shear stress. When the thickness ratio is less than 0.3 the maximum normal stress at the interface always decreases as the thickness ratio decreases (Fig. 13). When the thickness ratio is larger than 0.3 the maximum normal stress can decrease or increase according to the modular ratios.

Using Figs. 11,12 and 13 and the finding that thermally-induced axial and interface stresses have a linear relationship with $\Delta \varepsilon_T$ (Eq. 5), thermal stresses in the actual PC overlay can be determined as follows:

**Example** Find thermal-stresses in the PC overlay induced by temp. drop of 50 °F.

- PC overlay
  - $E_1 = 2 \times 10^6$ psi
  - $h_1 = 1.0$ in.
  - $\alpha_1 = 12.5 \times 10^{-6}$ in./in./°F

- PCC subbase
  - $E_2 = 4 \times 10^6$ psi
  - $h_2 = 8.0$ in.
  - $\alpha_2 = 5.5 \times 10^{-6}$ in./in./°F

- $n = E_1 / E_2 = 0.5$; $m = h_1 / h_2 = 0.125$;
- $\Delta \alpha = 7.0 \times 10^{-6}$ in./in./°F; $\Delta T = -50$°F; $\Delta \varepsilon_T = 350 \mu$m./in.

**Axial stress in PC**

$\sigma_x = 1070$ psi * (350 / 500) = 749 psi

**Interface stresses in PC**

$\tau = 309$ psi * (350 / 500) = 216.3 psi

$\sigma_y = 154$ psi * (350 / 500) = 107.8 psi
Al-Negheimish conducted a laboratory study of the end stresses in a PC-PCC composite beam under thermal cyclic loading. For a 64.0-in.(162.6-cm)-long composite beam having a 6.0-in.(15.2-cm)-thick PCC subbase and a 2.0-in.(5.1-cm)-thick PC overlay, the shear strain near the end of the beam was measured with strain rosettes that were installed close to the interface. Also, the beam curvature was measured at the mid span of the composite beam using the strain gages. The measured normal and shear strains were used to determine axial and shear stresses induced in the composite beam under the thermal cyclic loading. Axial stresses ($\sigma_x$) at the mid span and shear stresses near the end of the composite beam determined analytically were compared to the experimental result. Figure 14 shows the axial stresses over the depth of the composite beam. Analytically-determined axial stresses closely agree with those measured in the experiment. The difference between the calculated and measured axial stresses at the bottom fiber of the PCC comes from the effect of the longitudinal reinforcing steel used near the bottom of the test beam (two no. four bars). Figure 15 shows the comparison of shear stresses ($\tau$) near the end of the composite beam determined analytically and by test. The experimental results of Al-Negheimish represent the average of shear stresses determined from four cycles of thermal loading. The geometric center of the strain rosettes (2.0-cm x 2.0-cm) was 0.5-in.(1.3-cm) above and below the interface. Both analytical and test results show the concentration of high shear stresses toward the end of the PC-PCC composite beam subjected to a uniform temperature drop of 36 °F (20 °C). Shear stresses determined analytically and by test show close agreement at distances equal to 0.5-in.(1.3-cm) and 5.0-in.(12.7-cm) from the end. The experimentally measured shear stresses, at a distance equal to 2.5-in.(6.4-cm) from the end, are higher than those predicted analytically although the reason cannot be clearly explained. It could be suggested that the high stress concentration resulted from the non-homogeneity of the PC and PCC (which deviates from the assumptions on the homogeneity of materials in the analysis). The work of Al-Negheimish provides an experimental proof for the use of the proposed analysis procedure on the investigation of thermally-induced stresses in PC-PCC composite beams.
CONCLUSIONS

An analytical procedure was presented by which thermally-induced stresses in polymer concrete overlays can be determined. The result of analysis represents the thermally-induced stresses in a typical section of PC-PCC composite pavement subjected to an ideal uniform temperature change. The analysis results are summarized as follows:

1. When a PC-PCC composite beam is subjected to a temperature change internal stresses develop when the coefficients of thermal expansion of the two layers are not the same. Thermally-induced interface stresses are mostly concentrated near the end of the composite beam.

2. Axial stresses (σx) which are introduced by a temperature change remain constant for most of the beam span but begin to decay to zero near the end. The distance over which the axial stresses decay to zero is approximately equal to the total depth of the composite beam from the end.

3. The maximum shear stress (τ) at the interface is reached very near to the end of the composite beam. The maximum shear stress over the depth of the composite beam always develops at the interface.

4. The maximum normal stress (σy) at the interface is reached at the end of the beam or proximate to the end of the composite beam. The normal stress at the interface is typically of much lower magnitude than the stress which develops below the interface for a PC-PCC composite beam having the PC overlay much thinner than the PCC subbase.

5. The thickness ratio and the modular ratio of the PC to the PCC influence the development of interface stresses when it is subjected to a temperature change.

6. The interface shear stress (τ) decreases with decreasing thickness of the overlay for a constant modular ratio. The interface shear stress also decreases as the elastic modulus of the PC decrease for a constant thickness ratio.

7. When the thickness ratio of the PC to the PCC is less than 0.3, the interface normal stress (σy) also decreases as the thickness and the elastic modulus of the PC decrease.
It should to be noted that the thickness ratio is rarely as high as 0.3 in actual PC overlay as the typical maximum thickness of the PC overlay is in the order of 1.0-in. So in a realistic PC overlay, a PC-PCC composite section which has a thinner PC overlay and a lower elastic modulus of the PC develops smaller interface shear and normal stresses than one with a thicker PC overlay and a higher elastic modulus of the PC. Based on analysis results, it is recommended that the overlay thickness and the elastic modulus of the polymer concrete be lowered until the possibility of delamination is reduced when the calculated interface thermal stresses exceed the expected bond strength at the interface.

References

1. Al-Negheimish, A. I., "Bond Strength, Long Term Performance and Temperature-induced Stresses in Polymer Concrete-Portland Cement Concrete Composite Members." Ph.D. Dissertation presented to the University of Texas at Austin in partial fulfillment of the requirements for the degree of Doctor of Philosophy, 1988.


**APPENDIX**

Determination of Coefficients in Eq. 1

\[ C_1 = 2e_7; C_2 = 2e_{10} - 2e_4; C_3 = 2e_1; C_4 = e_8; C_5 = e_{11} + e_{12} - e_5; C_6 = e_2; \]
\[ C_7 = 2e_9; C_8 = 2e_{13} - 2e_6; C_9 = 2e_3; C_{10} = -e_{16}; C_{11} = -e_{17} \]

while,

\[ e_1 = \bar{\ell} + \left( A_1^2 - A_1A_2 + \frac{A_2^2}{3} \right) \bar{h}_2 / \bar{E}_2 \]
\[ e_2 = \frac{\bar{\ell}}{3} + (2A_1B_1 - A_1B_2 - A_2B_1 + \frac{2}{3}A_2B_2) \bar{h}_2 / \bar{E}_2 \]
\[ e_3 = \frac{\bar{\ell}}{3} + (B_1^2 - B_1B_2 + \frac{B_2^2}{3}) \bar{h}_2 / \bar{E}_2 \]
\[ e_4 = 2 \left( 1 + \nu_1 \right) / 3\bar{\ell} + 2 \left( 1 + \nu_3 \right) \bar{h}_3 / \bar{\ell} \bar{E}_3 \]
\[ + \left( 1 + \nu_2 \right) \left( \frac{2}{3}A_1^2 - \frac{5}{6}A_1A_2 + \frac{4}{15}A_2^2 \right) \bar{h}_3^2 / \bar{\ell} \bar{E}_2 \]
\[ e_5 = 5 \left( 1 + \nu_1 \right) / 6\bar{\ell} + 2(1 + \nu_3) \bar{h}_3 / \bar{\ell} \bar{E}_3 \]
\[ + \left( 1 + \nu_2 \right) \left( \frac{4}{3}A_1B_1 - \frac{5}{6}A_1B_2 - \frac{5}{6}A_2B_1 + \frac{8}{15}A_2B_2 \right) \bar{h}_3^3 / \bar{\ell} \bar{E}_2 \]
\[ e_6 = 4 \left( 1 + \nu_1 \right) / 15\bar{\ell} + (1 + \nu_3) \bar{h}_3 / 2\bar{\ell} \bar{E}_3 \]
\[ + \left( 1 + \nu_2 \right) \left( \frac{2}{3}B_1^2 - \frac{5}{6}B_1B_2 + \frac{4}{15}B_2^2 \right) \bar{h}_3^2 / \bar{\ell} \bar{E}_2 \]
\[ e_7 = 1/20 \bar{\ell}^3 + \left( \frac{1}{4} \bar{h}_3 + \frac{1}{2} \bar{h}_3^2 + \frac{1}{3} \bar{h}_3^3 \right) \bar{h}_2 / \bar{\ell}^3 \bar{E}_3 \]
\[ + \left( \frac{1}{20}A_1^2 - \frac{13}{180}A_1A_2 + \frac{11}{420}A_2^2 \right) \bar{h}_3^5 / \bar{\ell}^3 \bar{E}_2 \]
\[ e_8 = 13/180 \bar{\ell}^3 + \left( \frac{1}{3} \bar{h}_3 + \frac{7}{12} \bar{h}_3^2 + \frac{1}{3} \bar{h}_3^3 \right) \bar{h}_2^5 / \bar{\ell}^3 \bar{E}_3 \]
\[ + \left( \frac{1}{10}A_1B_1 - \frac{13}{180}A_1B_2 - \frac{13}{180}A_2B_1 + \frac{22}{420}A_2B_2 \right) \bar{h}_3^5 / \bar{\ell}^3 \bar{E}_2 \]
\[ e_9 = 11/420 \bar{\ell}^3 + \left( \frac{1}{9} \bar{h}_3 + \frac{1}{6} \bar{h}_3^2 + \frac{1}{12} \bar{h}_3^3 \right) \bar{h}_2^5 / \bar{\ell}^3 \bar{E}_3 \]
\[
\begin{align*}
c_{10} &= -\nu_1 / 3\ell + \nu_2 \left( -\frac{1}{3} A_1^2 + \frac{1}{3} A_1 A_2 - \frac{1}{15} A_2^2 \right) \bar{h}_2^2 / \ell \bar{E}_2 \\
c_{11} &= -\nu_1 / 4\ell + \nu_2 \left( -\frac{1}{3} A_1 B_1 + \frac{1}{4} A_1 A_2 + \frac{1}{12} A_2 B_1 \right) \\
&\quad - \frac{1}{15} A_2 B_2 \bar{h}_2^3 / \ell \bar{E}_2 \\
c_{12} &= -\nu_1 / 12\ell + \nu_2 \left( -\frac{1}{3} A_1 B_1 + \frac{1}{4} B_1 A_2 + \frac{1}{12} B_2 A_1 \right) \\
&\quad - \frac{1}{15} B_2 A_2 \bar{h}_2^3 / \ell \bar{E}_2 \\
c_{13} &= -\nu_1 / 15\ell + \nu_2 \left( -\frac{1}{3} B_1^2 + \frac{1}{3} B_1 B_2 - \frac{1}{15} B_2^2 \right) \bar{h}_2^2 / \ell \bar{E}_2 \\
c_{16} &= -2\nu_1 \bar{\ell} \alpha_1 t / (1 - \nu_1) + (2A_1 \bar{C}_1 - A_1 \bar{C}_2 - A_2 \bar{C}_1 + \frac{2}{3} A_2 \bar{C}_2) \bar{\ell} \bar{h}_2 / \bar{E}_2 \\
&\quad + \nu_2 (-2A_1 + A_2) \bar{\ell} \bar{h}_2 \alpha_2 t / (1 - \nu_2) \\
c_{17} &= -\nu_1 \bar{\ell} \alpha_1 t / (1 - \nu_1) + (2B_1 \bar{C}_1 - B_1 \bar{C}_2 - B_2 \bar{C}_1 + \frac{2}{3} B_2 \bar{C}_2) \bar{\ell} \bar{h}_2 / \bar{E}_2 \\
&\quad + \nu_2 (-2B_1 + B_2) \bar{\ell} \bar{h}_2 \alpha_2 t / (1 - \nu_2)
\end{align*}
\]

where
\[
\bar{\ell} = \ell / h_1, \quad \bar{h}_2 = h_2 / h_1, \quad \bar{h}_3 = h_3 / h_1, \\
\bar{E}_2 = E_2 / E_1, \quad \bar{E}_3 = E_3 / E_1, \\
\bar{C}_1 = C_1 / E_1, \quad \bar{C}_2 = C_2 / E_1
\]

in which
\[
\begin{align*}
A_1 &= -\left( \frac{h_1}{h_2} \right)^2 \left( \frac{4h_2}{h_1} + \frac{6h_3}{h_1} + 3 \right) \\
B_1 &= -\left( \frac{h_1}{h_2} \right)^2 \left( \frac{2h_2}{h_1} + \frac{3h_3}{h_1} + 2 \right) \\
C_1 &= \left( \frac{h_1}{h_2} \right)^2 \left( \frac{4h_2}{h_1} + \frac{6h_3}{h_1} + 3 \right) \cdot \frac{E_1 \alpha_1 t}{1 - \nu_1} + \frac{E_2 \alpha_2 t}{1 - \nu_2} \\
A_2 &= -\left( \frac{h_1}{h_2} \right)^2 \left( \frac{12h_3}{h_1} + 6 + \frac{6h_2}{h_1} \right) \\
B_2 &= -\left( \frac{h_1}{h_2} \right)^2 \left( \frac{6h_3}{h_1} + 4 + \frac{3h_2}{h_1} \right) \\
C_2 &= \left( \frac{h_1}{h_2} \right)^2 \left( \frac{12h_3}{h_1} + 6 + \frac{6h_2}{h_1} \right) \cdot \frac{E_1 \alpha_1 t}{1 - \nu_1}
\end{align*}
\]


**Interface stresses in PC**

\[
\sigma_x = \sigma_1(x) - \frac{E_1 \alpha_t t}{1 - \nu_t}
\]

\[
\tau_{xy} = \frac{d\sigma_i}{dx} h_i + \frac{d\sigma_i}{dx} \frac{h_i}{2}
\]

\[
\sigma_y = \frac{d^2\sigma_i}{dx^2} \frac{h_i^2}{2} + \frac{d^2\sigma_i}{dx^2} \frac{h_i^3}{3}
\]

while,

\[
\sigma_i = \bar{\sigma}_i E_t, \quad \sigma_i^* = \bar{\sigma}_i^* E_t
\]

\[
\bar{\sigma}_i = \bar{\sigma}_{ip} + \bar{\sigma}_{ih}, \quad \bar{\sigma}_i^* = \bar{\sigma}_{ip}^* + \bar{\sigma}_{ih}^*
\]

and

\[
\bar{\sigma}_{ip} = (-2e_3 \cdot e_{16} + e_2 \cdot e_{17}) / (4e_1 \cdot e_3 - e_2^2)
\]

\[
\bar{\sigma}_{ip}^* = (-2e_1 \cdot e_{17} + e_2 \cdot e_{16}) / (4e_1 \cdot e_3 - e_2^2)
\]

**TABLE 1 — THICKNESS RATIOS INCLUDED IN ANALYSES**

<table>
<thead>
<tr>
<th>Thickness of PC:</th>
<th>m</th>
<th>Thickness of PC:</th>
<th>m</th>
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<td>(h₁/h₂)</td>
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<td>(h₁/h₂)</td>
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<td>0.135</td>
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<tr>
<td>3.24</td>
<td>0.405</td>
<td>0.96</td>
<td>0.120</td>
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<tr>
<td>2.88</td>
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<tr>
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<tr>
<td>1.80</td>
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<td>0.180</td>
<td>0.24</td>
<td>0.030</td>
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<tr>
<td>1.20</td>
<td>0.150</td>
<td>0.12</td>
<td>0.015</td>
</tr>
</tbody>
</table>

a The thickness of the PCC (h₂) in all beam models is 8.0 in. (20.3 cm).

b The thickness of the adhesive layer (h₃) is 0.001 in. (0.0025 cm).
TABLE 2 — MODULAR RATIOS INCLUDED IN ANALYSES\textsuperscript{a,b}

<table>
<thead>
<tr>
<th>Elastic Modulus of PC : $E_1$ (psi)</th>
<th>$n$</th>
<th>Elastic Modulus of PC : $E_1$ (psi)</th>
<th>$n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>4,000,000</td>
<td>1.000</td>
<td>750,000</td>
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<tr>
<td>3,000,000</td>
<td>0.750</td>
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<tr>
<td>2,000,000</td>
<td>0.500</td>
<td>250,000</td>
<td>0.063</td>
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<tr>
<td>1,000,000</td>
<td>0.250</td>
<td>100,000</td>
<td>0.025</td>
</tr>
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</table>

\textsuperscript{a} Elastic modulus of the PCC ($E_2$) is 4,000,000 psi (27,560 MPa).

\textsuperscript{b} Elastic modulus of the adhesive layer ($E_3$) is 1,000,000 psi (6,890 MPa).

\begin{figure}
  \centering
  \includegraphics[width=\textwidth]{pc-pcc_composite_beam_model}
  \caption{PC-PCC composite beam model used in analyses of thermal stress}
\end{figure}
Fig. 2—Boundaries of Eigenvalues for Chen's equation (Eq. 1)
Fig. 3—Axial stress: Interface and extreme fiber stresses \((n = 0.25, m = 0.1)\)
DISTANCE = BEAM DEPTH

Fig. 4—Interface shear and normal stresses \( (n = 0.25, m = 0.1) \)
Fig. 5—Shear stress over the depth of a composite beam ($n = 0.25$, $m = 0.1$)
Fig. 6—Transverse normal stress over the depth of a composite beam \( n = 0.25, m = 0.1 \)
DISTANCE FROM END OF BEAM, $X_1$ (in.)

Fig. 7—Interface shear stresses versus thickness ratio ($r = 0.5$)
Fig. 8—Interface normal stresses versus thickness ratio \( (n = 0.5) \)
Fig. 9—Interface shear stresses versus modular ratio ($m = 0.06$)
Fig. 10—Interface normal stresses versus modular ratio ($m = 0.06$)
Fig. 11—Interface axial stress in PC ($\Delta e_1 = 500 \mu \text{ in./in.}, E_2 = 4 \times 10^6 \text{ psi}$)
Fig. 12—Maximum interface shear stress ($\Delta e_r = 500 \mu$ in./in. $E_2 = 4 \times 10^6$ psi)
Fig. 13—Maximum interface normal stress ($\Delta \epsilon_2 = 500 \, \mu \text{in./in.}, E_2 = 4 \times 10^6 \, \text{psi}$)
Fig. 14—Analysis versus test: axial stress ($\Delta T + .36\,^\circ F$)
Fig. 15—Analysis versus test: interface shear stress ($\Delta T = -36$ F)

Distance from end of beam, $X_1$ (in.)

Shear stress (psi)
Laboratory Study of Fatigue of Polymer-Modified Porous Concrete for Its Use as Top Layer of Concrete Pavements

by Miguel Angel Pindado, Antonio Aguado, Alejandro Josa, and Erik Onstenk

Synopsis: The use of porous materials as top-layers of pavements is currently increasing in several European countries due to their noise absorption effect and an improvement in the drainage properties of the pavement. These effects are considered essential because of environmental and safety reasons. In this context, porous concrete is being studied as an alternative to porous asphalt. Since the porosity of this material significantly reduces its strength, some additions, and in particular polymers, are required to obtain adequate mechanical properties and durability. These additions increase the cost of the pavement and, to counteract this, the thickness of porous material is reduced to a thin layer and a bottom dense concrete, bonded to the porous top, must be laid.

One such study was carried out by Dutch, German and Spanish companies within the scope of a research project funded by the European Commission. This project included the analysis of noise production mechanism and noise measurement, the study of the behaviour of porous concrete, the construction of test sections, the investigation of low noise by surface treatment and the assessment and establishment of a practice code and guideline for the design and construction of porous concrete pavements. With respect to the laboratory research on porous concrete, the main objective was the definition of several mix compositions and a study of their characteristics of behaviour.

This paper presents the results obtained in the fatigue testing programme carried out in this research project. It included compressive strength tests, the definition of the Wöhler curves (S-N diagrams) for several polymer contents and for different stress ratios, and the statistical analysis of experimental results.

Keywords: Fatigue (materials); laboratories; layered system; pavements; polymer concrete; tests
Miguel Angel Pindado is a Research Assistant in the Construction Engineering Department of the Universitat Politècnica de Catalunya (UPC), Barcelona, Spain. He has conducted several testing programs to study the fatigue of porous concrete, and has authored various papers in this field. The research reported here is part of his PhD Thesis.

Antonio Aguado is a Professor of Concrete Technology in the Construction Engineering Department and Sub-Director for Research in the School of Civil Engineering at the UPC. He has co-edited a book on concrete technology and co-authored several scientific and technical papers in this field. His current research interests are in high strength concrete, polymer concretes, technology transfer, and creep and shrinkage.

Alejandro Josa is an Associate Professor in the Geotechnical Engineering Department of the UPC. His research areas include the modelling of partially saturated soils, concrete behaviour and pavement design, and is co-author of several papers in these fields.

Erik Onstenk is a Civil Engineer at Intricon, Institute for materials and environmental research, Houten, The Netherlands. He has participated in several research programs on concrete behaviour and technology, and has authored many publications in these fields.

INTRODUCTION

The continuous growth of road traffic in Europe during the last decades has led to unacceptable levels of traffic noise and to high accident rates which the population is no longer prepared to accept. For this reason, the construction of quieter and safer roads, at an acceptable cost, is a main objective of road engineering research. In this context, porous materials are especially interesting due to their high porosity and permeability. Porous top-layers significantly reduce the noise produced by traffic (1) and the drainage properties of the pavement are improved due to the high permeability of the top-layer (2) which leads to a quick (vertical) drainage and reduces spray and splash of water. These effects are well-known in the case of porous asphalt which was developed and is an established procedure. Concrete pavements are a competitive alternative to asphalitic pavements, especially if their longer service life and low maintenance cost are taken into account. For these reasons, porous concrete for pavement top-layers began to be studied some years ago (3,4,5,6) and is now in an initial phase of study and experimentation.
Despite the fact that porous concrete has not been used as pavement top-layers, it has been used in other road applications taking advantage of its drainage properties. This is the case of its use in shoulders (7,8,9) or sub-bases and bases of concrete pavements (10,11,12). In these latter cases, porous concrete is part of the structural section of the pavement and its mechanical properties (strength, fatigue behaviour), have also been taken into account. Since porosity has opposite effects on permeability and strength (the higher the porosity the higher the permeability and the lower the strength), a compromise is required for a better overall design (2,13).

In top-layer applications, porous concrete is directly subjected to vehicle tires (possible pulling out of particles) and, in cold areas, to frost-thaw cycles and de-icing salt effects and, consequently, should be designed to resist such additional actions. Additionally, stresses in the top-layer are higher than in lower pavement layers. All this makes the design of this material more difficult, especially taking into account the need to reach a balance between permeability and strength (13,18). For this reason use of admixtures, in particular polymers, which increase concrete strength without reducing porosity, is necessary. These additions increase the cost of the pavement, but, to counteract this, it is advisable to reduce the amount of porous material used. This leads to structural sections composed of a thin (about 5 cm) porous concrete top and a conventional (dense) concrete bottom. An adequate bonding between both layers must be assured to get the desired pavement response.

The earlier discussions detail the motivations for the experimental study described in this paper which is part of the Brite-Euram project BE-3415 Optimization of the Surface Characteristics of Concrete Roads in Accordance with Environmental Acceptance and Traffic Safety, funded by the European Comission. Dutch (Cobeton, CUR and Intron), German (BASF, Hochtief, Technical University of Berlin and VDZ) and Spanish (Dragados, Geocisa and Technical University of Catalunya) companies were involved in the project.

The laboratory work on porous concrete was carried out by Intron, the Technical University of Catalunya and VDZ. The main objective of the laboratory programme was the determination of several mix compositions of porous concrete to fulfil the following conditions as well as to study the behaviour of these mixes. Those conditions include noise reduction and drainage properties comparable to those of porous asphalt; acceptable strength, stiffness and surface properties (e.g. skid resistance, evenness); minimum service life of 20 years (adequate bonding between porous and dense concrete, acceptable frost-thaw resistance and fatigue behaviour); construction possibilities using currently available techniques; and cost-effectiveness with respect to alternative materials, including design, construction and maintenance.

Since an important criterion was the easy application of these mixes in practical situations, three parallel studies were carried out, each one adapted to
the particular conditions (such as available sizes of aggregates, types of aggregates and cements) in the corresponding countries. The optimization of mix composition included a study of the following properties: workability, flexural, compressive, and splitting tensile strength, accessible porosity, permeability, frost-thaw resistance, and skid resistance. Once the three optimum mixes were defined, the following properties were studied: strength development with time, direct tensile strength, modulus of elasticity, fatigue behaviour, noise absorption, matrix structure, permeability and the effect of dirt input, de-icing behaviour, bond strength to dense concrete, ultraviolet (UV) resistance (only polymer-modified mixtures), and moisture resistance (only polymer-modified mixtures). Some of the results obtained have already been presented elsewhere (14,15).

The results from the fatigue tests on polymer-modified porous concrete with Dutch and Spanish specimens are presented in this paper.

FATIGUE TESTING

Fatigue of materials arise when they are subjected to successive cycles of loading and unloading up to stress levels lower than their static strength. These cycles progressively damage the material, which fails at stress levels below the static strength. The cycles can have different patterns of load evolution with time. Under actual service conditions the cycles have a stochastic pattern. A specific successively repeated pattern (see figure 1) is often used in the laboratory. This pattern is defined by its periodic shape (usually by the maximum and minimum stresses, $f_{\text{max}}$ and $f_{\text{min}}$, and the type of variation between them), and the frequency.

The usual way to characterize the fatigue behaviour of materials for pavement design, is the definition of the Wöhler curves ($S-N$ diagrams). These curves relate the ratio between the maximum stress in the test cycles ($f_{\text{max}}$) and the (static) strength of concrete ($f_{\text{c}}$), $S=f_{\text{max}}/f_{\text{c}}$, with the number of cycles to failure, $N$. The curves obtained vary depending on the type of test (compression; flexural), and the pattern of the cycles and its frequency. Consequently, it is advisable to adapt these factors as much as possible to the real loading conditions. Such results would provide a realistic fatigue model of the material.

Vehicles induce flexural stresses in the concrete slabs and, consequently, flexural tests are in principle the most appropriate configuration to establish the Wöhler curves for the slab as a whole. However, since the slab is not homogeneous in this case (porous top and dense bottom), it can be considered that the porous concrete will only be subjected to compressive stresses corresponding to the upper portion of the flexural gradient. In some circumstances (e.g. with high thermal gradients) the upper part of the pavement can be subjected to tension. However, the latter case is not frequent. Consequently, compression tests were carried out for obtaining the Wöhler curves
for porous concrete. This test configuration has the additional advantage of being more reproducible than the flexural configuration.

Since the effect of traffic on a section of the pavement can usually be modelled with a quick loading from 0 (when the vehicle arrives), immediately followed by a quick unloading to 0 (the higher the velocity of the vehicle the faster the loading and unloading), a very low stress ratio, \( R = f_{\text{min}} / f_{\text{max}} \), and a high frequency represent realistic parameters for the laboratory simulation. With rapid loading/unloading the shape of the cycle between \( f_{\text{min}} \) and \( f_{\text{max}} \) is not important.

**Porous Concrete Mixes**

Previous results from the Netherlands and Spain led to the following initial requirements for porous concrete (2,4,13,16,17,18): maximum size of the aggregate of about 10 mm; use of a gap-graded aggregate material; accessible porosity of 15 to 25 % (V/V); workability of fresh concrete corresponding to a compaction factor of 1.20±0.02 according to DIN 1048; a minimum compressive strength of 15-20 MPa after 28 days of cure; a minimum flexural tensile strength of 3.0 MPa after 28 days of cure; and a minimum splitting tensile strength of 2.0 MPa after 28 days of cure (all the tests conducted in accordance with ISO standards). Several reference mixes fulfilling these requirements were defined in the first part of the project testing programme (14,15). This study included the type and level of polymer. The studied polymers were a pure acrylic copolymer, a styrene-acrylic copolymer, a styrene-butadiene copolymer and emulsified epoxy resin. Using strength, durability and cost criteria, the acrylic copolymer was selected. The study also examined other additives, like superplasticizers, stabilizers and silicafume. The results from these studies and, in particular, the reference mixes finally selected, were used in defining the fatigue testing programme.

**Materials**

The following materials were used for porous concrete manufacture:

**Coarse aggregate:**
- Crushed 4-8 mm (Dutch specimens)
- Crushed 5-12 mm (Spanish specimens)

**Sand:**
- River 0-1 mm (Dutch specimens)
- Crushed 0-5 mm (Spanish specimens)
Cement: Portland fly-ash cement \textit{CE II-C 32.5R} (Dutch specimens) Portland fly-ash cement (40% fly-ash content) (Spanish specimens)

Polymer: Pure acrylic copolymer (latex VF-774, Forton BV, DSM group).

The different grain sizes of the Dutch and the Spanish mixes result from the need to use commercially available aggregates in each country. Only siliceous aggregate was selected as the coarse aggregate to assure an adequate polishing resistance to traffic loads on the pavement surface.

The main characteristics of the polymer used are as follows:

- Concentration (by mass) \hspace{3cm} 51 \% solids \hspace{3cm} 49 \% water
- Appearance \hspace{3cm} milky white
- Odour \hspace{3cm} mild
- Viscosity at 25^\circ C \hspace{3cm} 150 \text{ mPa.s}
- Acidity (acid value or pH) \hspace{3cm} 4,5-5,5
- Minimum Filmforming Temperature (MTF) \hspace{3cm} 7^\circ C
- Size polymer particles \hspace{3cm} 150-200 \text{ nm}
- Density (solids) at 20^\circ C \hspace{3cm} 1136 \text{ Kg/m}^3
- Working temperature range \hspace{3cm} 5<T<40^\circ C

Mix composition

Three different mixes were used (see Table 1). Two of these mixes contained different amounts of polymer solids (14.2 Kg/m$^3$ in the Spanish mix and 25 Kg/m$^3$ in the Dutch mix) and the third one did not contain polymer. These mixes approximately correspond to the reference mixes previously mentioned.

Polymer was supplied as a latex with a solids content of 51 \% by mass (the amount of polymer in table 1 correspond to polymer solids). In the case of the mix without polymer, a superplasticizer was added to reach approximately the same workability as in the polymer-modified mixes.

Production and curing

Fresh porous concrete was produced in a pan mixer using a mixing time of 1.5 minutes after adding all the constituent materials. The cylindrical specimens were 15 cm in diameter, 30 cm in height and were compacted by vibration. Mixes with polymer were cured the first day in the mould at 20 \textdegree C and
subsequently they were stored at 20 °C and 65 % R.H. (Dutch mix) or at 20 °C and 50 % R.H. (Spanish mix). The unmodified mixture was cured the first day in the mould at 20 °C and subsequently cured at 22.5 °C and 98 % R.H. These curing procedures are the usual ones in the corresponding countries.

TESTING PROGRAMME

The main characteristics of the testing programme were as follows:

- Study of porous concrete behaviour for high number of cycles according to Hsu classification (1,000 to 10,000,000) (19). This interval of cycles correspond to the usual cases in pavement design.

- \( R = \frac{f_{\text{min}}}{f_{\text{max}}} = 0.05 \) (\( f_{\text{min}} \) very low; \( f_{\text{min}} = 0 \) cannot be applied). As stated before, this scenario is the usual case for traffic loads. Wöhler curves were determined also for \( R = 0.2 \) and \( R = 0.5 \) in the case of the polymer-modified Spanish mix. Despite the fact that these latter values of \( R \) are not realistic in the case of traffic loads and that they are not used for pavement design, the corresponding results were useful to study possible variations of polymer behaviour when load cycles change.

- Frequency of cycles was 10 Hz for \( R = 0.05 \) and 15 Hz for the other values of \( R \). Pattern of cycles used is shown in Figure 1. All tests (static and dynamic) were conducted in an INSTRON-digital 8500 servo-hydraulic machine.

Each Wöhler curve was in general defined using 5 points between 0.60 and 0.90. The high number of individual tests for every \( S-N \) curve is needed due to the usual (high) scatter in results from fatigue tests. A statistical analysis was carried out to determine the confidence band of the \( S-N \) curves. A static test was carried out to obtain the strength of the corresponding specimens \((f'_c)\), prior to carrying out the fatigue tests, because fatigue tests are lengthy and concrete continues to cure (especially with fly ash cements), it is advisable to use an updated \( f'_c \) for each group of individual tests.

RESULTS AND DISCUSSION

Monotonic static tests for the compressive strength were conducted to serve as the reference for the cyclic fatigue tests. 5 tests were carried out for each \( S-N \) curve. The mean static strengths for the different concretes were:

[1] Spanish specimens with polymer: 20.6 MPa
[2] Spanish specimens without polymer: 26.8 MPa
[3] Dutch specimens with polymer: 23.2 MPa

These differences can be attributed to the variation in polymer percentages and aggregate gradings used in [1] and [3], and to the different type and percentage of the fines used in [1] and [2].

Fig. 2 to 8 show the results obtained. Fig. 2 and 3 compare several S-N curves and Fig. 4 to 8 show the individual results and the corresponding S-N curves and confidence bands. Each point in Fig. 4 to 8 corresponds to an individual fatigue test, including some that did not fail even at $2 \times 10^6$ cycles which are plotted in the figures as points with arrows.

Composition 1 (Spanish specimens with polymer) was used to study the influence of the $R$ value on the fatigue behavior of polymer modified porous concretes. Figure 2 shows the S-N curves for this concrete with respect to changes in the $R$-value ($R=0.5, 0.2, 0.05$). The last value represents practical traffic conditions. This figure confirms that the fatigue behavior of polymer-modified concrete and unmodified concrete are similar, with the fatigue strength increasing with an increase in the value of $R$. That is to say, for a constant value of $f_{\text{max}}$, if the amplitude of the loading increases, the fatigue strength of porous concrete decreases.

For small values of $N$ one can see that $S$-N curves converge to values of $S$ slightly larger than 1, which is attributed to the static, not dynamic, compressive strength reference test.

The fatigue behavior (S-N curves) is presented using straight lines. The slope of these lines can be expressed as $\beta \cdot (1 - \alpha \cdot R)$ where $\beta$ and $\alpha$ are concrete constants (20) (See figure 2). For Spanish polymer-modified porous concrete, the value of these constants are, respectively: 0.0714 ($\beta = 0.0714$) higher than usual for standard concrete ($\beta = 0.0685$) (21) and 0.6 ($\alpha = 0.6$) lower than usual for standard concrete ($\alpha = 1.0$).

Finally in the zone of highest values of $N$, it can be seen that when $R$ increases, $S$ increases too until, it can be supposed, values that for $R=1$ ($f_{\text{min}}=f_{\text{max}}$) will coincide with the values associated to the unfavourable effect of a sustained load.

Figure 3 shows S-N curves obtained for $R=0.05$ corresponding to mixes 1, 2 and 3. This figure shows that the addition of the polymer enhances the fatigue performance of porous concrete. Thus, for a fixed value of $S$ ($S=f_{\text{max}}/f_{\text{c}}$), the fatigue life ($N$) corresponding to a porous concrete with polymer (mixes 1 and 3) is improved by approximately an order of magnitude compared to the porous concrete without polymer (mix 2). This improvement is not consistent in all of the tested range (the regression lines in figure 3 are not parallel). For small
values of $N$ the improvement is maximum.

In the high number of cycle ($N$) regime, the difference between the values of $S$ of each composition, for a fixed $N$, is small, e.g., for $N=2\times10^6$ cycles this difference is approximately $0.04x\sqrt{f_c}$. In practice this difference is negligible.

This improvement in the fatigue performance of porous concrete due to polymer addition, higher in the zone of high values of $S$ (low number of cycles) and decreasing with $N$, is also observed in other kinds of concrete with polymer and in other kinds of loadings. An example is the behavior of plain polymer concrete (PC) beams to flexural fatigue (22).

Figure 3 also includes the $S-N$ curve proposed by Tepfers and Kutti (21) for plain, ordinary and lightweight concrete ($R=0.05$). It can be seen that the $S-N$ curves obtained for dense and porous concrete, with and without polymer, are different. Consequently, it is not possible extrapolate $S-N$ curves corresponding to dense concrete to characterize the fatigue behavior of porous concrete.

The differences found in the fatigue behavior between mixes 1 and 3, both with polymer, are very small (for $N=1000$ cycles $\Delta S=0.04x\sqrt{f_c}$ and for $N=10^6$ cycles $\Delta S=0$). These differences are attributed to the different proportion and grading of the aggregates in the two mixes (see Table 1).

Analytical expressions

$S-N$ curves mentioned above are given by the following analytical expressions where $r$ is the coefficient of correlation:

- Spanish polymer-modified porous concrete (mix 1):

  $S = 1.058 - 0.050 \cdot \log N \quad R = 0.5 \quad r = 0.947 \quad (1)$

  $S = 1.084 - 0.065 \cdot \log N \quad R = 0.2 \quad r = 0.958 \quad (2)$

  $S = 1.080 - 0.069 \cdot \log N \quad R = 0.05 \quad r = 0.841 \quad (3)$

- Spanish conventional porous concrete (mix 2):

  $S = 0.920 - 0.050 \cdot \log N \quad R = 0.05 \quad r = 0.806 \quad (4)$
- Dutch polymer-modified porous concrete (mix 3):

\[ S = 1.003 - 0.057 \cdot \log N \quad R = 0.05 \quad r = 0.834 \]  

Additionally, a statistical analysis of each S-N curve was carried out. The confidence bands for a 90\% confidence level were determined. Figures 4, 5 and 6 show narrow confidence bands of the mean for the mix 1 with \( R \) changing from \( R=0.5 \) to \( R=0.05 \). Figures 7 and 8 show confidence bands corresponding to mixes 2 and 3 for \( R=0.05 \). The last two confidence bands are wider than that obtained for \( R=0.05 \) for mix 1 (see figure 4).

CONCLUSIONS

The following conclusions can be made based on the study of the behavior of polymer-modified porous concrete to compressive fatigue stress:

- Fatigue behaviour of polymer-modified porous concrete can be modeled adequately with a linear relationship between \( S \) and \( \log N \) (Wöhler curve). The slope of these straight lines depends on the value of \( R \) in a manner similar to that of dense cement concrete but with different values.

- Polymer-modified porous concrete exhibits a better behavior than porous concrete without polymer for a low number of fatigue cycles. This difference decreases when the number of fatigue cycles increases. For a high number of cycles, which corresponds to the case of actual traffic, the difference in fatigue performance is negligible.

- Polymer-modified porous concrete exhibits a better fatigue performance than conventional dense concrete. Consequently the Wöhler curves for dense concrete are not adequate for the design of polymer-modified porous concrete. It is hence necessary to establish specific S-N curves for polymer-modified porous concrete.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the financial support of the European Comission through the Brite-Euram project BE-3415 entitled "Optimization of the Surface Characteristics of Concrete Roads in Accordance with Environmental Acceptance and Traffic Safety". Also, the first author acknowledge to Comisión Interministerial de Ciencia y Tecnología (Spanish Goverment) the grant given for his PhD thesis, included in the CICYT project MAT93-0293.
REFERENCES


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1: Spanish specimens with polymer
2: Spanish specimens without polymer
3: Dutch specimens with polymer

Fig. 1—Cyclic loading pattern
Fig. 2—S-N curves of Spanish polymer-modified porous concrete (mix 1) for three values of $R$

Fig. 3—S-N curves of polymer-modified porous concrete, porous concrete without polymer and plain concrete for $R = 0.05$
Fig. 4—90 percent confidence limits of S-N curve of Spanish polymer-modified porous concrete for $R = 0.5$.

Fig. 5—90 percent confidence limits of S-N curve of Spanish polymer-modified porous concrete for $R = 0.2$. 

Fig. 6—90 percent confidence limits of S-N curve of Spanish polymer-modified porous concrete for $R = 0.05$

Fig. 7—90 percent confidence limits of S-N curve of Spanish porous concrete without polymer for $R = 0.05$
Fig. 8—90 percent confidence limits of S-N curve of Dutch polymer-modified porous concrete for $R = 0.05$
A New Family of Latexes for Portland Cement

by Lou A. Kuhlmann

**Synopsis:** A new family of latexes has been developed for use in portland cement that has a minimum film-formation temperature (MFFT) well above working temperature, eliminating the two major drawbacks of latex-modified mixtures: formation of a crust on the surface, and difficulty in cleaning tools. Instead of coalescing to form a film, as do the typical latex modifiers for portland cement, these latex particles maintain their shape as spheres. Of the several formulations studied, two are reported here: a styrene polymer and a methyl methacrylate polymer - both carboxylated. In addition to extensive laboratory testing of both polymers, two field trials with the styrene latex formulation were conducted. These laboratory and field studies demonstrated that film formation is not necessary for latexes to contribute to the performance of portland cement mixes. The data from these studies are encouraging, but also revealed that much more work needs to be done to fully understand the capabilities and limitations of this family of latexes.

**Keywords:** Latex; methyl methacrylate; permeability; polymer concretes; portland cement; styrene; temperature
Lou Kuhlmann is a Development Associate with The Dow Chemical Company, specializing in latex modified mortar and concrete. He has a B.Sc. in Chemical Engineering and M.Sc. in Engineering Mechanics from the University of Missouri at Rolla. He is a member of ACI, ASTM, TRB, where he has published frequently.

INTRODUCTION

Latexes currently used for modifying portland cement contribute many property improvements to mortar and concrete. Many different polymers are used to make these latexes, but all have one thing in common: a minimum film-formation temperature (MFFT) less than 50°F, meaning that they form a film at normal working temperatures. It is the formation of this film that is generally understood to be critical to the performance of these latexes in portland cement and responsible for the property improvements in the hardened mortar and concrete. This film-formation is also the cause of the two drawbacks associated with latex-modified mixtures - crusting of the surface as it dries and difficulty in cleaning tools and equipment.

A new family of latexes has been developed for use in portland cement that has a MFFT considerably above room temperature. The result is a latex-modified mixture that does not form a film at working temperatures, thus overcoming the crusting and cleaning problems. Thus, mixtures with these latexes are easier to finish than the normal latex-modified mixtures. In addition, use of these materials in ready-mixed concrete, a feature not practical with normal latexes, has been demonstrated. The mortar and concrete properties - workability, air content, compressive strength, bond strength, permeability and durability - were equivalent to, or better than, quality concrete. One formulation had several properties that were better than normal latex-modified mixtures.
LABORATORY PROCEDURES

The following test methods were used in this study:

A. Mortar

1. Workability - Flow, ASTM C 230
2. Compressive strength - ASTM C 109
3. Air Content - ASTM C 185, paragraph 10

B. Concrete

1. Slump - ASTM C 143
2. Air Content - ASTM C 231
3. Compressive strength - ASTM C 39
5. Flexural strength - ASTM C 78
6. Permeability - ASTM C 1202
7. Freeze/Thaw - ASTM C 666, Procedure B

The mixture proportions and curing schedule are given in Table 1. (This curing schedule will be identified as 'normal' in this paper.) Exceptions are noted on the tables and graphs. The air contents shown in Figures 2 & 8 were calculated from mortar density, according to ASTM C 185. Table 2 lists the properties of the two experimental latexes and the styrene-butadiene (S/B Control) latex used in this study. (Use of the word 'latex' in the terms 'latex content' and 'latex/cement', means the latex polymer, or solids.)

EXPERIMENTAL STYRENE LATEX

ACI's State-of-the-Art Report on Polymer-Modified Concrete (1) describes what occurs when particles in film-forming latexes change from the emulsion to a film in cured portland cement mixes. This film is visible in photographs taken at high magnification. Similar photographs of cured mortar made with a non film-forming latex clearly show the
latex particles as discreet, individual spheres, just as they were in the original emulsion. Figure 1A shows the smooth, as cast surface of the sample; Figure 1B the fractured surface, after the sample was tested in compression. Of particular interest in the photographs of Figure 1B is the apparent distortion of the particles, highlighted by the bright spots on the tips of the particles. This suggests the possibility of chemical bonding - that the particles may adhere to surfaces within the mortar matrix and distort when stressed.

Mortar mixtures at different latex-cement ratios and constant water-cement ratio were evaluated for effect on compressive strength. The results (Figure 2) show an increase in compressive strength as latex content decreases. Of significance is the dramatic decrease in workability of the mortar at the lowest latex-cement ratio, 0.05, suggesting that this level would not make a usable mixture without an increase in water content. The flow and compressive strength at 0.15 latex-cement were comparable to the S/B Control at the same latex content.

Figure 3 shows weight loss versus curing time for two-inch cubes of mortar made with the experimental styrene latex compared to the S/B Control. The samples were cured normally. These data indicate that the lack of film formation by the experimental styrene latex permitted a higher loss of water during the first day of being exposed to air; after that the weight loss rates of the two mixtures were the same.

Properties of four concrete mixtures prepared with the experimental styrene latex are shown in Table 3. Three were made in the laboratory; one was prepared in the field in a ready-mix truck for placement of a sidewalk. Mixtures A and B were cured normally. (Table 4 contains properties of two concrete mixtures made with the S/B Control latex.) All mixtures had similar proportioning. Figure 4, compressive strength versus curing time of Mixtures A and B, shows both of the experimental styrene latex mixtures yielded higher compressive strengths than the S/B Control, at all of the curing times studied. Figure 5 is a plot of the tensile bond
strength of Mixtures A and B, compared to S/B Control. Here the Control yielded higher strengths than both of the experimental mixtures. Figure 6 shows the shrinkage properties of Mixture B to be comparable to that of the Control, both cured at the normal schedule. The concrete made with the experimental latex had shrinkage properties similar to the S/B Control mixture.

Michigan State University Laboratory Study

An evaluation of the experimental styrene latex in concrete was conducted at the Civil Engineering Department of Michigan State University in Lansing, Michigan. The study evaluated two different latex-cement ratios and two different curing schedules. In addition to plastic properties, the compressive, bond, flexural, impact, abrasion, and chloride permeability properties were measured. Of particular interest was the positive effect of wet curing on the concrete modified with the experimental styrene latex.

Field Trials

Two field trials were conducted with this experimental latex. In June, 1991, at a chemical plant in Midland, MI, a sidewalk was poured with ready-mixed concrete. No problems were encountered with the mixing or placement of this material. Four cores were taken from the sidewalk after 9 months of outdoor exposure and tested for permeability using ASTM C 1202. The cores yielded an average value of 1066 coulombs. Other properties of this concrete are shown in Column D of Table 3.

In May, 1992, a sidewalk in downtown Chicago was poured with ready-mixed concrete utilizing this experimental styrene latex. The contractor\(^1\), who placed 80 yd\(^3\) of this modified concrete at the Rookery Building, reported that the installation and finishing went very well.

\(^1\) Camp Flooring & Construction Co., Inc.
7705 W. 99th St., Hickory Hills, Illinois
EXPERIMENTAL METHYL METHACRYLATE LATEX

Mortar

The other non film-forming latex evaluated in portland cement mixes was a carboxylated methyl methacrylate polymer. Table 5 lists the compressive strength of mortar made with this latex at various latex-cement ratios, and with several brands of portland cement. Figure 7 is a plot of compressive strength of 4 latex levels for one brand of Type I cement, compared to the S/B Control latex at 0.15 latex-cement ratio. All of the mixtures with the experimental latex yielded higher compressive strengths than the Control at all of the curing times studied.

Figure 8 shows the effect of type of curing and air content on compressive strength of mortar mixtures made with this latex at latex solids/cement of 0.15. Two different air contents were achieved by adjusting the level of antifoam in the latex. This plot shows:

1. The higher air content significantly decreased compressive strength

2. Air curing after 7 days of wet curing increased compressive strength compared to continuous wet curing for both air contents studied.

Abrasion Resistance -- The abrasion resistance of mortar made with the experimental methyl methacrylate latex was compared to mortars made with the S/B Control latex and with silica fume. Both latexes demonstrated equivalent abrasion resistance in this test; the mortar made with silica fume exhibited the poorest abrasion resistance. The experimental latex had the highest compressive strength of the three mortars tested.

An evaluation of the abrasion resistance of the methyl methacrylate latex formulation, XU31064.50, was made using
procedures similar to ASTM C131, "Standard Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine". Three mortar mixes were evaluated, two made with latexes (XU31064.50 and Control) and one made with silica fume. The mixture proportions of the mortar were constant: concrete sand-cement ratio of 3.25; water-cement ratio of 0.37; admixture-cement ration of 0.10. Details are given in Table 6.

The specimens were 2-inch mortar cubes, cured 1 day covered and in the molds, 27 days at 50% R.H., 72°F. The test equipment was a 24-inch diameter horizontal revolving drum, rotating at 20 rpm, with 4 internal baffles that would lift and tumble the specimens.

Compressive strength was determined at 1, 7 and 28 days. At 28 days, each abrasion specimen was identified and weighed. Six cubes of each mix were placed inside the drum, along with eighteen round stones approximately 1 to 1.5 inch in diameter to accelerate abrasion, and the drum rotated for 25 minutes (500 revolutions). The cubes were then removed, brushed to remove loose particles, weighed and the average weight loss based on original weight for each mix calculated. This procedure was repeated at intervals of 500 revolutions until the samples had been exposed to a total of 2000 revolutions. The average weight loss for each mix at 500 revolutions intervals is plotted in Figure 9; the total weight loss for each mix is shown in Table 6.

Of the three tested, the silica fume mortar experienced the highest weight loss while the mortar made with the XU31064.50 latex demonstrated an abrasion resistance similar to that of the mortar made with Control. This was true at all intervals of the abrasion testing. The XU31064.50 modified mortar demonstrated that it is possible to have a mortar with high compressive strength as well as good abrasion resistance.
Concrete

Several concrete mixtures were made with Lafarge Type I cement and this latex. Table 7 lists the properties of these concrete mixtures compared to a similar mixture made with the S/B Control latex. Two latex-cement ratios, 0.15 and 0.10, were evaluated in this series. This study revealed that even at the lower latex content, high slumps and high compressive strengths were achieved with the methyl methacrylate latex. Figure 10 is a plot of compressive strength versus curing time for the three concrete mixtures made with Lafarge cement and the experimental latex, compared to concrete made with the S/B Control. It is clear that this experimental latex produces concrete of significantly higher compressive strength than the S/B Control. Another unique feature of concrete made with the experimental latex at 0.10 latex solids/cement was the excellent workability, as shown in the slump loss curve (Figure 11). Here, concrete made with this latex is compared to several different concretes made with S/B Control, at 0.15 latex solids/cement (3, 4) and different water-cement ratios (0.40; 0.30). All three had initial slumps in the range of 4 - 6 inches. Concrete made with the MMA latex (water/cement of 0.37) had the unique combination of initial high slump (11 inches) and long working time (fifty percent slump loss after 2 hours and 10 minutes), plus high compressive strength (4860 psi) at 24 hours.

Other properties of this latex-modified concrete, shown in Table 6, indicate that bond strength, flexural strength, and permeability, are comparable to the S/B Control. It was noted that the 28 day permeability value for the S/B Control concrete in this table was higher than the value reported for the Control in Table 4. The reason for this is not known. The composition of the two mixtures was identical except for the brand of cement.

Shrinkage was studied at different curing schedules and compared to the S/B Control. Figure 12 shows that with a normal curing schedule of only 1 day in the mold, followed by air curing, shrinkage is greater than the S/B Control.
However, when concrete made with the MMA latex is kept damp for the initial 7 days and then air cured, shrinkage is reduced significantly. More shrinkage studies are necessary to fully understand this property of the experimental latex.

CONCLUSION

It has been demonstrated by both laboratory and field mixtures that film formation is not necessary for latexes to contribute to the performance of portland cement mixes. Laboratory studies of portland cement mortar and concrete made with two experimental latex formulations indicate that property improvements can be achieved. Although these data are encouraging, much more needs to be done to fully understand not only the capabilities but also the limitations of this family of latexes.

REFERENCES


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Normal curing schedule for latex-modified mixtures was:
1 day covered, in molds; remaining time at 50% relative humidity, 72°F
### Table 2 — Properties of the Latexes

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<th>Polymer</th>
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*by Fisher Sand & Gravel, Midland, MI **remainder @ 50% R.H., 72°F ***no scaling
### TABLE 4 — PROPERTIES OF CONCRETE WITH S/B CONTROL LATEX

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**Properties:**

- **Slump, in:**
  - 10.5
  - 10

- **Air content, %:**
  - 4.8
  - 4.7

**Compressive, psi**

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**Tensile Bond, psi**

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**Flexural Strength, psi**

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**Permeability, coulombs**

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**Freeze/Thaw, Cycles**

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<td>H</td>
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<th>28 day</th>
<th>Weight Loss, %</th>
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<td>XU31064.50</td>
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<td>145</td>
<td>8.3</td>
<td>6085</td>
<td>9300</td>
<td>9730</td>
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</table>

Cubes were cured for 1 day covered, in molds; remaining time at 50% R.H., 72ºF

Aggregate was ASTM C33 natural sand; cement was Lafarge, Type I

Mixture Proportions:
- sand/cement = 3.25
- admixture/cement = 0.10
- water/cement = 0.37

*included 2.5% water reducer; 0.002% air-entraining agent
### TABLE 7 — PROPERTIES OF CONCRETE WITH EXPERIMENTAL MMA LATEX

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<thead>
<tr>
<th>Mixture:</th>
<th>O</th>
<th>P</th>
<th>Q</th>
<th>R</th>
<th>S</th>
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<tr>
<td></td>
<td>Lafarge</td>
<td>Lafarge</td>
<td>Lafarge</td>
<td>Hercules</td>
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<td>1.00</td>
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<td>1.00</td>
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<tr>
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<td>1.7</td>
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<td>1.7</td>
<td>1.7</td>
<td>1.7</td>
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<td>2.6</td>
<td>2.6</td>
</tr>
<tr>
<td>latex solids</td>
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<td>0.10</td>
<td>0.10</td>
<td>0.15</td>
<td>0.15</td>
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</tr>
<tr>
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<td>0.36</td>
<td>0.37</td>
<td>0.37</td>
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<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
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<td>Slump, in</td>
<td>11</td>
<td>11</td>
<td>10.2</td>
<td>11</td>
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<td>Air content, %</td>
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<td>4.1</td>
<td>5.3</td>
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<td><strong>Curing</strong></td>
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<td>normal</td>
<td>7d. wet**</td>
<td>normal</td>
<td>normal</td>
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<tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1 day</td>
<td>5225</td>
<td>4860</td>
<td>5300</td>
<td>4610</td>
<td>3505</td>
<td></td>
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<tr>
<td>7 days</td>
<td>7320</td>
<td>7775</td>
<td>7955</td>
<td>7060</td>
<td>5635</td>
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<td>28 days</td>
<td>9500</td>
<td>9980</td>
<td>10,040</td>
<td>8570</td>
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<td></td>
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<td></td>
<td></td>
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<tr>
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<td>380</td>
<td>313</td>
<td>---</td>
<td>---</td>
<td>273</td>
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<tr>
<td>7 days</td>
<td>388</td>
<td>434</td>
<td>---</td>
<td>---</td>
<td>446</td>
<td></td>
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<tr>
<td>28 days</td>
<td>423</td>
<td>391</td>
<td>---</td>
<td>---</td>
<td>417</td>
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<td><strong>Flexural Strength, psi</strong></td>
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<td>28 days</td>
<td>725</td>
<td>--</td>
<td>901</td>
<td>---</td>
<td>994</td>
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<tr>
<td><strong>Permeability, coulombs</strong></td>
<td></td>
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<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>28 days</td>
<td>1665</td>
<td>2111</td>
<td>1917</td>
<td>2035</td>
<td>1653</td>
<td></td>
</tr>
</tbody>
</table>

**remainder of time at 50% RH, 72°F**
Fig. 1—Micrographs of latex particles; a) Cast surface of cured modified mortar, b) fractured surface of cured modified mortar at 50,000 X
Fig. 1—Micrographs of latex particles; a) Cast surface of cured modified mortar, b) fractured surface of cured modified mortar at 50,000 X
Fig. 2—Effect of latex level on the compressive strength of mortar made with experimental styrene latex

Fig. 3—Weight loss of mortar cubes made with experimental styrene latex
Fig. 4—Compressive strength of concrete made with experimental styrene latex

Fig. 5—Tensile bond strength of concrete made with experimental styrene latex
Fig. 6—Shrinkage of concrete made with experimental styrene latex

Fig. 7—Effect of latex level on compressive strength of mortar made with experimental MMA latex
Fig. 8—Effect of cure schedule on compressive strength of mortar with two air contents made with experimental MMA latex

Fig. 9—Weight loss of mortar cubes subjected to abrasion
Polymers in Concrete

Compressive Strength, psi

Cure time, days

Mixture
O
P
Q
S/B Control - S

Fig. 10—Compressive strength of concrete made with experimental MMA latex

% of Initial Slump

Fig. 11—Slump loss of concrete made with experimental MMA latex
Fig. 12—Shrinkage of concrete at different cure schedules made with experimental MMA latex
Polymer-Modified Concrete Overlays on Industrial Asphalt Floors

by Johan Silfwerbrand

Synopsis: Industrial floors of asphalt concrete or other bituminous products are deformed under sustained concentrated loads. They are also dark in colour and difficult to clean. Consequently, they need to be renovated. The use of polymer-modified concrete (PMC) overlays is an interesting alternative. Reinforced and unreinforced overlays were subjected to static and rolling wheel loads. Reinforced PMC overlays on asphalt showed a high load-carrying capacity. Shrinkage tests were carried out on PMC prisms and on concrete and bituminous prisms overlaid with PMC. A two-layer overlay with wear and levelling layers was less prone to shrinkage than an overlay solely consisting of a wear layer.

Keywords: Asphalts; floors; loads (forces); polymer concretes; resurfacing; shrinkage
BIOGRAPHICAL SKETCH

ACI member Johan Silfwerbrand is currently associate professor of Structural Engineering at the Royal Institute of Technology in Stockholm, Sweden. He received his Ph.D. in Civil Engineering from the Royal Institute of Technology in 1987. His research interests are in repair, pavements, overlays, and steel fibre reinforced concrete. He is a consulting member of ACI Committee 345, Bridge Construction and Maintenance, and 342, Evaluation of Concrete Bridges and Bridge Elements, an associate member of 546, Repair, and a member of CEB Task group 3/3, Assessment and Redesign of Existing Reinforced Concrete Structures.

INTRODUCTION

A large number of industrial floors are made of asphalt concrete or other bituminous materials. They are dark in colour, sensitive to oil or similar liquids and difficult to clean. The deflection under sustained concentrated loads is high. Consequently, many such floors need to be renovated. Conventional methods of renovation remove the old floor and replace it with a better alternative. The removal is, however, expensive. Placing an overlay on top of the old floor is a more economic solution, if it works structurally.

Self-levelling polymer-modified Portland concrete (PMC) has been used successfully to renovate deteriorated concrete floors (1, 2, 3). A Swedish approach is to make a two-layer overlay. The top layer has a high resistance to wear. The second layer is made of a less expensive material and used for levelling.

A PMC overlay on an asphalt floor leads to two complications. First, wide shrinkage cracks may occur because of low friction forces between the overlay and the soft asphalt layer. Second, the load carrying capacity of the overlay may be too low. A PMC overlay on asphalt concrete acts similarly to a slab on grade (2, 3). A soft grade increases the risk of flexural and punching failures.

A theoretical solution to solve these problems is to reinforce the PMC overlay. A welded-wire fabric ought to reduce the crack widths and improve the load-carrying capacity.

RESEARCH SIGNIFICANCE

A large number of bituminous industrial floors need repair. The use of PMC overlays is an interesting renovation method. Problems with shrinkage cracks and
heavy wheel loads might, however, arise. The research reported herein provides information on methods to solve these problems.

PREVIOUS TEST

Three circular slabs with unreinforced PMC overlays were subjected to wheel loading in a previous study (2, 3). The wheel was made of a hard plastic material. The wheel diameter was 250 mm and the wheel width was 86 mm. According to the truck manufacturer, the wheel should not carry more than 25 kN. Overloads up to 50 per cent above this level were, however, judged to occur in rare cases. The overlay consisted of two layers with thicknesses of 5 and 20 mm, respectively. On two slabs, the overlay was placed on top of 300 mm concrete. On the third one, the overlay was placed on top of 100 mm asphalt concrete. The first two slabs were both subjected to the intended maximum load of 40 kN without any cracking. The overlay on asphalt failed, however, at 25 kN.

TEST PROGRAMME

In order to study the behaviour of reinforced PMC overlays on asphalt floors, fourteen overlays were cast and subjected to wheel loading. Some overlays were subjected to static loads, others were loaded in fatigue, and some overlays were loaded by a constant wheel load and simultaneously subjected to a horizontal fatigue movement. The latter loading intended to simulate the truck traffic on the industrial floor. This paper only deals with static loading and the truck traffic simulating loading. Results from the complete programme are given in the test report (4).

Some tests were also conducted to study shrinkage. These tests covered measurements of free shrinkage of polymer-modified concretes used for wear and levelling layers and shrinkage measurements on PMC overlays on small bituminous and concrete specimens. They will be discussed briefly below.

TEST SPECIMEN PREPARATION

All specimens were square with the side length of 1 m. Each consisted of a PMC overlay on top of a 100 mm thick asphalt concrete layer and a 100 mm thick concrete layer (Fig. 1). The concrete formed a base for the specimen and a necessary bottom for the asphalt placement. Eight concrete slabs were placed in a row facilitating the placement of the asphalt concrete. After placement, the continuous asphalt layer again was cut into the eight test specimens. Due to economical reasons, six of the slabs were used as support for the PMC overlay twice. In these cases, the first overlay was removed and replaced by a new overlay. Thus the test series consisted of two parts. Overlays in the first part were
labelled with Arabic numerals, overlays in the second part were labelled with capital Roman letters.

The PMCs used were all self-levelling. They were delivered as a pre-mixed dry concrete containing quartz sand, Portland cement, special cement, gypsum, filler, polymers (of vinyl-acetate basis) and admixtures. An amount of 15 to 20 per cent water was added to the mix. Physical and mechanical properties are given in Table 1. Usually, the overlay was made in two layers: a wear layer and a levelling layer. In present study, the wear layer varied between 5 and 7 mm and the levelling layer varied between 15 and 35 mm. In two cases, there was only one single layer with a thickness of 20 mm.

Three different types of reinforcement were studied: (I) a welded-wire fabric with diameter $\Phi = 4$ mm and spacing $s = 100$ mm, (II) a galvanized welded-wire fabric with $\Phi = 2$ mm and $s = 25$ mm (usually used in plaster), and (III) two overlapping welded-wire fabrics with $\Phi = 4$ mm resulting in a spacing $s = 50$ mm. The yield strength of reinforcement in type I and type III was measured to 695 MPa. A yield strength of 437 MPa was obtained for reinforcement type II. There were also some unreinforced overlays for comparison. The fabrics were placed on the asphalt surface without any chairs or spacers.

In connection with the loading tests, material tests were carried out on cylinders and prisms. After the tests, overlay thicknesses were measured by drilling cores through the overlays (Table 2).

The shrinkage specimens consisted of eight PMC prisms 20×20×160 mm, four prisms made of mix type No. 145 (used for wear layer) and four of No. 345 (used for levelling layer), three 20×20×160 mm bituminous prisms with overlays and three 100×80×400 mm concrete prisms with overlays. On both of the latter types, one overlay consisted solely of 7-9 mm mix type No. 145, another of 6-7 mm No. 145 on top of 10 mm No. 345, and the third one consisted of 6 mm of mix type No. 145 on top of 20 mm of mix type No. 345. All prisms were cast in connection with the first part of the test series.

All slabs and specimens were cured in air and stored indoors. The temperature varied mainly between 18 and 20°C with some occasional readings of 17 and 21°C. The relative humidity varied between 40 and 60 per cent. The average was just above 50 per cent.

STATIC WHEEL LOADING

Seven PMC overlays were subjected to static wheel loading (Table 3). The slab was placed on a car that in turn was placed on two rails (Fig. 2). Horizontal movements were prevented. The load was applied by a vertical load unit via the hard plastic wheel described in the "Previous Test" section. The load was
increased in load steps, each step was 2 kN. The load was increased during about
10 seconds and was then kept constant during another 110 seconds before next
step. On four overlays, the load was increased until failure. On two other
overlays, the load was increased to 100 kN - four times the nominal wheel load -
without failure or visible cracking. The load could not be increased further due to
limitations in the testing rig. The final overlay was originally dedicated to fatigue
loading under a wheel load of 25 kN. Consequently, the static loading on this
overlay was interrupted at F=25 kN.

The highest load-carrying capacity was obtained on overlays Nos. B and H.
They were both comparatively thick (about 40 mm), contained a high strength
reinforcement with a dense spacing, and the PMC strength was high. The
unreinforced overlay No. F with comparative thickness and PMC strength only
carried half the load (Fig. 3). The unreinforced and homogeneous overlays Nos. C
and D showed high load-carrying capacity in comparison to thickness. They were,
however, both made of PMC mixes with high flexural and tensile strength values.

Overlays Nos. 7 and 8 failed at very low loads (Fig. 3). They were both rather
thin (25-30 mm), the strength of the levelling layer was low, and the
reinforcement type showed to be far from optimum. According to the PMC
producer, the strength values were uncommonly low in these overlays (compare
Table 2 with Table 1). In the second part of the tests, strength values almost twice
as high were obtained for the same mix (No. 345). The reinforcement type I had a
spacing of 100 mm. Thus, the spacing was more than three times longer than the
overlay thickness. The reinforcement type II was delivered in rolls and, thus, it
was difficult to make even and difficult to fix to the asphalt. It also developed
lower yield strength than reinforcement in types I and III.

FATIGUE LOADING

Seven overlays were subjected to the combined effects of a static wheel load
and a horizontal fatigue movement simulating rolling wheel loads (Table 4). The
vertical load was intended to be equal to the truck manufacturer's nominal load of
25 kN. On two overlays, the load was decreased to 13 kN because the static tests
on similar overlays indicated that load-carrying capacity would not reach 25 kN.
At testing, the slab was placed on a car that in turn was placed on two rails (Fig.
2). The slab was pushed back and forth by a horizontal load unit. The horizontal
movement was sinusoidal with an amplitude of ±50 mm (total movement =
100 mm). The frequency varied between 0.25 and 0.5 Hz.

First, the vertical load was applied. On overlays Nos. C and D, this loading
followed the schedule for static loading described above. On the other overlays,
the vertical load was applied rapidly, from zero to maximum load within less than
30 seconds. Thereafter, the horizontal fatigue movement was started. The number
of cycles of repeated loading at cracking and the crack patterns were recorded (Table 4).

The thick and strong overlays reinforced with type III reinforcement (Nos. A and G) showed the best structural behaviour also for fatigue loading. The thinner overlays made with poor concrete quality and less efficient reinforcement (Nos. 4 and 6) cracked earlier and obtained deeper damage in spite of the minor load. It is, however, interesting that the difference between the reinforced overlays Nos. A and G and the unreinforced overlays C and E was not that large. It can be explained in two ways: (i) The PMC especially in the thin overlay No. C (mix No. 422) demonstrated excellent fatigue properties, or (ii) the fatigue horizontal movement is a severe loading. A combination might also be possible.

The fact that a reinforced overlay, which easily carried a static load of 100 kN without cracking, failed in less than 10 000 cycles of repeated loading at the load level of 25 kN must be considered. This is an important loading case since it fairly well simulates the passage of a truck in the field. Obviously, this kind of fatigue loading is more severe than traditional fatigue loading. As the wheel passes over the floor, the stresses change between compression and tension (4). Under conventional fatigue loading tests, the stresses always have the same sign and only the magnitude varies. On the other hand, only minor damages developed in the thick reinforced overlays. In the field, such cracks might be repaired by injection. Efforts should also be taken to distribute the truck traffic making it less track bound.

SHRINKAGE TESTS

Shrinkage of PMC was measured during the first two months after placement (Table 5). Considerable shrinkage was obtained in PMCs used both for wear and levelling layers. The measured free shrinkage was 0.65 and 0.40 mm/m, respectively, i.e., close to the PMC producer's shrinkage values (Table 1). If the shrinkage strain is restrained, there is risk of cracking. The restraint is dependent on the stiffness of the underlying layer and on the friction between this layer and the overlay. A concrete floor is a stiff layer and if the surface is clean, a good bond will develop between concrete and PMC overlay (Table 1). This will promote the development of a good crack pattern with many, uniformly distributed tiny, hardly visible cracks. No visible cracks were observed on the prisms Nos. 736-738 with PMC overlays on concrete. On the other hand, there was not a complete restraint, since the prism length decreased at a rate of 0.25 mm/m. The value 0.25 was 0.17 mm/m higher than the estimated concrete shrinkage during the measuring period. (The concrete was two months old when measurement began. The concrete was presumed to have shrunk about 0.08 mm/m during the subsequent two months).
If a PMC overlay is placed on a bituminous material, the restraint will be low even if there is bond between overlay and asphalt. The cause is the large creep of the bituminous material during the long time perspective necessary to develop the shrinkage. That means that overlays of limited extent might shrink without cracking. On large areas, however, a restraint sufficiently high to cause cracking might be built up. Consequently, uncontrolled cracking and large crack widths might occur in large overlays and also in overlays fixed to connected walls, columns or other floors. By placing the wear layer on top of a levelling layer, the resulting shrinkage can be reduced because the PMC in the levelling layer is less prone to shrink. The tests indicated that the reduction will be around 0.15 mm/m. (Compare the measured shrinkage on prism No. K1 with the shrinkage of prisms Nos. K2 and K3).

CONCLUSIONS

The findings from the investigation are summarized as follows:

1. The use of PMC overlays seems to be a promising method of repairing distressed and worn out industrial asphalt floors.

2. Reinforcing the PMC overlay increases its load-carrying capacity. Reinforced overlays developed ultimate loads twice as high as unreinforced overlays with similar thickness.

3. The spacing of the welded-wire fabric should not exceed twice the overlay thickness. Fabrics delivered in rolls are difficult to make even and to fix to the asphalt layer and should be avoided.

4. The PMC producer needs a QA programme to guarantee uniformly high strengths. A considerable strength drop resulted in very low load-carrying capacity for some of the tested overlays.

5. Repeated rolling wheel loads that frequently occur on industrial floors are more severe than static loading. Reinforced overlays with proven high load-carrying capacity for static loading cracked within 10 000 cycles of repeated loading at a load level less than 25 per cent of maximum static loads. The damage was, however, much less than on unreinforced overlays subjected to the same fatigue loading.

6. PMC mixes utilized for wear and levelling layers obtained high shrinkage values of 0.65 and 0.40 mm/m, respectively.

7. By placing the wear layer on top of a levelling layer less prone to shrinkage, it is possible to reduce the overall overlay shrinkage with an amount of 0.15 mm/m and, consequently, decrease the risk of shrinkage cracking.
ACKNOWLEDGEMENT

The author wishes to express his gratitude to Dr. J. Asztély, ABS Byggsystem AB, who initiated the project. The author also would like to thank the laboratory staff at the Dept. of Structural Mechanics and Engineering at the Royal University of Technology under the supervision of Mr. K. Lindgren. Financial supports from the Development Fund of Swedish Construction Industry and from ABS Byggsystem are gratefully acknowledged.

REFERENCES


### TABLE 1 — MATERIAL DESCRIPTIONS ACCORDING TO THE PMC PRODUCER

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<tr>
<th>Mix type (trade name)</th>
<th>Dry mix proportions (%)</th>
<th>Mechanical properties</th>
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<tr>
<td></td>
<td>Quartz sand</td>
<td>Cement</td>
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<tr>
<td>145</td>
<td>35-40</td>
<td>35-40</td>
</tr>
<tr>
<td>345</td>
<td>&gt; 50</td>
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<td>422</td>
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<td>30-40</td>
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<tr>
<td>471</td>
<td>50-60</td>
<td>20-30</td>
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#### TABLE 2 — THICKNESSES, STRENGTH VALUES, AND REINFORCING TYPES

<table>
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<tr>
<th>Slab No.</th>
<th>Mix type</th>
<th>Wear layer</th>
<th>Levelling layer</th>
<th>Reinf. type</th>
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<td></td>
<td>Thickness (mm)</td>
<td>f_{cc} (MPa)</td>
<td>f_{ef} (MPa)</td>
<td>f_{ct.snl} (MPa)</td>
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<td>4</td>
<td>145</td>
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<td>25</td>
<td>9.0</td>
</tr>
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<td>41</td>
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<td>H</td>
<td>145</td>
<td>7</td>
<td>41</td>
<td>13</td>
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</table>

Note. f_{cc}=compressive strength of cylinders; f_{ef}=flexural strength of prisms 40x40x160 mm; f_{ct.snl}=splitting tensile strength of cylinders with diameter=50 and height=100 mm.
### TABLE 3 — STATIC WHEEL LOADING

<table>
<thead>
<tr>
<th>Slab No.</th>
<th>Reinforcement type</th>
<th>Failure load (kN)</th>
<th>Extension of ring formed crack (mm)</th>
<th>Number of radial cracks</th>
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<td>7</td>
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<td>16</td>
<td>130 x 130</td>
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<tr>
<td>8</td>
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<td>16</td>
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<td>None</td>
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<tr>
<td>B</td>
<td>III</td>
<td>&gt; 100</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>F</td>
<td>None</td>
<td>53</td>
<td>250 x 250</td>
<td>5</td>
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<tr>
<td>H</td>
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<td>C</td>
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<td>&gt; 25</td>
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<tr>
<td>D</td>
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<td>20</td>
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<td>6</td>
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### TABLE 4 — FATIGUE LOADING

<table>
<thead>
<tr>
<th>Slab No.</th>
<th>Reinforcement type</th>
<th>Load (kN)</th>
<th>Number of cycles at cracking</th>
<th>Total number cycles</th>
<th>Extension of ring formed crack (mm)</th>
<th>Number of radial cracks</th>
<th>Damage depth (mm)</th>
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<td>I</td>
<td>13</td>
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<td>1 660</td>
<td>130 x 190</td>
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<td>Not measured</td>
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<tr>
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<td>III</td>
<td>25</td>
<td>~ 7 000</td>
<td>24 400</td>
<td>120 x 170</td>
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<tr>
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<td>650</td>
<td>230 x 240</td>
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<tr>
<td>G</td>
<td>III</td>
<td>25</td>
<td>3 100</td>
<td>8 720</td>
<td>110 x 160</td>
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<td>3.6</td>
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</table>

Note. < 1 indicates that the overlay cracked already at the application of the vertical load.

### TABLE 5 — SHRINKAGE DURING TWO MONTHS

<table>
<thead>
<tr>
<th>Prism Nos.</th>
<th>Prism length x width (mm)</th>
<th>Prism thicknesses (mm)</th>
<th>Linear shrinkage (mm/m)</th>
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<tr>
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<td>Concrete layer</td>
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<td>Levelling layer</td>
</tr>
<tr>
<td></td>
<td></td>
<td>aggregate slab</td>
<td>Mix No. 345</td>
</tr>
<tr>
<td>U1, U2,</td>
<td>160 x 20</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>U3, U4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S1, S2,</td>
<td>160 x 20</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>S3, S4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>K1</td>
<td>160 x 40</td>
<td>-</td>
<td>20</td>
</tr>
<tr>
<td>K2</td>
<td>160 x 40</td>
<td>-</td>
<td>21</td>
</tr>
<tr>
<td>K3</td>
<td>160 x 40</td>
<td>-</td>
<td>20</td>
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<td>80 - 81</td>
<td>-</td>
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<td>736</td>
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<td>-</td>
</tr>
<tr>
<td>737</td>
<td>400 x 100</td>
<td>82</td>
<td>-</td>
</tr>
</tbody>
</table>
Fig. 1—Test slabs. Measures in mm

Fig. 2—Test rig
Fig. 3—Failure modes of slabs No. F (left) and 8 (right)
Synopsis: This research concerns the release of liquid methyl methacrylate from inside of the porous fibers into hardened concrete matrices to reduce permeability. Low heat is applied to the composite. It melts the wax coating on the fibers and dries the matrix, both of which act to move the methyl methacrylate and wax out into the matrix surrounding the fiber. The heat is increased, and the monomer becomes polymerized in the dispersed state into the matrix. Research results showed reduction in matrix permeability.

Keywords: Concretes; fibers (discrete fibers); methyl methacrylate; permeability; plastics, polymers and resins; waxes
INTRODUCTION

Incorporation of chemicals released into the matrix of cement, with later release occurring upon heat stimulation, alters the matrix permeability, therefore durability. The advantages of this design is the ability to reduce maintenance and repair costs and delay the time of repair.

Specific problems of concrete and cements such as permeability and brittleness can be addressed by changing their properties over time. The hypothesis is that by incorporating chemicals in fibers, which are later released, the material's matrix can change over time. Parameters such as permeability, pore, and crack structure can be altered. (Permeability, with its related porosity and crack structure, is the property that controls the rate of penetration of the environment.

Design to Reduce Permeability

In this design to reduce permeability, methyl methacrylate will release from hollow polypropylene porous fibers coated with wax when heat is applied after the matrix is set. The heat also dries out the matrix in preparation to draw in the methyl methacrylate. Polymerization of the methyl methacrylate in the matrix pores next occurs due to extra heat applied.

The assumption underlying this idea of timed, internal release is 1) if a chemical is released internally, it can move more easily and quickly throughout the concrete matrix giving better coverage, and 2) if the matrix or cracks are already defined in the matrix then they can be filled.

Experimentation

Visual assessment was utilized to assess the release of dyed methyl methacrylate from fibers into white cement upon heating. As seen in Photo 1, the release was successful. Scanning electron microscope photos reaffirmed the release of a chemical in the vicinity of the hollow porous fibers as seen in Photo 2. Other photos under an SEM captured the moment of release from the hollow porous fibers as seen in Photo 3.

Permeability tests were done on samples using the apparatus tests described as "A Simple Method for Measuring Water Permeability of Concrete" [4] (see Photo 4). The positive results of these tests can be seen in Figures 1 and 2.
The question was raised concerning possible damaging effects of heating or strength properties. For these tests fibers containing another fluid, calcium nitrite, were chosen. The results confirmed that heating increases the compressive strength (see Figure 3).

Tests were done with various volumes of fibers to ascertain any strength deterioration due to these volumes of fibers used. The results showed no deterioration (see Figures 4, 5, 6).

A comparison of internal release versus exterior application with gravity feed was made. The results showed samples with internally released chemicals had better compressive strength (see Figure 7) and less permeability (see Figure 8) than those using conventional surface application of methyl methacrylate and heating.

REFERENCES


2. Ibid.

3. Ibid.


6. Ibid.
Ibid.

Ibid.

Ibid.

Ibid.

Ibid.

Ibid.
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![Graph](image.png)

Fig. 1—Permeability of white cement containing methyl methacrylate and wax released from porous polypropylene fibers (samples cured for 3 days; heated to 120 F for 30 min)
Fig. 2—Permeability of portland cement containing emthyl methacrylate and wax released from polypropylene fibers (samples cured for 7 days; heated to 212 F for 30 min).  

Fig. 3—Effect of heating on compressive strength of white cement containing calcium nitrite and wax released from porous polypropylene fibers (samples cured for 3 days—heated for 30 min).
Tests were done with various volumes of fibers to ascertain any strength deterioration due to these volumes of fibers used. The results showed no deterioration (see Figures 4, 5, 6).

A comparison of internal release versus exterior application with gravity feed was made. The results showed samples with internally released chemicals had better compressive strength (see Figure 7) and less permeability (see Figure 8) than those using conventional surface application of methyl methacrylate and heating.

Fig. 4—Effect of volume of fibers on compression strength of portland cement containing methyl methacrylate and wax released from porous polypropylene fibers (samples cured for 7 days—heated to 212 F)
Fig. 5—Effect of volume of fibers on bending strength of portland cement containing methyl methacrylate and wax released from porous polypropylene fibers (samples cured for 7 days—heated for 30 min)

Fig. 6—Effect of volume of fibers on compression strength of portland cement containing methyl methacrylate and wax released from porous polypropylene fibers (samples cured for 14 days—heated to 212 F for 30 min)
Fig. 7—Effect of internal release of methyl methacrylate and wax from porous polypropylene fibers and internal polymerization versus conventional application from the exterior top surface on compressive strength (samples heated to 212 F for 30 min)\textsuperscript{11}

Fig. 8—Effect of internal release of methyl methacrylate and wax from porous polypropylene fibers and internal polymerization versus conventional application from the exterior top surface on permeability (samples cured 3 days—heated to 212 F for 30 min)\textsuperscript{12}
Photo 1—Release of red-dyed chemical from fiber into white cement matrix

Photo 2—Scanning electron microscope photograph of area around a fiber into which chemical was released
Photo 3—Scanning electron microscope photograph of porous fibers releasing their chemical.

Photo 4—Gravity flow permeability apparatus is being set up.
Synopsis: Given the breakthrough technology creating non-shrinking unsaturated polyester resin, this paper examines what this new technology might do if utilized in polymer concrete. The paper defines the criteria for success for polymer concrete in cast metal applications; and utilizing these criteria, compares the performance of the newly developed, low shrink, polyester-based systems with an accepted standard epoxy. Criteria examined include (1) stiffness-to-weight ratio equal to cast iron, (2) low coefficient of thermal expansion, (3) temperature insensitive mechanical properties, (4) adhesion to insert materials, (5) low shrinkage, (6) good composite flow and consolidation characteristics, and (7) comparable cost to machined cast metals. The data tends to show that for most applications, these new low-shrink, polyester-based polymer concretes may in fact be a new polymer-based alternative for cast metals. Given the lower costs of these low shrink polymer concrete systems, we may be defining an exciting new opportunity. Observations on initial field trials are also noted.

Keywords: Aggregates; basalt; epoxy resins; esters; mechanical properties; modulus of elasticity; polymer concrete; sands; shrinkage; temperature; thermal expansion
James E. Maass - Director of Marketing, Reichhold Chemicals, Inc., Research Triangle Park, North Carolina. Jim received a B.S. in Chemical Engineering in 1966 and an M.B.A. in Marketing in 1969. Both degrees from the University of Southern California. Jim joined Reichhold Chemicals in 1966 and since has served the company in various technical, sales, production and management functions.

INTRODUCTION

The technology of improving the properties of hardened concrete by adding polymers is now in its fifth decade(1). In the 1940's and 50's, both polymer concrete and polymer-modified concrete were studied and used. Polymer concrete was being used in the mid-50's in the production of prefabricated wall panels and floor tiles. Polymers being used included epoxy, methyl methacrylate, and unsaturated polyester styrene-based systems(2).

Precast polymer concrete panels cure in hours rather than the days required for precast concrete. Because strength ratios are three to five times higher than Portland cement, materials usage can be reduced dramatically, thereby reducing weight. In turn, lighter weight reduces installation costs.

When compared to precast concrete, polyester-based polymer concrete has superior chemical resistance, critical for exposure to extreme weather conditions and environments. Coupled with polymer concrete's high strength-to-weight ratio, numerous applications have become commercially viable. Examples of these applications are fiber optics and other underground utility boxes, manhole covers, animal stalls, transportation items such as road barriers and curbs, electrical insulators, hazardous waste disposal and chemical containment boxes and vaults, drainage systems, architectural panels, and a variety of functional items replacing decorative or structural reinforced concrete articles.

The most widely used polymer for these applications is unsaturated polyester resins. Based on current SPI statistical data, these markets represent less than twenty million pounds annually(3) less than two percent of the total polyester market. Another approximately one million
pounds is field applied for highway overlay. This business has sales estimated at more than 100 million dollars a year. Although significant, these sales are dwarfed by other construction materials.

Why has this market not grown? The answer is simple: cost. No matter what is done to reduce resin cost or usage in the composite or to utilize waste or recycled materials, polymer concrete is substantially more expensive than Portland cement. Thus it is used as a specialty or niche material, presenting no appreciable impact on the precast Portland cement concrete industry.

A much smaller segment of the precast polymer concrete business also exists. In this segment, polymer concrete products are being used as substitutes for more expensive metal castings, weldments, or precision granite surface plates. Polymer concrete manufacture is cleaner than the alternatives. It does not corrode as do metals and is normally non-conductive. By choice of filler, it can be made either lighter or heavier. It can be gel coated or post finished to create almost any aesthetic appearance. The primary consumption of these products is in the machine tool and instrumentation industries. A merchant market is developing with several precast polymer concrete suppliers devoting effort to profit from custom casting for other manufacturers.

The machine tool segment has evolved primarily in Europe, independent of the larger resin concrete industry, and is far more technical in terms of its needs and requirements than that of conventional polymer concrete. As such, there has been little relation between the two segments. The polymer concrete precast concrete segment has grown based on low cost orthophthalic or DCPD-based unsaturated polyesters, while the second segment -- polymer concrete as a cast metal replacement -- has been based on far more costly epoxy-based systems.

If lower cost unsaturated polyester could be used as a substitute for high cost epoxy systems, unprecedented uses for polymer concrete might result. One major obstacle has precluded use of polyesters for cast metal: shrinkage. Cure shrinkage of cast metals is low, as is the cure shrinkage of cast epoxy-based polymer concrete. Traditionally, polyester-based systems have demonstrated comparatively high shrinkage (approximately 2-7% linear). Shrinkage created stress, insert adhesion problems, mold design problems, and so on.

A breakthrough last year in polyester resin systems has altered conventional thinking. We are now able to formulate non-shrinking, room temperature, polyester-based polymer concrete systems at far lower cost than traditional epoxy systems. If these materials perform like
expensive epoxy systems, major markets formerly dominated by cast iron or other metal castings will open up to polymer concrete.

OBJECTIVES

Our objectives are now clear: (A) Define the criteria for success for polymer concrete in cast metal applications, and (B) utilizing these criteria, compare the performance of the newly developed, low-shrink, polyester-based systems with the accepted standard, epoxy.

A. Criteria for Machine Tool Applications

1. Stiffness-to-weight ratio equal to cast iron-- A basic requirement for a machine or instrument structural component is dimensional stability. The inherent mechanical property controlling dimensional changes of a material is the modulus of elasticity. Typically, the modulus of grey cast iron used for machine tool components is 15,000,000 psi. Cast iron's density is about three times that of polymer concrete. Therefore, equal stiffness-to-weight ratio results from a polymer concrete with a modulus of only 5,000,000 psi.

2. Low Coefficient of Thermal Expansion-- Thermal expansion of the polymer concrete within the operating temperature ranges of assembled machine tools and instruments must not be such that excess movement creates unacceptable performance. The thermal expansion of the composite material should be designed to minimize differences with other materials, i.e., cast iron, steel, aluminum, and others.

3. Temperature Insensitive Mechanical Properties-- The operating temperature of general purpose, non-precision machine tools can reach the maximum coolant temperature of about 140 to 150°F. During a temperature change, movement in the components of a machine must not result in dimensional changes in the machined parts. Therefore, the stiffness of the polymer concrete must be relatively stable for the short term temperature changes of the machine tool.

4. Adhesion to Insert Materials-- Epoxy-based polymer concrete has excellent adhesion to all types of metallic and nonmetallic insert or coring materials in precast items. Metal inserts in conventional polyester resin-based polymer concrete castings have been known to fail. The new systems must match epoxy's performance.
5. **Low Shrinkage**-- Cure shrinkage of polymer concrete is low relative to shrinkage from solidification of cast metals. However, it is important to minimize stresses that arise from polymer concrete shrinkage between inserts in casting molds. The low-shrink, epoxy-based polymer concrete generally has never presented any shrinkage problems except for improper mold design. On the other hand, conventional polyester resin for casting-based polymer concrete has limited applications in multiple insert or precision castings.

6. **Good Composite Flow and Consolidation Characteristics**-- A structural polymer concrete material must exhibit continuous, isotropic and homogeneous properties so that predictable performance may be attained with typical commercial manufacturing operations. The polymer concrete, when formulated to meet design and structural requirements, must be easily cast.

7. **Comparable Cost to Machined Cast Metals**-- The composite’s overall material costs, including inserts, and processing costs must be comparable to the finished cost of machined cast iron castings. However, in many cases, end user manufacturers are willing to pay a premium for real or perceived performance or marketing benefits that polymer concrete components produce.

**B. Comparison Study**

A new low-shrink unsaturated polyester resin was evaluated in the polymer concrete formulation versus two-component epoxy. The polyester-based polymer concrete was found to have the following performance characteristics relative to the control:

1. Raw material cost of low shrinkage polymer concrete was $0.256/lb. versus $0.458/lb. for the epoxy control.

2. The workability, castability, and surface finish of the polymer concrete were equal to that of the epoxy.

3. Linear shrinkage for the polymer concrete was 0.0007 inches/inch versus 0.0016 inches/inch for the epoxy after 4 hours cure at 150°F.
4. The dimensional stability of the polymer concrete, once cured, appeared to be superior to the epoxy system evaluated.

5. The polyester polymer concrete had a significant advantage in terms of apparent linear coefficient of thermal expansion: 7.1 min/in/°F versus 16.2 min/in/°F for the epoxy.

6. The apparent compressive strength and modulus of the polymer concrete appeared to be deficient relative to the epoxy properties, especially at elevated temperatures.

7. Quarter-inch insert torque pull-out tests for both the polymer concrete and the epoxy control failed, with bolt shear, at greater than 265 inch-lbs. Unfortunately, this result does not demonstrate any quantitative differences in adhesion to steel.

**DETAILED DATA INCLUDING TEST RESULTS(4)**

**A. Summary**

A study was completed aimed at evaluating the technical potential of a low shrinkage, unsaturated polyester resin to be used as a commercial binder in precast polymer concrete for machine tool applications. A working mix design based upon commercially available fillers and aggregates was developed for use in evaluating the low shrinkage resin. Comparison studies of physical, mechanical, and other application properties were made versus the accepted industry standard epoxy. The following sections contain as much detail as possible on the materials used and test methods, and discussion concerning mix design and performance testing along with test results.

**B. Description of Materials**

1. **Aggregate**— Two sizes of crushed basalts were supplied. The following table presents the screen size distribution for the aggregates used in this study. (Table I.)

2. **Sand**: Table II contains the screen size distribution and properties of the silica sand used in this work.
3. **Fine Filler:** Several fine microcrystalline silica fine fillers were evaluated with the resin polymer concretes to optimize flow and mechanical property performance. The Unimim Corporation's Tamsil 10 was found to have the best balance of performance properties. (Table III.)

4. **Polymer Concrete Resin Binders**

   a. **Epoxy Resin:** The control resin system used in this study was a two-component epoxy industry standard. This epoxy resin system is specially formulated for precast applications in the precision machine tool industry. The weight ratio of epoxy resin to hardener is 100:30 -- the precise ratio used in this study. Some of the resin features of this system are presented in Table IV. The principal features of this epoxy resin are its complete ambient cure without need for post cure, its low shrinkage, and its thermal/mechanical property stability. The thermal stability refers to the epoxy composite's retention of strength and modulus of elasticity over the operating temperature range of typical machine tools.

   b. **Polyester Resins:** The intent of this study was to conduct an academic comparison of an unsaturated polyester resin and the new non-shrink unsaturated polyester resin. Each resin system has its own curing components, and the following tables (V and VI) present the components and weight ratios used in the polymer concrete studies.

C. **Development of Polymer Concrete Formulation:**

1. **Evaluation Aggregates:** The thermal expansion characteristics of a cast composite depend upon the properties of the raw materials, the relative volume fractions of the components, and, among other factors, the packing of the constituents. The mix design objective for a precast machine bed is to maximize the amount of low coefficient materials in the composite. For that reason, a basalt aggregate was chosen for the high end of the mix particle size distribution. Table VII presents a comparison of the linear coefficients of thermal expansion for various aggregates and other materials.
2. Optimization of the Packing of Coarse and Fine Basalts: The objective of blending two or more gradations of basalt is to maximize the density of the mix. Therefore, a minimum void volume of the mix is produced into which the fine fillers and resin binder will fill. The size distributions of the basalts are in Table I. Bulk density of coarse and fine basalt blends was determined by filling the preblended dry aggregates into a 250-ml glass graduated cylinder while the cylinder is being vibrated at 60 Hertz. Once the aggregate stops consolidating, in approximately 30 seconds, the weight and bulk density of the blended aggregates were determined. The bulk densities versus blended percentages of the coarse and fine basalts were plotted. The optimum empirical result closely fits the optimum Fuller's curve for the two aggregates. The peak density resulted from using 60 percent coarse and 40 percent fine. This ratio was used in this study.

The smaller size aggregates and fillers in cast composites represent a smaller volume in a mix design but a much greater proportion of surface area. The fine fillers are more important in terms of flow, strength, creep, and stiffness.

3. Addition of Microcrystalline Silica: To maximize mechanical properties, a very large surface area of fine filler must be added to a resin matrix system in a composite material. Previous studies of various commercial mineral and rock fillers, including crushed versus microcrystalline silica, SiO₂, have resulted in consistent findings that microcrystalline silica produces the highest possible polymer concrete modulus of elasticity.

4. Wetability Studies: In order to evaluate the relative "wetability" of the low shrinkage resin systems to the epoxy and conventional polyester controls, a test was devised utilizing a pigment oil absorption test standard. ASTM D-1483, "Standard Test Method for Oil Absorption of Pigments by Gardner-Coleman Method," covers the determination of the amount a standard oil, linseed oil, required to completely "wet" the surface of 100 grams of pigment. Values are determined in volume units and converted to an oil absorption value of grams of linseed oil per 100 grams of pigment.

In addition to using linseed oil as a control, the various resin systems were evaluated with three different fine fillers typically used in polymer concrete formulations. Differences in oil
absorption between the resins would indicate differences in "wetability" of the resins and components.

The fine fillers chosen for this study represent different particle sizes as well as a calcium carbonate, ground limestone, and two microcrystalline silicas. The filler characteristics are presented in Table VIII. The results of the oil absorption studies using the three fine fillers and the various resin components and controls are contained in Table IX.

5. Addition of Silica Sand: Due to a gap in the distribution between the basalts and silica, an additional filler was necessary to optimize packing. The desired size and quantity of a foundry sand added to the mix was determined from the Fuller's curve. The physical properties and size distribution are contained in Table II. Round grained particles in this size range also improve the settling velocity, flow, and consolidation characteristics of the polymer concrete when being vibrated in complex molds.

Table X presents a summary of the composition of the aggregate filler system for the basic polymer concrete mix design chosen for this study. For a thermally stable polymer concrete casting, the volume fraction of the low thermal expansion aggregate should be maximized. Conversely, the resin and silica should be minimized. It is very important to note that the maximum size of the basalt in other situations may be increased, thereby further decreasing both cost and thermal expansion. The rule is that the maximum aggregate diameter should be one third the minimum wall thickness of a casting.

6. Binder Levels: Mixes were prepared using the epoxy control and each polyester resin and evaluated for packing in the optimized Fuller's curve and empirically developed mix design. The binder content was varied between 8, 10, and 12 weight percentage of the mix. In all cases, there was an excess of resin from vibrated samples at the twelve percent levels. Therefore, a design mix was established for the further polymer concrete application studies. The next section contains the formulation used.

D. Polymer Concrete Formulation

The polymer concrete formulation used in the comparison study was identical on a weight basis for both the polyester resins and
the epoxy control. The following tables (XI and XII) contain the formulation alone with volume fractions of components and costs.

The final costs for the compositions are based upon bulked, bagged basalt price of $0.13 per pound, an assumed price of $1.50 per pound for the low shrinkage polyester resin, and all materials at FOB source pricing. The low shrinkage polyester resin polymer concrete cost of $0.256/lb. is substantially less (i.e., 44%) than the epoxy concrete cost of $0.458/lb. If bulk basalt prices were used, the polymer concrete would be about $0.21/lb.

E. Casting Trials

Casting trials of flat plates were conducted in a vertical rigid aluminum mold. The purpose was to determine the degree of flow and consolidation of the polymer concrete in difficult, thin, 1-inch walled geometry. The mold surface was machined smooth and a paraffin based mold release agent applied. Mixes were mechanically mixed with a drill mixer, weighed, and placed into the plate mold while being vibrated vertically at 60 Hertz. The mold was vibrated for 10 minutes and allowed to cure at room temperature. After demolding in 24 hours, the average density was determined to establish packing efficiency. The densities, reported in Table XIV, were representative for all test specimens and for each resin system. Samples all had similar surface properties with the exception of the polyester control, which had some air inhibition of the surface.

F. Physical Properties

1. Resins: The density of resin components was determined using a 500 cc graduated flask and a balance accurate to 0.0001 gram. A blended mixture of each system was also measured for specific gravity. A 100-gram sample was ambient temperature cured and density determined by an Archimedes method. The values for the resins and calculated volume shrinkage are contained in Table XIII. The linear shrinkage of the resin systems was determined using a 10-inch x 1.5-inch, half-cylindrical mold.

2. Polymer Concretes: The wet and dry densities of several polymer concrete mixes containing 10 weight percent resin were accurately determined to establish the unrestrained volume changes of the composite systems. These determinations are not nearly as accurate as for the resin systems, but still follow the expected pattern established for the resin behavior. The low
shrinkage polyester system did not expand when filled. Table XIV contains results of those determinations. The linear shrinkage is the maximum possible as calculated from the volume change determination. The measured linear shrinkage was determined by casting inserts accurately set in a mold at 10 inches ± 0.001 using a steel fixture. In 24 hours, the fixture was removed and samples post cured 4 hours at 150°F. Those results are reported in Table XIV as "measured linear change." Finally, the samples were remeasured after a month's storage at room temperature.

G. Polymer Concrete Mechanical Properties

1. Coefficient of Thermal Expansion: Polymer concrete beams, 2 inches square by 12 inches long, were cast having 10 weight percentage resin. The linear expansion characteristic were determined using Micro-Measurements Group 4-inch gauge length CEA Series constantan alloy strain gauges. Resistance measurements, using reference strain gauges on steel controls, were determined at room temperature, while the beams were immersed into an ice water bath and at 110°F. To double check determinations and make sure the gauges were unaffected by moisture, the beams were checked by mechanical strain gauge techniques. Accuracy of the mechanical strain gauges is within ±0.0001 inches. The methods closely agreed, and the values are reported in Table XV.

2. Compressive Strength & Modulus of Elasticity: The following two tables (XVI and XVII) present test results of 2-inch diameter by 4-inch ASTM C-39 compressive strength and ASTM C-469 compressive modulus of elasticity testing. The test cylinders were all cured at room temperature for one week or more and not heated prior to room temperature testing. The heated samples were placed in an oven and allowed to equilibrate for three hours before testing. Five specimens of each sample were tested, and average compressive strength and modulus of elasticity are presented in Table XVI.

3. Insert Pull-Out Tests: Six round-threaded hole steel inserts for 8-mm bolts were cast into beams 3 inches square by 18 inches long. The inserts were rigidly placed on the center line on the bottom of a sheet metal mold, 2.5 inches apart. After the beams were cured, no shrinkage cracks due to insert restraint were observed. After room temperature curing, the inserts were tested in a torque pull-out mode to attempt to determine if there
was satisfactory adhesion between the polymer concrete and steel inserts. The outside diameter of the inserts were machined round and smooth, thus they were free to spin when turned if adhesion levels were low. This poor adhesion behavior has been observed with standard polyester casting resins.

Standard low carbon steel bolts were used for the pull-out tests. In all polymer concrete cases for the epoxy, low shrinkage, or control resin systems, the bolts failed. The steel bolts failed in shear at >265 inch-lbs. When the inserts were removed using a hammer, both polyester polymer concrete systems did not appear to have good surface adhesion to the steel when compared to the epoxy, although both passed the pull-out test.

CONCLUSIONS

Our testing and study have left us optimistic. We feel that these low shrink, polyester resin-based polymer concretes may in fact be a new polymer-based alternative for cast metals. Given the lower cost of these low shrink polymer concrete systems, we may have identified a significant new market for unsaturated polyesters.


3. SPI Committee on Resin Statistics, as compiled by Ernst & Young, 1992.

4. The author acknowledges the work of Gandalf, Inc. (John F. Kane), whose study and testing for Reichhold Chemicals formed the basis for this paper.
### TABLE I — BASALT PRODUCT SIZE DISTRIBUTIONS

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### TABLE II — SILICA SAND PROPERTIES

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</tr>
<tr>
<td>No. 200</td>
<td>0.074</td>
<td>6.8</td>
</tr>
<tr>
<td>No. 270</td>
<td>0.053</td>
<td>0.9</td>
</tr>
<tr>
<td>Pan</td>
<td>--</td>
<td>trace</td>
</tr>
</tbody>
</table>

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture Content:</td>
<td>--</td>
<td>&lt;= 0.1%</td>
</tr>
<tr>
<td>Specific Gravity:</td>
<td>--</td>
<td>2.65</td>
</tr>
<tr>
<td>Roundness:</td>
<td>--</td>
<td>0.8 - 0.9</td>
</tr>
</tbody>
</table>
### TABLE III — MICROCRYSTALLINE SILICA FINE FILLER PROPERTIES

<table>
<thead>
<tr>
<th>PROPERTY</th>
<th>FILLER</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oil Absorption, g/100g, ASTM D-1483</td>
<td>28.0</td>
</tr>
<tr>
<td>Percent Passing 325 Mesh</td>
<td>100.0</td>
</tr>
<tr>
<td>Mean Particle Size, microns</td>
<td>2.1</td>
</tr>
<tr>
<td>Surface Area, m/g</td>
<td>2.0</td>
</tr>
<tr>
<td>Moisture, %</td>
<td>0.2</td>
</tr>
</tbody>
</table>

### TABLE IV — EPOXY RESIN FEATURES

<table>
<thead>
<tr>
<th>RESIN SYSTEM</th>
<th>EPOXY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Combined Density, g/cc</td>
<td>1.062</td>
</tr>
<tr>
<td>Combined Viscosity, cps</td>
<td>170</td>
</tr>
<tr>
<td>Gel Time, 100 g/min</td>
<td>100</td>
</tr>
<tr>
<td>Peak Exotherm, °C</td>
<td>252</td>
</tr>
<tr>
<td>HDT, °F</td>
<td>175</td>
</tr>
<tr>
<td>Linear Thermal Expansion, min/in/°F</td>
<td>63</td>
</tr>
</tbody>
</table>

### TABLE V — POLYLITE® LOW SHRINKAGE RESIN SYSTEM

<table>
<thead>
<tr>
<th>LOW SHRINK POLYESTER COMPONENT</th>
<th>WEIGHT PROPORTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>New Low Shrink Polyester</td>
<td>100.00</td>
</tr>
<tr>
<td>Mooney Ten-Cen 12% Cobalt</td>
<td>0.12</td>
</tr>
<tr>
<td>n,n dimethyl aniline</td>
<td>0.12</td>
</tr>
<tr>
<td>MEK Peroxide</td>
<td>1.25</td>
</tr>
</tbody>
</table>

Comment: Sample curing trials of the neat polyester resin systems were conducted on 100-gram sample sizes. Subsequent curing trials were conducted on filled systems, and catalyst levels were chosen to allow enough open time to fill the polymer concrete in molds. With both the low profile and control systems, no ambient cure problems were observed in the 100-gram trials.
TABLE VI — POLYLITE CONTROL POLYMER CONCRETE RESIN SYSTEM

<table>
<thead>
<tr>
<th>POLYESTER CONTROL COMPONENT</th>
<th>WEIGHT PROPORTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>New Low Shrink Polyester</td>
<td>100.00</td>
</tr>
<tr>
<td>Promoter</td>
<td>1.00</td>
</tr>
<tr>
<td>Cumene Hydroperoxide</td>
<td>1.25</td>
</tr>
</tbody>
</table>

TABLE VII — TYPICAL COEFFICIENT OF THERMAL EXPANSION FOR SEVERAL MATERIALS AND AGGREGATES

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>LINEAR EXPANSION, min/in/°F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>6.0</td>
</tr>
<tr>
<td>Cast Iron</td>
<td>6.7</td>
</tr>
<tr>
<td>Aluminum</td>
<td>12.5</td>
</tr>
<tr>
<td>Basalt</td>
<td>3.0</td>
</tr>
<tr>
<td>Marble</td>
<td>3.9</td>
</tr>
<tr>
<td>Limestone</td>
<td>4.4</td>
</tr>
<tr>
<td>Granite</td>
<td>4.4</td>
</tr>
<tr>
<td>Sandstone</td>
<td>5.6</td>
</tr>
<tr>
<td>Quartz</td>
<td>6.1</td>
</tr>
</tbody>
</table>

Basalt is commonly glassy, or if crystallized, fine grained, dark colored, igneous rock. It is readily available in the northeastern and northwestern parts of the United States. Several suppliers quarry, crush into angular shapes, package the aggregate in bags; however, there are limited size distributions available. In bulk, basalt may be obtained for less than $0.038/lb.

The maximum size of the aggregate used in this study was limited to 1/4 inch. The mix design could then be processed in all small sized commercial continuous mixers. The desired shape for the basalt is as angular as possible.

Picomter determinations of the aggregate's density for the basalt used in this study was 9.953 g/cc.
## TABLE VIII — FINE FILLER CHARACTERISTICS

<table>
<thead>
<tr>
<th>FILLER</th>
<th>GAMA-SPERSE 255</th>
<th>NOVACITE L-207A</th>
<th>TAMSIL T-10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Supplier:</td>
<td>Georgia Marble Company</td>
<td>Malvern Minerals Company</td>
<td>Unimin Corp.</td>
</tr>
<tr>
<td>Compositio n:</td>
<td>95 Ca &amp; Mg Carbonates</td>
<td>99.5% Microcrystalline SiO²</td>
<td>99.5% Microcrystalline SiO²</td>
</tr>
<tr>
<td>Specific Gravity, g/cc</td>
<td>2.71</td>
<td>2.65</td>
<td>2.65</td>
</tr>
<tr>
<td>Median Particle Size, m</td>
<td>12.0</td>
<td>4.2</td>
<td>2.1</td>
</tr>
<tr>
<td>Hardness</td>
<td>3.0</td>
<td>7.0</td>
<td>7.0</td>
</tr>
<tr>
<td>Oil Absorption, g/100g</td>
<td>9.0</td>
<td>18.5</td>
<td>28</td>
</tr>
</tbody>
</table>

## TABLE IX — RESIN WET-OUT CHARACTERISTICS

<table>
<thead>
<tr>
<th>FILLER:</th>
<th>GAMA-SPERSE 255</th>
<th>NOVACITE L-207A</th>
<th>TAMSIL T-10</th>
</tr>
</thead>
<tbody>
<tr>
<td>RESIN:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Epoxy</td>
<td>18.0</td>
<td>18.6</td>
<td>27.6</td>
</tr>
<tr>
<td>Polyester Control</td>
<td>15.2</td>
<td>11.6</td>
<td>25.6</td>
</tr>
<tr>
<td>Low-Shrink Polyester</td>
<td>14.8</td>
<td>15.0</td>
<td>24.2</td>
</tr>
<tr>
<td>Linseed Oil Control</td>
<td>10.7</td>
<td>13.0</td>
<td>24.5</td>
</tr>
</tbody>
</table>

The results of the wet-out studies in Table IX showed that both polyester systems were quite similar and "better" than the epoxy control for all fillers tried. The polyester resin wet-out of the small particle size (4.2m), silane surface treated Novacite was very good when compared to the untreated, larger size gama-sperse. This would indicate the improvements in polymer concrete.
wetability for either polyester resin would be achieved through surface treatment of the fine filler. Since this study was aimed at maximizing stiffness of the polymer concrete, the finest filler was chosen.

TABLE X — VOLUME AND WEIGHT RATIOS OF BASALT AND SILICA

<table>
<thead>
<tr>
<th>AGGREGATE/FILLER:</th>
<th>BASALT</th>
<th>SILICA</th>
<th>COMBINED</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight Percent</td>
<td>72</td>
<td>28</td>
<td>100.0</td>
</tr>
<tr>
<td>Density, g/cc</td>
<td>2.953</td>
<td>2.65</td>
<td>2.861</td>
</tr>
<tr>
<td>Volume, cc</td>
<td>24.382</td>
<td>10.566</td>
<td>34.948</td>
</tr>
<tr>
<td>Volume, %</td>
<td>69.8</td>
<td>30.2</td>
<td>100.0</td>
</tr>
<tr>
<td>Linear Expansion, min/in/°F</td>
<td>3.0</td>
<td>6.1</td>
<td>3.9</td>
</tr>
</tbody>
</table>

TABLE XI — FORMULATION AND COST FOR EPOXY POLYMER CONCRETE

<table>
<thead>
<tr>
<th>INGREDIENT</th>
<th>UNIT COST, $/lb.*</th>
<th>WEIGHT, %</th>
<th>VOLUME, %</th>
<th>COST, $</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse Basalt</td>
<td>0.13</td>
<td>40.0</td>
<td>33.1</td>
<td>5.20</td>
</tr>
<tr>
<td>Fine Basalt</td>
<td>0.13</td>
<td>25.0</td>
<td>20.6</td>
<td>3.25</td>
</tr>
<tr>
<td>Sand</td>
<td>0.02</td>
<td>10.0</td>
<td>9.2</td>
<td>0.20</td>
</tr>
<tr>
<td>Silica</td>
<td>0.13</td>
<td>15.0</td>
<td>14.1</td>
<td>1.95</td>
</tr>
<tr>
<td>Epoxy Resin</td>
<td>3.52</td>
<td>10.0</td>
<td>23.0</td>
<td>35.20</td>
</tr>
<tr>
<td>TOTAL</td>
<td>--</td>
<td>100.0</td>
<td>100.0</td>
<td>45.80</td>
</tr>
</tbody>
</table>
TABLE XII — FORMULATION AND COST FOR LOW SHRINKAGE POLYMER CONCRETE

<table>
<thead>
<tr>
<th>INGREDIENT</th>
<th>UNIT COST, $/lb.*</th>
<th>WEIGHT, %</th>
<th>VOLUME, %</th>
<th>COST, $</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse Basalt</td>
<td>0.13</td>
<td>40.0</td>
<td>32.7</td>
<td>5.20</td>
</tr>
<tr>
<td>Fine Basalt</td>
<td>0.13</td>
<td>25.0</td>
<td>20.5</td>
<td>3.25</td>
</tr>
<tr>
<td>Sand</td>
<td>0.02</td>
<td>10.0</td>
<td>9.1</td>
<td>0.20</td>
</tr>
<tr>
<td>Silica</td>
<td>0.13</td>
<td>15.0</td>
<td>13.9</td>
<td>1.95</td>
</tr>
<tr>
<td>Resin</td>
<td>1.50</td>
<td>10.0</td>
<td>23.8</td>
<td>15.00</td>
</tr>
<tr>
<td>TOTAL</td>
<td>--</td>
<td>100.0</td>
<td>100.0</td>
<td>25.60</td>
</tr>
</tbody>
</table>

*Truckload pricing, FOB source.

TABLE XIII — RESIN SPECIFIC GRAVITY AND SHRINKAGE DETERMINATIONS

<table>
<thead>
<tr>
<th>RESIN SYSTEM:</th>
<th>EPOXY</th>
<th>POLYESTER CONTROL</th>
<th>LOW SHRINK POLYESTER</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific Gravity, wet</td>
<td>1.061</td>
<td>1.060</td>
<td>1.015</td>
</tr>
<tr>
<td>Specific Gravity, cured</td>
<td>1.088 ± 0.002</td>
<td>1.198 ± 0.008</td>
<td>1.003 ± 0.005</td>
</tr>
<tr>
<td>Volume Change, %</td>
<td>-2.48</td>
<td>-11.52</td>
<td>+1.20</td>
</tr>
<tr>
<td>Calculated Line Change, inches/inch</td>
<td>-0.0083</td>
<td>-0.0384</td>
<td>+0.0040</td>
</tr>
<tr>
<td>Comments</td>
<td>clear, green, no voids</td>
<td>clear, amber, slight entrapped air</td>
<td>white, cracked, friable</td>
</tr>
<tr>
<td>Measured Linear Change, inches/inch</td>
<td>-0.0043</td>
<td>-0.0225</td>
<td>+0.0037</td>
</tr>
</tbody>
</table>

The calculated linear shrinkage is determined from unrestrained sample molds and is consequently greater than the 10-inch linear shrinkage determinations. The low shrinkage polyester resin expanded and cracked. Since the specific gravity determination
was via an Archimedes method, ignoring cracks, the expansion is representative of the cured, unfilled, low shrinkage resin system.

**TABLE XIV — DENSITIES AND SHRINKAGE OF EPOXY AND POLYMER CONCRETES**

<table>
<thead>
<tr>
<th>RESIN SYSTEM:</th>
<th>EPOXY</th>
<th>POLYESTER CONTROL</th>
<th>LOW SHRINK POLYESTER</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific Gravity, wet</td>
<td>2.329</td>
<td>2.435</td>
<td>2.347</td>
</tr>
<tr>
<td>Specific Gravity, cured</td>
<td>2.319</td>
<td>2.369</td>
<td>2.344</td>
</tr>
<tr>
<td>Volume Change, %</td>
<td>0.45</td>
<td>2.79</td>
<td>0.12</td>
</tr>
<tr>
<td>Calculated Line Change, inches/inch</td>
<td>-0.0015</td>
<td>-0.0093</td>
<td>-0.0004</td>
</tr>
<tr>
<td>Measured Linear Change, inches/inch</td>
<td>-0.0016</td>
<td>-0.0069</td>
<td>-0.0007</td>
</tr>
<tr>
<td>Measured Linear Change between 24 hours and 1 month, inches/inch</td>
<td>-0.0004</td>
<td>-0.0027</td>
<td>-0.0001</td>
</tr>
</tbody>
</table>

The results indicate that the low shrinkage polyester system exhibited less than half the shrinkage of the epoxy control polymer concrete. In addition, once the low shrinkage polyester polymer concrete cured, it remained dimensionally stable.

The shrinkage of the low shrinkage polyester resin polymer concrete was found to be significantly below that of the epoxy polymer concrete.
### TABLE XV — LINEAR COEFFICIENT OF THERMAL EXPANSION FOR EPOXY VERSUS POLYMER CONCRETES

<table>
<thead>
<tr>
<th>POLYMER CONCRETE</th>
<th>EPOXY</th>
<th>POLYESTER CONTROL</th>
<th>LOW SHRINK POLYESTER</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Coefficient of Thermal Expansion, min/in/°F</td>
<td>16.2</td>
<td>20.8</td>
<td>7.08</td>
</tr>
</tbody>
</table>

### TABLE XVI — ROOM TEMPERATURE MECHANICAL PROPERTIES

**ASTM C-39: 2"x4" Cylinders @ 72°F**

<table>
<thead>
<tr>
<th>POLYMER CONCRETE</th>
<th>EPOXY</th>
<th>POLYESTER CONTROL</th>
<th>LOW SHRINK POLYESTER</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength, psi</td>
<td>15,100 ± 390</td>
<td>13,394 ± 1,125</td>
<td>12,670 ± 420</td>
</tr>
<tr>
<td>Compressive Modulus of Elasticity, psi</td>
<td>5.04 ± 0.012 x 10^6</td>
<td>4.85 ± 0.142 x 10^6</td>
<td>4.80 ± 0.288 x 10^6</td>
</tr>
</tbody>
</table>

### TABLE XVII — ELEVATED TEMPERATURE MECHANICAL PROPERTIES

**ASTM C-39: 2"x4" Cylinders @ 108°F**

<table>
<thead>
<tr>
<th>POLYMER CONCRETE</th>
<th>EPOXY</th>
<th>POLYESTER CONTROL</th>
<th>LOW SHRINK POLYESTER</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength, psi</td>
<td>14,240 ± 470</td>
<td>8,586 ± 786</td>
<td>10,780 ± 670</td>
</tr>
<tr>
<td>Compressive Modulus of Elasticity, psi</td>
<td>4.97 ± 0.108 x 10^6</td>
<td>2.92 ± 0.230 x 10^6</td>
<td>3.26 ± 0.522 x 10^6</td>
</tr>
</tbody>
</table>

The polyester control polymer concrete had mechanical properties at room temperature or at 100°F equal to the epoxy control. The plastic deformation at break for the low shrinkage polyester polymer concrete was much greater: 6,400 to 7,000...
mstrain versus the epoxy polymer concrete at 3,900 to 4,300 mstrain. This indicates that more energy may be used to fracture the low shrinkage polymer concrete than the epoxy.
15-Year Tracking Study: Comparing Epoxy Polymer Concrete to Portland Cement Concrete Applied on Slab-on-Grade and Bridge Decks

by Floyd E. Dimmick, Sr.

Synopsis:
A major problem confronting transportation departments is the surface deterioration of portland cement concrete (PCC) pavements and bridge decks. Some of these defects include cracking, spalling, polishing and surface erosion. Each of these defects contributes to further deterioration within the concrete structure by allowing an infiltration of moisture, oxygen, deicing salts, chlorides and other contaminants. Upon contact with the reinforcing steel, rusting occurs causing internal tensile stresses that result in further surface spalling, hollow plane delamination and cracking.

One effective technique used since the middle 1950's (1) is to retard this corrosion process by preventing the penetration of chlorides and moisture into the concrete with an impermeable epoxy polymer concrete (EPC) overlay. These overlays also provide wear-resistant surfaces and extend the service life of the pavement or deck. Documented experience indicates that EPC overlays are cost effective, reduce overall annual maintenance costs and provide a safe driving surface.

This paper presents two project tracking studies. The first is a comparison of a new PCC slab placement to a thin EPC overlayment on an existing PCC pavement installed 15 years ago. The pavements are side-by-side and the documentation compares traffic volumes and surface deteriorations such as wearability, spalling, polishing and cracks. The second study involves a thin EPC overlay placed on a badly deteriorated PCC bridge deck 10 years ago to improve skid properties and provide an overall safer driving surface.

Keywords: Concretes; cracking (fracturing); durability; epoxies; erosion; finishes; polymer concrete; portland cements; spalling; surface defects; traffic volume
Floyd E. Dimmick, Sr., Technical Director of the Thermal-Chem Division of Armitage Industries, Inc. has 30 years experience in polymer concrete structural restoration/preservation. An active member of ACI committees 503, 546 and 548 and a member of the Transportation Research Board and ASTM committees, he teaches an accredited concrete restoration school for engineers and contractors. He is the author/contributing author of numerous restoration manuals and published papers.

TRACKING STUDY NO. 1: EPC OVERLAY COMPARISON TO NEW SLAB PLACEMENT OF PCC

INTRODUCTION

In 1977, the New Jersey Turnpike Authority decided to compare a proprietary EPC with PCC for surface wearability and pavement durability. A restoration project was in the final stages of completion located at Interchange No. 14 toll plaza near Newark International Airport. All approach, departure and toll booth slabs had been replaced during the previous months except the slabs at the toll booths on lanes one and two.

The daytime ambient temperatures were $39 \pm 5^\circ F (4 \pm 2^\circ C)$. The concrete substrate maintained an even $37^\circ F (3^\circ C)$ temperature reading throughout the day. The sun shone with occasional cloud cover. There were nearly windless conditions most of the day; a perfect day for PCC and EPC placements.

PCC Placement - Lane No. 2

Full depth removal of the deteriorated concrete took place on the previous day including all clean-up and forming. A ready mix concrete truck delivered the PCC and the workmen placed, consolidated and finished the PCC with a textured surface. A curing compound was evenly sprayed over the newly placed concrete and allowed to cure for seven days before use. The PCC averaged 6,000 psi (41.4 MPa) compressive strength, a proven high quality mix design.
Surface Preparation - Lane No. 1 (EPC Lane)

The concrete surface on this lane was badly worn in two track areas where the vehicle tires were forced to drive. Surface preparation included abrasive blasting with a sharp angular sand. All sand and debris were removed and the surface blown dust free with air. The cleaned concrete was tested for a pH reading. It tested 9.5 and therefore no acid etching was required.

A abrasive blasting exposed a shrinkage crack in the center of the slab. The crack did not penetrate deeper than approximately 3/8 inch (9 mm), therefore it was cut out with a saw to a depth of 5/8 inch (16 mm) by 1/4 inch (6 mm) wide.

EPC Material and Mixing

The EPC consisted of a patented epoxy adhesive and a blended dry silica sand mixture. The adhesive binder formulation consisted of a modified bisphenol epichlorohydrin epoxy resin, and a modified aliphatic amine curing agent. The epoxy used was Thermal-Chem Mortar Resin, Product No. 3, color - C-Gray, rapid cure. The aggregate blend consisted of sub-angular silica sands, in a gradation selected for its density and non-skid properties. The air content of the placed EPC averaged 2.5%. The epoxy content of the EPC mixture was 14%, parts by weight.

Both epoxy components were measured separately, parts by weight, and poured into a 30 gallon (115 l) mixing vessel. They were mixed with a slow-speed (approximately 400 rpm) electric drill with a paint stirrer attachment for 2 minutes. Immediately thereafter, the aggregate was slowly added and blended to a homogenous mixture. Each 0.5 ft³ (0.014 m³) batch was hauled to the placement area in the mixing vessel. Potlife (working time) of the material was approximately 35 minutes.

The Overlay Process

The PCC surface was primed with the epoxy adhesive, in a neat consistency to an average thickness of 10 mils (0.25 mm). Care was taken not to allow the epoxy to form puddles or to fill the saw cut opening. The purpose of priming the substrate with the neat epoxy is to
wet-out the PCC and allow a drier consistency EPC formula to be placed. The EPC overlay was applied over the wet, primed surface. Consolidation and placement was accomplished with the use of a wooden screed bar. Care was taken not to leave low areas. An average depth of 5/8 in. (16 mm) was placed in the wheel path areas and feather-edged in the center and on the sides next to the curbs.

The joint edges were badly spalled. A form was placed in the joint opening and the EPC filled the spalled areas to a depth of approximately 3 in. (75mm). The sawed out crack area was filled and compacted with the EPC as part of the continuous placement procedure (Picture 1). The EPC will not shrink or swell during placement and curing. Since the formula will not self-level, therefore, the surface profile must be accomplished by the spreading and compaction technique.

The EPC overlay adjacent to the joint forms were troweled and finished in the exact same manner as the PCC in Lane 2. The completed EPC overlay, surface profile matched the PCC brush finished texture in Lane 2 (Picture 2).

**Cure Time**

The EPC overlay was tack-free and useable 4 hours after placement, however, it was not reopened to traffic until the next day. Table 1 describes the EPC overlay cured physical strength properties. The joints were unable to be filled on the PCC placement for 24 hours. Therefore, all joints in both lanes were filled at the same time on the following day. The placement of this EPC overlay was reported by Howard University and the National Science Foundation in December 1981 (2).

**PCC AND EPC COMPARISON AT SIX YEARS**

According to the New Jersey Turnpike Authority, lanes one and two are used mainly by trucks and buses. Both lanes were exposed to similar traffic conditions during the 6 year comparison.

**PCC Lane No. 2 - Condition**

The PCC showed severe erosion of the surface caused by vehicular
wheel traffic. The depth of these erosion areas varied from 1/2 to 3/4 in. (13-19 mm). Deeper spalling was present over a 10 ft² (1 m²) area. Random and longitudinal cracks were present in both track areas causing spalling to occur at the surface edges of these cracks. The overall surface condition of this lane was polished and very slippery when wet. The PCC pavement wore out at about 97 million vehicle usage (Picture 3).

**EPC Lane No. 1 - Condition**

The EPC overlay showed no surface deterioration. It continued to maintain an excellent textured surface profile and provided a safe stopping and starting surface (Picture 4).

**Results of the Six Year Comparison**

All the other PCC lanes that had been newly placed six years earlier at the same toll booth Interchange, approximately 86,000 ft² (7,990 m²), had become deteriorated and worn similar to Lane No. 2, and within the next two years these lanes would be overlayed with the same EPC premixed formula. (Figure 1). The average depth of the EPC overlay thickness was 5/8 in. (16mm).

**15 YEAR TRACKING STUDY - A CLOSE-UP OF THE EPC SURFACE**

The New Jersey Turnpike Authority has provided detailed 5 year annual traffic volume counts for the years of 1988-1992 of over 81 million vehicles using Lane No. 1 (Table 2). Volume numbers for the 5 year period extrapolated over a 15 year period would conservatively support the summation that Lane No. 1 has provided service for approximately 243 million vehicles.

With 15 years of service life on the EPC overlay in Lane No. 1, the surface profile is still providing an excellent skid resistant wear surface and is protecting the PCC substrate from moisture and chloride intrusion.

At the time of reporting these observations, the EPC overlay has provided 2.5 times the service life of the PCC (Figure 2).
A Small Patch is Required

Lane No. 1 required its first maintenance after 14 years of service. A small area adjacent to a control joint had delaminated under the EPC overlay. After closely examining the small delamination, it was obvious that the moisture intrusion started in the deteriorated control joint. After sounding the overlay it was determined that approximately 1 ft² (.09 m²) will need to be cleaned and patched. A repair or replacement of the control joint material will protect the concrete from further deterioration (Picture 5).

Surface Gouge

The only other noted damage to the EPC overlay was a gouge in one of the driving wheel track areas. It is approximately 16 in. (.4 m) long, 1/4 to 1/2 inch (6 to 13 mm) deep and 1/2 inch (13 mm) average width. This damage has not widened or spalled on the edges of the cut since it was first observed two years ago according to the original installation contractor who has routinely inspected these projects.

Surface Profile

The surface has maintained an even wear pattern in the wheel track areas of principal use. Approximately 1/8 in. (3 mm) of wear has occurred in 15 years. The worn areas do not exhibit polishing or a slippery condition when wet. It will be interesting to periodically observe this application in the future to determine the full service life of a thin EPC overlay under these severe conditions of usage by mainly trucks and buses.

TRACKING STUDY NO. 2: EPC PLACEMENT FOR SAFETY TEXTURED PROFILE ON AN INTERSTATE BRIDGE DECK

Introduction

In 1983, the Ohio Department of Transportation (ODOT) found it necessary to overlay 3 bridge decks in the same interchange located
approximately 15 miles east of Cleveland, Ohio to improve the tire traction during rain, snow or ice conditions. The PCC bridge deck being reported in this paper is Project No. 811 on I-90 eastbound over I-271. The two lane bridge number is LAK-90-0243R and was built in 1962.

### Deck Surface Conditions

ODOT considered the bridge deck surface condition to be in need of a new wear surface. The deck had some transverse structural cracks, spalling and the previously grooved surface had worn to the full depth of the grooves in many areas. The original PCC physical strength properties and mix design are unknown.

### Job Site Conditions

The EPC overlay was placed in October 1983. The daytime temperatures averaged 50°F (10°C). The mornings were overcast with the sun appearing in the late morning and remaining for the balance of the day. Early morning dew was reported which evaporated within a few hours. The night temperatures were not reported but believed to be in the upper 30°F's (3-5°C).

One lane remained open during the overlay process. The deck was long, 2 lanes wide and on a curve. ODOT posted reduced speed signs to 35 mph (60 km/h) on the lane that was not being overlayed, however slowing traffic down below 55 mph (90 km/h) was a real problem. Fast moving trucks would cause the deck to vibrate as they passed.

### Surface Preparation

The surface preparation included a dust free abrasive shot blasting of the PCC surface. The angular shaped steel shot removed different depths of the weak PCC surface. Adjacent to the steel expansion joints a 1/4 in. (6 mm) depth x 12 in. (.3 m) width of the PCC surface was removed to provide a smooth transition over the deck joints.

A close examination of the abraded surface next to the uncleaned surface showed that most of the safety grooves previously cut in the concrete for traction were being removed by the abrasive blasting (Picture 6). It is believed that the grooving process permitted easier
access by chlorides and water, thereby weakening the upper portion of the PCC. Combined with freeze-thaw damage, high speed and high traffic volumes, this led to faster deterioration. The steel shot blasting also removed the softer aggregates already damaged by heavy salting and other environmental conditions. Small, irregular shaped holes up to 3/8 in. (9 mm) in diameter by equal depth were also very noticeable (Picture 7).

The abrasive blasted cleaned deck was tested for its pH reading. The average readings exceeded 13, therefore, the surface was acid etched with a 10% solution of muriatic acid at the rate of 1 gallon/40 ft² (1 l/1 m²) and thoroughly rinsed with clean water trucked to the job site. The new pH reading was recorded as 9.2.

The structural cracks were not repaired because the life expectancy of the deck was only 5 years. The patching of the spalled surfaces and other surface defects were completed as part of the overlay process.

EPC OVERLAYMENT PROCEDURE

Surface Priming

Immediately after the surface preparation was completed, the process of overlay placement began. The highest elevated lane was placed in its entirety first. The surface was primed by rolling the neat epoxy polymer onto the deck to wet out the entire PCC surface. The typical rate of application was 1 gallon per 160 ft² (1 l/3.75 m²), however, some porous areas absorbed twice that quantity without a film buildup.

EPC Material and Mixing

The EPC placed on this bridge deck was Thermal-Chem Mortar Resin, Product No. 3. The epoxy components were temperature preconditioned in a heating box to 77° F (25° C) before they were mixed together with a slow speed electric paint type mixing unit. The mixed polymer was poured into a concrete drum mixer. The agitation was started and the graded angular silica sand was added to the epoxy polymer and blended until a homogenous EPC mass developed. The EPC was removed from the mixing drum and hauled to the location for placement. Each batch contained 2 1/2 ft³ (.07 m³). The epoxy content of the EPC mixture was 10.5%, parts by weight.
EPC Placement and Cure

The EPC was spread evenly over the wet primed surface at an average 1/4 in. (6 mm) thickness, consolidated and screeded. A wooden screed bar was selected that provided a texturing effect as required by the ODOT. The textured surface profile desired was a high profile with irregular transverse ridges. These ridges provided an excellent driving surface which also held de-icing materials better on this steeply graded deck (Picture 8). Approximately 10 hours after the first lane was placed, traffic was moved onto the newly placed EPC. The same surface preparation and EPC overlayment was then placed on the second lane. The total surface area overlayed with the EPC was 17,676 ft² (1,637 m²).

10 YEAR TRACKING STUDY RESULTS

Traffic Volumes and Skid Resistance Data

ODOT has provided daily traffic volume counts of 33,000 vehicles per day, or approximately 120.8 million vehicles have safely traveled over the EPC overlay in the last 10 years. The deck still maintains excellent anti-skid properties. Table 3 shows the current wet skid resistance numbers and the approximate traffic volumes for the EPC overlayment at the time of testing.

ODOT typically uses skid numbers in the mid-20 range for surfaces that are candidates for overlayment or replacement. The EPC overlay has also already doubled its original life expectancy and is still in use (Picture 9).

A Patch is Required

Over the years ODOT has reported that only a couple of square yards (meters) has debonded and required patching. It was not reported whether the EPC did not adhere, or if the EPC remained bonded to the PCC and a weak layer of PCC failed. In either situation the maintenance required on this EPC overlay for a ten year period has been minimal.
CONCLUSIONS

1. EPC Compatibility

These tracking studies of EPC overlays on a bridge deck and slab-on-grade confirm the compatibility between higher strength EPC and PCC. Since the coefficient of thermal expansion of EPC differs from that of PCC, the EPC will not stress the PCC when sufficient tensile elongation is designed into the EPC (3). It is also believed that the wider area of molecular-entanglement (bond-line) of the two concretes improves the compatibility by reducing critical tensile stresses.

When the correct physical strength properties are designed and formulated into an epoxy resin system, and a low void (air content) gradation of silica sands or other aggregate combinations are properly mixed together, EPC provides excellent compatibility with PCC. The typical air content of EPC should range between 2 and 4 percent.

2. Adhesion

The EPC formulations used on these projects have unique bonding and wetting properties. The epoxy polymer in its liquid state wets the PCC surface through the lateral spreading of the epoxy film. Penetration of the uncured resin into the PCC cavities and the cement molecules is a function of physical (mechanical) and chemical absorption processes (4). The addition of the chemisorbed process and molecular entanglement of bonding sites combined with the normal mechanical methods of adhesion to the cement creates a monolithic interfacial attachment to the PCC. This function increases the adhesive-joint strength so that a cohesive-plateau state can be attained on clean PCC. The interfacial absorption functionality of this given epoxy system remains constant.

Hence, the tracking study confirms this phenomenon of adhesion under two different adverse and normal conditions of application; 1. high speed traffic adjacent to the uncured EPC placement on the bridge deck; and, 2. low temperature PCC substrate conditions during placement of the EPC overlay on slab-on-grade applications. Both EPC overlays were subjected to high traffic volumes of automobile, bus and truck loads, deicing materials combined with natural exposures and environmental conditions spanning a 120° F (57° C) temperature range over a time span of 10 to 15 years.
3. **Traction and Wearability**

The excellent skid properties obtained by both EPC overlays were attributed to several factors. The angular shape and hardness of the aggregates in a properly proportioned mixture that is neither resin rich or starved. Too much epoxy will create slippery driving conditions when the surface is wet. Too little epoxy will create an excellent temporary skid resistance wear surface, but will deteriorate very quickly, because the aggregate will break-out of the overlay without being worn down. Also, it will allow the transfer of water and chlorides into the PCC by not creating an impervious overlay. Balanced polymer and aggregate formulations are required for successful EPC overlays.

**ACKNOWLEDGMENTS**

The author gratefully acknowledges the following for their cooperation and assistance on these projects:

Tracking Study No. 1
Contractor: Donald Ehret, Sr., President, Ehret and Company General Contractors, Inc., West Trenton, New Jersey
Engineer: Robert Gerberth, P.E. Manager, Roadway Maintenance, New Jersey Turnpike Authority, Newark, New Jersey

Tracking Study No. 2
Engineer: George Maki II, P.E., District Bridge Engineer, Ohio Department of Transportation, Garfield Heights, Ohio.

**REFERENCES**

1. McConnell, W.R., "Epoxy Surface Treatments for Portland Cement Concrete Pavements", Epoxies with Concrete, ACI publ. SP-21, 1966


### TABLE 1 — EPOXY POLYMER CONCRETE TECHNICAL DATA

<table>
<thead>
<tr>
<th>Polymer:</th>
<th>Thermal-Chem Mortar Resin, Product No. 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aggregate:</td>
<td>Select gradation of angular silica sands</td>
</tr>
<tr>
<td>Unit Weight:</td>
<td>122 lb/ft$^3$ (1,958 kg/m$^3$)</td>
</tr>
<tr>
<td>Air Void Content:</td>
<td>2.5%</td>
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</table>

#### Compressive Strength, ASTM C109

<table>
<thead>
<tr>
<th>Time (days)</th>
<th>Average Compressive Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 day (24 hrs.)</td>
<td>9,610 psi (66.3 MPa)</td>
</tr>
<tr>
<td>3 days (72 hrs.)</td>
<td>11,400 psi (78.7 MPa)</td>
</tr>
<tr>
<td>5 days (120 hrs.)</td>
<td>13,993 psi (96.3 MPa)</td>
</tr>
<tr>
<td>7 days (168 hrs.)</td>
<td>14,360 psi (99.0 MPa)</td>
</tr>
<tr>
<td>28 days (672 hrs.)</td>
<td>14,673 psi (101.2 MPa)</td>
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#### Compressive Shear Strength - Federal Spec. MMM-B-380a

<table>
<thead>
<tr>
<th>Time (days)</th>
<th>Average Shear Strength (psi)</th>
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</thead>
<tbody>
<tr>
<td>7 days (168 hrs.)</td>
<td>510 psi (3.5 MPa)</td>
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</tbody>
</table>

#### Splitting Tensile Strength - ASTM C496

<table>
<thead>
<tr>
<th>Time (days)</th>
<th>Average Splitting Tensile Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 days (72 hrs.)</td>
<td>1,515 psi (10.5 MPa)</td>
</tr>
<tr>
<td>7 days (168 hrs.)</td>
<td>1,760 psi (12.1 MPa)</td>
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</table>

#### Modulus of Elasticity

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<th>Average Modulus (psi)</th>
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<tbody>
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<td>Elastic Modulus</td>
<td>$1.3 \times 10^6$</td>
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#### Tensile Strength - ASTM C190

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<th>Time (days)</th>
<th>Average Tensile Strength (psi)</th>
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</thead>
<tbody>
<tr>
<td>3 days (72 hrs.)</td>
<td>1,590 psi (11.0 MPa)</td>
</tr>
<tr>
<td>7 days (168 hrs.)</td>
<td>1,820 psi (12.5 MPa)</td>
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</tbody>
</table>

#### Tensile Strength - ASTM D638

<table>
<thead>
<tr>
<th>Time (days)</th>
<th>Average Tensile Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Neat Material Only</td>
<td>6,900 psi (47.8 MPa)</td>
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</table>

#### Flexural Strength - ASTM D790

<table>
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<tr>
<th>Time (days)</th>
<th>Average Flexural Strength (psi)</th>
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</thead>
<tbody>
<tr>
<td>7 days (168 hrs.)</td>
<td>3,320 psi (22.9 MPa)</td>
</tr>
</tbody>
</table>

#### Coefficient of Linear Thermal Expansion - ASTM D696

<table>
<thead>
<tr>
<th>Time (days)</th>
<th>Average Coefficient (1/°F)</th>
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</thead>
<tbody>
<tr>
<td>7 days (168 hrs.)</td>
<td>$1.81 \times 10^{-4}$</td>
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</table>

#### Freeze Thaw Resistance - ASTM C666

<table>
<thead>
<tr>
<th>No. Cycles</th>
<th>Rating</th>
<th>P/C Durability Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>Avg. 100.0</td>
</tr>
<tr>
<td>120</td>
<td>0</td>
<td>Avg. 100.0</td>
</tr>
<tr>
<td>150</td>
<td>0</td>
<td>Avg. 100.0</td>
</tr>
<tr>
<td>210</td>
<td>0</td>
<td>Avg. 99.2</td>
</tr>
<tr>
<td>300</td>
<td>0</td>
<td>Avg. 98.2</td>
</tr>
</tbody>
</table>

Rating: 0 Condition of Surface: No Scaling

#### Scaling Resistance of Surface Exposed to Delining Chemicals - ASTM C672

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<tr>
<th>No. Cycles</th>
<th>Rating</th>
<th>Condition of Surface</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>0</td>
<td>No Scaling</td>
</tr>
</tbody>
</table>

Rating: 0 Condition of Surface: No Scaling
### TABLE 2 — VEHICLE TRAFFIC VOLUMES, LANE 1, NEW JERSEY TURNPIKE INTERCHANGE 14

<table>
<thead>
<tr>
<th>Year</th>
<th>Traffic Volumes</th>
</tr>
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<tbody>
<tr>
<td>1988</td>
<td>16,685,740</td>
</tr>
<tr>
<td>1989</td>
<td>16,158,206</td>
</tr>
<tr>
<td>1990</td>
<td>16,653,117</td>
</tr>
<tr>
<td>1991</td>
<td>15,750,994</td>
</tr>
<tr>
<td>1992</td>
<td>15,786,515</td>
</tr>
<tr>
<td>Actual 5 Year Count</td>
<td>81,034,572</td>
</tr>
</tbody>
</table>

15 Year Average: 243 Million

Primary Usage: Mainly truck and bus lane
Ref: New Jersey Turnpike Authority Operations Department, May 20, 1993

### TABLE 3 — SKID RESISTANCE DATA FOR EPC OVERLAY ON BRIDGE DECK

Test Method: AASHTO T 242/ASTM E274
Frictional Properties of Paved Surfaces Using Full-Scale Tire

<table>
<thead>
<tr>
<th>Test Data</th>
<th>Skid Number</th>
<th>Approximate Traffic Volumes</th>
</tr>
</thead>
<tbody>
<tr>
<td>4-84</td>
<td>50.0</td>
<td>6 million</td>
</tr>
<tr>
<td>11-84</td>
<td>48.0</td>
<td>13 million</td>
</tr>
<tr>
<td>5-86</td>
<td>38.8</td>
<td>30.8 million</td>
</tr>
<tr>
<td>5-89</td>
<td>38.0</td>
<td>64.4 million</td>
</tr>
</tbody>
</table>

Ref: Ohio DOT data provided June, 1993
Location: New Jersey Turnpike Interchange No. 14 Newark International Airport

Fig. 1—15 years of successful service and still providing concrete protection and a safe driving surface.
New Jersey Turnpike Interchange 14

Lane 1, EPC Overlay
- 243 Million Vehicle Usage
- 2 1/2 Times Longer Service Life than PCC
- Excellent Surface Profile
- EPC Still in Use

Lane 2, PCC Overlay
- 100% PCC Service Life
- 97 Million Vehicle Usage

All Lanes EPC Overlaid within 2 Year Period

Ref: New Jersey Turnpike Authority Operations Department, May 20, 1993

Fig. 2—Service life tracking study PCC and EPC comparison
Photo 1—New Jersey Turnpike interchange No. 14—Placement of EPC overlay, lane 1

Photo 2—New Jersey Turnpike interchange No. 14—EPC finished surface profile, lane 1
Photo 3—New Jersey Turnpike interchange No. 14—6 yr closeup of EPC, lane 1

Photo 4—New Jersey Turnpike interchange No. 14—6 yr closeup of PCC, lane 2
Photo 5—New Jersey Turnpike interchange No. 14—15 yr closeup of EPC, lane 1

Photo 6—ODOT bridge deck—surface preparation, abrasive blasting
Photo 7—ODOT bridge deck—surface preparation, closeup

Photo 8—ODOT bridge deck—EPC placement
Photo 9—ODOT bridge deck—10 yr EPC with truck traffic
Epoxy Asphalt Concrete—
A Polymer Concrete
with 25 Years’ Experience

by Robert W. Gaul

Synopsis: Epoxy Asphalt Concrete is a polymer concrete with a 25 year history of application as a bridge deck surfacing. Since 1967 over 100 million pounds (50,000 tons) have been installed on twenty-two bridge decks totaling 6.5 million square feet. Most installations have exhibited excellent performance. The Epoxy Asphalt binder is a two phase, thermoset chemical system in which the continuous phase is an acid cured epoxy and the discontinuous phase is a mixture of asphalts. The aggregates and gradation are similar those used in asphalt concrete. The Epoxy Asphalt binder components are pre-mixed before being combined with the heated aggregate in a conventional asphalt batch plant and applied through conventional asphalt paving equipment. Epoxy Asphalt Concrete has found use as a pavement for new orthotropic steel bridge decks and as an overlay for existing concrete bridge decks. Epoxy asphalt has also been applied as a chip-seal. On one project the epoxy asphalt concrete was shop applied to steel plates that were later installed as a bridge deck. Several installations have not performed as expected. Successful installations require close temperature control of aggregates and careful attention to early compaction. This paper also provides a history of the commercial use of Epoxy Asphalt in the United States and Canada.

Keywords: Aggregates; bituminous concretes; bridge decks; compaction; epoxy resins; polymer concretes; temperature
WHAT IS EPOXY ASPHALT CONCRETE?
Epoxy Asphalt Concrete is a polymer concrete that is composed of a slow curing, Epoxy Asphalt binder mixed together with hot, standard asphalt concrete aggregates in the pug mill of an asphalt plant. The Epoxy Asphalt binder is a two phase chemical system in which the continuous phase is an acid cured epoxy and the discontinuous phase is a mixture of asphalts. A hot spray application of an Epoxy Asphalt bond (tack) coat precedes the laying of the Epoxy Asphalt Concrete. The bond coat is a more viscous version of the binder. The Epoxy Asphalt concrete is applied and compacted with conventional asphalt concrete paving equipment. The pavement is ready for traffic in its partially cured state once it has cooled to ambient temperature. It develops full cure over two to six weeks depending on ambient temperatures.

Shell Oil Company developed Epoxy Asphalt in the late 1950's as a jet fuel and jet blast resistant specialty pavement for airfield applications. The product enjoyed technical, but not commercial success. In 1967 Adhesive Engineering Company, under license from Shell Oil, supplied Epoxy Asphalt for the first commercial application on a bridge deck which was the paving of the mile long orthotropic steel deck span of the San Mateo-Hayward Bridge across San Francisco Bay.
CHEMICAL PROPERTIES OF BINDER AND BOND COAT

The chemical properties of the epoxy resin component and the curing agent/asphalt component of the Epoxy Asphalt binder and bond coat are listed in Table I.

<table>
<thead>
<tr>
<th>PROPERTY</th>
<th>TEST VALUE</th>
<th>TEST METHOD</th>
</tr>
</thead>
<tbody>
<tr>
<td>PART &quot;A&quot; RESIN COMPONENT</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Epoxide Equivalent</td>
<td>185 to 192</td>
<td>185 to 192</td>
</tr>
<tr>
<td>Viscosity @ 73°F, cps</td>
<td>10,000 to 16,000</td>
<td>10,000 to 16,000</td>
</tr>
<tr>
<td>Minimum Flash Point °F</td>
<td>400</td>
<td>400</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>1.16 to 1.17</td>
<td>1.16 TO 1.17</td>
</tr>
<tr>
<td>PART &quot;B&quot; CURING AGENT/ASPHALT</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Acid value, mg. KOH</td>
<td>44 to 55</td>
<td>44 to 55</td>
</tr>
<tr>
<td>Viscosity, cps @ 210°F</td>
<td>140 to 250</td>
<td>1200 to 1600</td>
</tr>
<tr>
<td>Flash Point, °F, min.</td>
<td>400</td>
<td>520</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>1.00 to 1.01</td>
<td>1.00 to 1.01</td>
</tr>
</tbody>
</table>

PHYSICAL PROPERTIES -- BINDER AND BOND COAT

The two phase Epoxy Asphalt binder is a thermoset polymer, i.e. it will not melt after it has cured, as opposed to a thermoplastic polymer, such as conventional asphalt, which will readily melt at elevated temperatures.

Because most of the service temperature range of bridge decks is above it's Heat Deflection Temperature (a rough measure of the glass transition temperature), Epoxy Asphalt softens at elevated temperatures. However, even though it softens, it retains its thermoset character. Typical physical properties of the Epoxy Asphalt binder and bond coat are listed in Table II.

<table>
<thead>
<tr>
<th>PROPERTY</th>
<th>TEST VALUE</th>
<th>TEST METHOD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile Strength, psi, @ 73°F, min.</td>
<td>200</td>
<td>700</td>
</tr>
<tr>
<td>Tensile Elongation, %, @ 73°F, min.</td>
<td>175</td>
<td>175</td>
</tr>
<tr>
<td>Water Absorption, %, 7 days @77°F, max.</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>Heat Deflection Temp., °F</td>
<td>-13 to 0</td>
<td>0 to 5</td>
</tr>
</tbody>
</table>
PHYSICAL PROPERTIES OF EPOXY ASPHALT CONCRETE

Because Epoxy Asphalt pavements are usually opened to traffic before the chemical reaction of the resin and curing agent is completed, both the initial physical properties and the final, cured properties are important. The initial properties are similar to those of an asphalt pavement. See Table III.

### TABLE III — TYPICAL PROPERTIES — EPOXY ASPHALT CONCRETE

<table>
<thead>
<tr>
<th>PROPERTY</th>
<th>TEST VALUE</th>
<th>TEST METHOD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>EPOXY</td>
<td>EPOXY</td>
</tr>
<tr>
<td></td>
<td>ASPHALT</td>
<td>ASPHALT</td>
</tr>
<tr>
<td></td>
<td>AFTER</td>
<td>AFTER</td>
</tr>
<tr>
<td></td>
<td>INITIAL</td>
<td>FINAL</td>
</tr>
<tr>
<td>Marshall Stability @ 140°F, lb.</td>
<td>1200 to 1400</td>
<td>8000 to 14000</td>
</tr>
<tr>
<td>Flow Value @ 140°F, inches</td>
<td>.08</td>
<td>.08</td>
</tr>
<tr>
<td>Recovery %, min.</td>
<td>0</td>
<td>60</td>
</tr>
<tr>
<td>Marshall Stability @ 400°F, lb.</td>
<td>4000</td>
<td>Melts</td>
</tr>
<tr>
<td>Compressive Properties @ 77°F</td>
<td>n/a</td>
<td>3400</td>
</tr>
<tr>
<td>Strength, psi</td>
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<td></td>
</tr>
<tr>
<td>Modulus of Elasticity, psi</td>
<td>n/a</td>
<td>167,000</td>
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<tr>
<td>Flexural Properties @ 77°F</td>
<td>n/a</td>
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<td>Modulus of Rupture, psi</td>
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<td>Modulus of Elasticity, psi</td>
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<tr>
<td>Maximum</td>
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<td>Deflection, inches</td>
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<td>Air Voids, %</td>
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</tbody>
</table>

Unlike conventional asphalt concrete, Epoxy Asphalt which cures to a thermoset state is susceptible to rutting only in its early stages of cure, usually a few days. Thus, the mix can be designed with a low void content without the danger of bleeding or flushing. The result is an impervious pavement. Permeability test data are reported in Table IV. (Ref 1).

### TABLE IV — CHLORIDE ION PERMEABILITY — AASHTO T 259

<table>
<thead>
<tr>
<th>Epoxy Asphalt Pavement Thickness - Inches</th>
<th>Concrete Penetration Depth - Inches</th>
<th>Percent Cl⁻ by Weight of Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4 *</td>
<td>1/16 to 1/2</td>
<td>&lt;.003</td>
</tr>
<tr>
<td>3/4 *</td>
<td>1/2 to 1</td>
<td>&lt;.003</td>
</tr>
<tr>
<td>1**</td>
<td>1/16 to 1/2</td>
<td>&lt;.003</td>
</tr>
<tr>
<td>1**</td>
<td>1/2 to 1</td>
<td>&lt;.003</td>
</tr>
</tbody>
</table>

* Tack Coat @ .225 gal/sq. yd.
** Tack Coat @ .15 gal/sq. yd.
Prior to 1975 all Epoxy Asphalt installations used the binder as a tack coat. It was observed that the tack coat had a tendency to wick up into the paving mat leaving little or no continuous film on the deck. This resulted in reduced bond strength. After 1975, a more viscous version of bond coat was introduced which overcame this problem. The level of bond strength of the pavement system to the deck that is achieved using the more viscous bond coat is indicated in Table V.

**TABLE V — TYPICAL PULL-OFF STRENGTHS**

<table>
<thead>
<tr>
<th>SUBSTRATE</th>
<th>TEST TEMPERATURE</th>
<th>PULL-OFF STRENGTH</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland Cement Concrete</td>
<td>73°F</td>
<td>360 psi (PC)</td>
</tr>
<tr>
<td>Steel -Coated with Inorganic Zinc</td>
<td>77°F</td>
<td>520 psi (B)</td>
</tr>
<tr>
<td></td>
<td>160°F</td>
<td>140 psi (B)</td>
</tr>
<tr>
<td>Steel -Coated with Inorganic Zinc after 5,200,000 flexural fatigue cycles</td>
<td>77°F</td>
<td>440 psi (B)</td>
</tr>
</tbody>
</table>

* Test Method: ACI 503, Appendix A.

(PC): Portland Cement Concrete failure

(B): Bond failure

**MIX DESIGN**

The determination of binder content and specific aggregate gradation for dense graded Epoxy Asphalt is made using the Marshall method, as is used in conventional asphalt concrete mix design. A series of binder contents is tested in order to optimize the Marshall Stability while maintaining a void content under 3%. The aggregate gradation required is described in Table VI. The mix contains a relatively high amount of fines. Bag house dust is limited to 1% of the total aggregate and the addition of ground limestone dust is allowed up to a maximum of 4% of the total aggregate.

**TABLE VI — AGGREGATE FOR DENSE GRADED EPOXY ASPHALT CONCRETE**

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>100</td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>25-100</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>80-95</td>
</tr>
<tr>
<td>#4</td>
<td>58-75</td>
</tr>
<tr>
<td>#8</td>
<td>43-60</td>
</tr>
<tr>
<td>#30</td>
<td>20-35</td>
</tr>
<tr>
<td>#200</td>
<td>7-14</td>
</tr>
</tbody>
</table>

**COST**

The installed cost of Epoxy Asphalt varies significantly depending on job conditions including:

- Size of the job
- Number of working hours available each day
- Night work vs. days
- Thickness of pavement
- Traffic control
The range of installed costs, excluding surface preparation and traffic control, on all projects since 1967, in 1993 dollars, has been approximately $4.00 to $8.00 per square foot.

**HISTORY OF APPLICATIONS OF EPOXY ASPHALT**

Epoxy Asphalt concrete has been used in dense graded, open graded and chip seal forms and on both steel and concrete decks. It has been successfully used in climates where winter temperatures fall below 0°F and where deck temperatures reach 140°F in the summer.

*Orthotropic Steel Decks - New Bridges*

Epoxy Asphalt concrete found its first commercial application because conventional paving materials were either too rigid or did not have the fatigue resistance required to resist cracking on very flexible orthotropic steel decks. In 1967 the San Francisco Bay Toll Crossings selected Epoxy Asphalt for the San Mateo Hayward orthotropic span *(Fig. 1)* after an extensive laboratory and field test program that evaluated fatigue resistance and constructability of asphalt, epoxy and Epoxy Asphalt concretes. While the epoxy concretes demonstrated adequate fatigue resistance in the Bay Toll Crossings test program, at that time in the late 1960's, construction equipment and techniques to install large quantities were not available.(Ref 2, 3)

Since the 5/8" steel deck plate of this bridge acts as the top flange of the two parallel box girders which are the main supporting members of the 5500 foot long 80 foot wide orthotropic span, corrosion protection was also a concern. An inorganic zinc prime coat was shop-coated on the deck plate to provide added protection in the event that inadequate compaction resulted in a degree of permeability that would allow water to reach the deck plate. The pavement was installed in two lifts, a 1-1/8" leveling course which covered the bolt heads over the deck splice plates and a wearing course which brought the total thickness to 2".

The San Mateo-Hayward bridge deck is in excellent condition today after 26 years of service. One longitudinal crack in the pavement appeared over the outer web of the box girder in the early 1980's, but no loss of bond has resulted. Since 1967 seven other new orthotropic bridges have utilized Epoxy Asphalt and two of these have been repaved with Epoxy Asphalt after 20 years of successful service. The pavement on the orthotropic decks of two of the seven other new bridges have not performed well for specific reasons discussed in "Critical Factors for Success" below.

The San Diego-Coronado Bay Bridge, repaved with Epoxy Asphalt in early 1993, was paved as a new bridge in 1969 with Epoxy Asphalt. The Epoxy Asphalt on this very flexible 3/8" thick steel deck began to crack in 1979, but remained a serviceable pavement until the 1993 repaving project *(Fig. 2)*. Another 3/8" steel decked bridge, the McKay Bridge in Halifax, N.S. was first paved, when new, with Epoxy Asphalt in 1971. The first cracks appeared in 1975 and eventually about 25% of the deck was replaced with a variety of asphalt concretes in areas where bond was lost. None of the replacement materials proved satisfactory. The Epoxy Asphalt that was left on the deck remained serviceable until the deck was entirely repaved with Epoxy Asphalt in 1989.
Orthotropic Steel Replacements of Concrete Decks

Epoxy Asphalt has also found application as the surfacing for orthotropic plates that have replaced existing concrete decks without taking the bridge completely out of service. In the first of these projects, in 1975, 15/32" orthotropic steel plates were paved with 1-1/2" of Epoxy Asphalt on the ground prior to placement on 77,000 sq. ft. of the north viaduct of the Lions Gate Bridge in Vancouver, BC. (Ref 4)

On the Golden Gate Bridge in San Francisco, orthotropic steel deck replacement plates were shop coated with an Epoxy Asphalt chip seal which provided skid and wear resistance until all of the approximately 800 plates were installed over a two year period, 1986-1987. The chip seal was similar to conventional asphalt chip seal with the chips being rolled into the liquid Epoxy Asphalt bond coat resulting in a 3/8" temporary wearing surface (Figs. 3,4, 5). A 1-5/8" thick Epoxy Asphalt concrete placed on top of the chip seal at the end of the two year project provides the final wearing course. This 575,000 square foot complete deck replacement was accomplished with no lane closures between 5 AM and 9 PM and always at least one lane in each direction open from 9 PM to 5 AM.

In a similar application a 1-1/4" pavement was shop applied on orthotropic plates for the 632,00 sq. ft. deck on the Ben Franklin Bridge in Philadelphia in 1986. (Fig. 6) In this case the Epoxy Asphalt served as a permanent impervious membrane and provided the wearing surface until a conventional asphalt concrete wearing surface was placed once all the plates were installed. (Ref 5)

All three of these applications of Epoxy Asphalt have performed very well and show no distress, although the Lions Gate is showing normal wear of approximately 1 mm/year after 18 years.

The Champlain Bridge in Montreal, Quebec is currently undergoing a deck replacement that utilizes an Epoxy Asphalt chip seal as the temporary riding surface in the same manner as the Golden Gate Bridge, but uses a conventional asphalt pavement as the permanent wear course. Since this deck is constructed with very stiff steel plates the designers do not expect fatigue cracking of the asphalt pavement.

Overlays on Concrete Decks

The first overlay on a concrete bridge deck was an open graded (high void content after compaction) 1/2" thick pavement on 155,000 sq. ft. area of the west end of the upper deck of San Francisco-Oakland Bay Bridge where there had been skid resistance and hydroplaning problems. Although some raveling occurred before the pavement fully cured, the application well achieved its purpose of allowing water to drain into the pavement thus preventing hydroplaning and stayed in place until the entire bridge was paved with Epoxy Asphalt in 1976/77.

There have been seven other overlays on concrete bridge decks, all more densely graded (low void content) mixes. The 1/2" overlay of 270,000 sq. ft. on the Evergreen Point Bridge across Lake Washington in 1972 and the 1/2" pavement on the 146,000 sq. ft. Ross Island Bridge in Portland, OR in 1972 performed very well, but the 1/2" overlay of the 119,000 sq. ft. MacDonald bridge in Halifax, NS in 1971 failed after a few
years (see "Critical Factors for Success", below). Overlays of less than 3/4" are no longer installed.

In perhaps the largest polymer concrete overlay ever the San Francisco-Oakland Bay Bridge was surfaced with a 3/4" thick Epoxy Asphalt concrete that used a combination of very hard metagraywackie aggregate and a soft, light weight expanded shale to provide differential wear in order to develop a macro-texture in the pavement for improved skid resistance. The 1,475,000 sq. ft. upper deck was completed in 1976 and the 1,290,000 sq. ft. lower deck a year later. This pavement remains in excellent condition today. (Ref 6)(Fig. 7)

APPLICATION METHODS

Epoxy Asphalt handles like conventional asphalt except that its two liquid components must be mixed together before being used and they have a limited working life after being mixed together. After the components are mixed, conventional asphalt equipment and installation techniques are used.

Bond Coat and Chip Seals

The bond coat components are heated (180° F for the resin and 300° F for the curing agent/asphalt) prior to mixing. The truck mounted machine that sprays the bond coat utilizes centrifugal pumps, controlled by mass flow meters, that pump the bond coat through a static mixing head. From the mixing head the mixed bond coat passes through a reactor that advances the cure to provide the proper tack on the bridge deck. A workman walks behind the spray machine applying the bond coat with a spray-wand. (Fig. 8) The wand is used in preference to a spray bar to better cover all deck depressions and protrusions. The bond coat is applied at a rate of .15 gal/sq. yd. (approx. 30 mils) on concrete or steel and at .10 gal/sq./yd. (approx. 20 mils) when used as a bond coat between lifts of Epoxy Asphalt pavements. For chip seal applications the application rate is .25 gal/sq. yd. (approx. 50 mils).

Manufacture of Epoxy Asphalt Concrete

At the asphalt batch plant a Meter/Mix machine pumps and meters the heated components, which are continuously circulating through insulated lines, out of and back into heated reservoirs. (Figs. 9) At the command of an electronic control system, the two components are diverted through a four way valve to a dynamic mixing head and into the pug mill. The control system redirects the components into the recirculating mode once the proper amount of mixed binder for each batch has been pumped into the pug mill. (Fig. 10) A typical batch size can vary between 2 and 4 tons. The batches are loaded into trucks which usually carry 10 to 12 tons (Fig. 11).
The Epoxy Asphalt concrete must remain in the truck before it is dumped into the paving machine to advance the cure enough for the viscosity of binder to reach 1000 cps. which is the optimal viscosity for compaction. However, the mix must be emptied from the truck before the chemical reaction between the two binder components increases the viscosity to the point where the mix cannot be adequately compacted. This "haul time window" depends on the average batch temperature of the truck load. At a temperature of 240°F the minimum haul time is 42 minutes and the maximum is 62 minutes. At 260°F this window is from 35 to 43 minutes.

**Paving Operations**

Once the mix is dumped into the paving machine the paving operation proceeds just as if conventional asphalt concrete were being used. Breakdown rolling must be completed before the temperature of the mat has fallen below 180°F and finish rolling must be completed before mat temperature has reached 150°F in order to achieve adequate compaction. (Fig. 12) Ambient and deck temperatures must be at least 50°F during paving operations.

**Opening to Traffic**

About two hours after finish rolling the pavement temperature will have dropped to near ambient temperature (except in direct sunlight) which provides enough stability so that it can be opened to traffic without rutting even though the Epoxy Asphalt binder has not fully cured. However, in hot weather and under heavy traffic during the two to six week cure period some flushing can occur which results in a slick surface. This Epoxy Asphalt binder on the surface is usually worn off by traffic, but can also be removed by grit blasting. The flushing will not occur again because cured Epoxy Asphalt will not flow. Similarly, as with portland cement and asphalt concrete pavements, grinding will correct the variations in elevation of any areas of the pavement that are not within the specified grade tolerances.

**CRITICAL FACTORS FOR SUCCESS**

Like all polymer concretes, Epoxy Asphalt installations require:

- Proper design
- Selection of appropriate aggregates
- Strict compliance with component ratio and mixing requirements
- Close attention to established installation procedures.

In addition, there are several specific factors that have led to the few instances where performance of Epoxy Asphalt has not lived up to expectations:

**Fatigue Testing for Orthotropic Decks**

On orthotropic steel decks all pavement are susceptible to fatigue cracking in the negative moment areas over the longitudinal stiffeners. Fatigue resistance of paving materials on orthotropic decks can only be predicted by conducting tests which simulate the deflections and radius of curvature that the pavement will see on the bridge deck under heavy traffic. For many polymer concretes, including Epoxy Asphalt, the most severe conditions exist at the highest service temperatures expected because at low temperatures the increased stiffness of the polymer concrete results in reduced...
deflections in the composite (steel/polymer concrete) deck and thus smaller strains in the pavement surface.

The exact mix design, including the actual proposed aggregate, must be used in the test. Where cracking problems have been encountered, the aggregate choice and mix design were not tested in advance for their influence on fatigue resistance. Failure to conduct fatigue tests lead to early cracking and localized pavement deterioration on some sections of the Rio-Niteroi Bridge in Rio de Janeiro, paved in 1973. Jobsite changes in mix design (substantial increase in binder content) in order to make the paving mix easier to compact lead to early pavement deterioration on the Luling Bridge which crosses the Mississippi River near New Orleans. It was paved in 1983. Cracking and localized pavement breakdown occurred within the first year.

Fatigue testing requires special equipment and even accelerated tests consume weeks rather than days. However, the time and expense are insignificant compared to the benefits of a longer lived pavement that requires minimal maintenance.

Temperature Control of Batch Plant
The temperature of the Epoxy Asphalt batches as they are discharged from the batch plant is determined by the temperature of the aggregate fed to the pug mill. If this temperature varies by as much as 15 or 20 degrees from batch to batch there is a danger of part of one truck load being too high in viscosity (too far advanced in cure) which results in some areas of the pavement not being compacted enough. Inadequate compaction can results in a permeable pavement and accelerated wear.

Prompt Rolling of Mat
As the temperature of the Epoxy Asphalt drops after it is laid by the paving machine its viscosity increases. If the roller compaction operation is delayed so that the breakdown rolling is not completed before the mat temperature has fallen below 180°F and the finish rolling before the temperature reaches 150°F, the viscosity will be so high that complete compaction will be impossible. No amount of subsequent traffic will solve the problem because the binder will have cured to a solid that will not flow. Failure to observe these time limits lead to greater than normal wear on the Fremont Avenue Bridge orthotropic steel deck in Portland, Oregon in 1973. Fortunately, the commercialization of hand-held infrared thermometers in recent years now makes measurement of the mat temperature practical and provides an excellent method of controlling the timing of the compaction process.

Pavement Thickness
Although there have been several long lasting concrete bridge deck installations with a 1/2" thickness, there have also been problems with 1/2" mat tearing when it is rolled, especially with dense graded mixes. Tearing of the mats has resulted in permeability. For this reason, installations of less than 3/4" are not recommended.

Sound Concrete Substrate for Overlays
Like all other overlays for concrete pavements, Epoxy Asphalt depends on the quality of the underlying concrete if the installation is to be long lasting without extensive maintenance. The application of an Epoxy Asphalt overlay on MacDonald Bridge in
Halifax N.S. in 1971 was not successful because of continued concrete deterioration. The deterioration was the result of high chloride content that remained in the deck despite the removal of the top 1/2" of concrete prior to installation of the Epoxy Asphalt. In addition to not having a deleterious level of salt, the concrete should have a minimum 200 psi tensile pull-out strength.

SUMMARY
Epoxy Asphalt Concrete has a history spanning 26 years of application as a surfacing for orthotropic steel bridge decks and as an overlay for portland cement concrete decks. While most applications have provided very satisfactory, minimum maintenance service life, several have not. In order to assure satisfactory performance:

- Fatigue tests on the proposed mix design must be carried out when the application is on an orthotropic steel deck and, if the tests are successful, no significant changes in the mix design should be made during the course of the job,
- Careful attention must be paid to temperature control during manufacture of the Epoxy Asphalt concrete,
- Compaction must be completed before the temperature of the paving mix has dropped below the temperature at which further compactive effort is ineffective.
- Minimum pavement thickness should be 3/4" and concrete substrates must be sound.

CONVERSION FACTORS

1 lb = 4.448 N
1 sq ft = 0.0929 m²
1 in = 25.4 mm
1 mil = 0.0254 mm
1 cps = 0.001 Pa·s
°F = (1.8)(°C) + 32
1 psi = 0.006895 MPa
1 gal/sq yd = 0.263 litre/m²

REFERENCES
2. Balala, B., Studies Leading to Choice of Epoxy Asphalt for Pavement of Steel Orthotropic Bridge Deck of San Mateo-Hayward Bridge, Highway Research Record No. 287
ADDITIONAL REFERENCES


Fig. 1—San Mateo Hayward Bridge paving
Fig. 2—Repaving San Diego-Coronado Bay Bridge

Fig. 3—Spraying epoxy asphalt for Golden Gate chip seal
Fig. 4—Spreading chips on Golden Gate Bridge orthotropic plates

Fig. 5—Finished chip seal on Golden Gate plates
Fig. 6—Ben Franklin Bridge orthotropic panels being paved in shop

Fig. 7—San Francisco-Oakland Bay Bridge paving
Fig. 8—Tack coat spray wand application

Fig. 9—Epoxy-asphalt liquid supply system at batch plant
Fig. 10—Epoxy asphalt concrete being discharged from pug mill
Fig. 11—Batch being loaded into haul truck
Fig. 12—Rollers and temperature measurement of epoxy asphalt mat
Design of Buried Gas Distribution System Vaults Made with Fiber Reinforced Polymer Concrete

by R. P. Webster, C. A. Miller, and J. J. Fontana

Synopsis: A methodology has been developed for designing precast, fiber-reinforced polymer concrete (FPC) vaults to be used in underground applications. The approach used in the design was to consider the vault as a series of plates: cover, walls, and foundation slab. Each plate was subjected to loads resulting from soil pressure, live loading, and dead weight and was analyzed using classical plate theory. This approach was verified by testing two quarter-scale models of a typical vault. Upon completion of the laboratory evaluation, two vaults were designed for use as underground, natural gas-regulator stations. The vaults were manufactured and subsequently placed into service by the Brooklyn Union Gas Company and the Consolidated Edison Company of New York.

Keywords: Fiber-reinforced concretes; polymer concrete; structural design; vaults
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C. A. Miller is the Chairman of the Department of Civil Engineering at The City College of New York, New York, NY.

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RESEARCH SIGNIFICANCE

Precast vaults for natural gas regulator stations used by the Brooklyn Union Gas Company (BUG), Brooklyn, NY and the Consolidated Edison Company of New York (ConEd) have traditionally been fabricated with steel-reinforced portland cement concrete (PCC). When used to replace existing vaults it is often necessary to off-set the gas lines to align them with the penetrations in the vault or field-cut new penetrations in the vault which could subsequently weaken the structure and/or allow water ingress. The need to cut new penetrations in the field can be costly since it involves additional labor during the installation of the vaults. In addition, some of the PCC vaults weigh in excess of 25,000 lb, requiring the use of large cranes during installation.

With the advent of advanced materials of construction, such as polymer concrete (PC), it may be possible to cast vaults that have a longer service life resulting from the superior physical and durability characteristics of these materials. In addition, the higher strength values of PC should allow the designer to reduce the wall thicknesses, making the vaults lighter for easier transportation and installation. Field penetrations can be cut after casting to match existing street lines.

To meet this need the Process Materials Group of the Energy Efficiency and Conservation Division at Brookhaven National Laboratory has developed several
fiber-reinforced, polymer concrete (FPC) systems which can be used as construction materials for precast, underground vaults [1][2]. One such system is an orthophthalic polyester resin FPC system. The mix design of the system is as follows:

10 wt% polyester resin
54 wt% blended silica sand
27 wt% graded coarse aggregate (1/2 in. max.)
9 wt% 1.5-in. long corrugated, carbon steel fibers

Typical properties of this system, at 72°F, are summarized below.

Flexural yield strength = 3500 psi.
Ultimate flexure strength = 4685 psi.
Flexural modulus of elasticity = 2.45 x 10^6 psi.
Compressive strength = 16,005 psi.
Density ≈ 150 lb/cu. ft.
Water absorption = 0.12 wt. percent.

A typical flexure load versus deflection curve for this system is presented in Figure 1.

DESIGN APPROACH

Stress Evaluation

A vault was assumed to consist of six plate elements; four walls, a foundation slab, and a cover. Classical plate theory was used to evaluate the bending stresses in each plate.

Because the vaults will be cast monolithically (or in some cases sections of the vaults are connected with joints having strengths equivalent to a monolithic pour), it was assumed that the boundary conditions of adjoining plates could be taken as fixed. Therefore, all foundation slab and walls were designed as having fixed boundary conditions on all sides. The edge adjoining the cover to the top of the walls was initially assumed to be fixed. If the FPC cover is adequately bonded, the wall-cover joint will act as fixed. If not, the edges of the cover and the top of the walls should be treated as being simply supported.

Equations for each of these load cases are given in tabular form by Szilard [3] and were used in the design of the vaults.
Loading Conditions

The loads acting on a typical vault contain three components:

1. The dead weight of the vault.
2. The effect of soil pressure on the vault (the density of the soil was taken to be 110 lb/cu ft).
3. A live load (highway) component that is taken as the most severe of the following loading conditions:
   a. A uniform vertical load of 600 psf (as required by the New York City (NYC) building code).
   b. The HS-20 AASHTO loading, which consists of the more severe of a uniform lane load of 640 lb/ft distributed over a 10 ft lane width, or two concentrated wheel loads of 20,800 lb each (including a 30 percent impact factor) spaced at 5 ft.
   c. A 15 cu yd concrete truck; the rear wheels of the truck produces four concentrated loads of 20,250 lb spaced at 5 ft in one direction and 6 ft in the other direction.

The loading on the walls and foundation slab were taken to be uniform and the loading on the cover was taken to be either a uniform load or a point load applied at the center of the plate, which ever was more severe. A diagram of the loading conditions is presented in Figure 2.

The forces acting on each element of the vault are discussed below.

Forces on the cover -- The cover was loaded with the dead weight of the cover slab plus any soil overburden applied as a uniform load on the cover plate. Because the cover will generally be close to the surface (the cover is usually about one foot below the ground surface), the highway loading was applied directly on the cover. The cover was designed for both a uniform live load of 600 psf and a single wheel load of 20,800 lb placed at the center of the cover.

For vaults that have plan dimensions greater than 10 ft, consideration should be given to placing the two or four wheel loads (depending on whether the box is designed for HS-20 or the 15 cu yd concrete truck) on
the cover in a position leading to the highest bending moments.

**Forces on the walls** -- The loading on the walls, which are in contact with the soil, was taken as the soil pressure component (resulting from soil weight and highway loading) that is normal to the surface of the plate (i.e., the loading on the vertical plates is equal to the horizontal soil pressure at the depth of interest). In all cases, the horizontal soil pressure was taken as one-half of the vertical pressure. While the actual loading on the wall will vary over the depth of the wall, a uniform pressure equal to the average loading applied to the wall was used.

The loading component of the soil pressure due to its own weight is:

\[ p_s (\text{psf}) = \frac{\gamma z}{2} \]  

(Eq. 1)

where: \( \gamma \) = soil density, lb/cu ft  
\( z \) = depth below the surface, ft

The loading component for the 15 cu yd concrete truck was computed by converting the four wheel loads to a uniform load applied over an area of 5 ft by 6 ft, and spreading the load by 45 degrees as the depth from the surface increased. The pressure acting on the wall is one-half of the vertical component of the soil pressure. Therefore, the pressure acting on the wall is:

\[ P_{CT} (\text{psf}) = \frac{81,000}{[(2)(5+2z)(6+2z)]} \]  

(Eq. 2)

The loading component for the HS-20 loading will be critical for the two wheel load case. The loading was taken to act on an area of 6 ft by 1 ft on the surface and to spread out at a 45 degree angle with depth. Therefore, the pressure acting on the wall is:

\[ P_{HS20} (\text{psf}) = \frac{20,800}{[(2)(6+2z)(1+2z)]} \]  

(Eq. 3)

The uniform pressure of 600 psf was not attenuated with depth so that the resulting load on the wall is:

\[ P_{NYC} (\text{psf}) = 600/2 = 300 \text{ psf} \]  

(Eq. 4)

**Forces on the foundation slab** -- The forces on the foundation slab were taken as a uniform pressure equal to the weight of the vault plus the weight of the soil over the cover plus the total highway load acting on the cover divided by the area of the slab. Therefore,
p_{fs} (psf) = \gamma h + [(W_{b} + W_{h})/\Lambda] \quad (Eq. 5)

where: h = depth of the soil over the roof, ft
W_{b} = weight of the vault, lb
W_{h} = 81,000 lb for the concrete truck
= 41,600 lb for the HS-20 loading
= 600 psf x A for the NYC building code
\Lambda = plan area of the foundation slab, sq ft
\gamma = soil density of 110 lb/cu ft

This pressure must be less than the allowable soil bearing pressure. If it is not, the plan dimensions of the manhole must be increased.

Material Properties

The properties of FPC can be adjusted by the proper selection of resin type, resin content, aggregate type, and fiber characteristics. The most significant properties of the FPC that are used for design are the strength and the ductility of the system when subjected to flexural loads. Flexure data for the polyester FPC system was obtained by performing modulus of rupture tests (ASTM C78), to failure, on 2-in. by 2-in. by 12-in. long beams. Information obtained from these tests included: flexure yield strength, ultimate flexure strength, flexural modulus of elasticity (E), and midspan deflection data. The flexure yield strength (f_{R}) is defined as the stress corresponding to the point at the end of the linear portion of the load vs. deflection curve. One-half of this value (f_{R}/2) is used as the maximum allowable design stress.

EXPERIMENTAL VERIFICATION

To verify the validity of the design approach discussed above two quarter-scale model vaults were constructed and tested. The center-to-center dimensions of the vaults were 23.0-in. by 17.5-in. by 17.5-in. high with wall and cover thicknesses of 1.0-in. and a floor thickness of 1.5-in. The walls and floor of the model were cast monolithically. The cover was cast separately and bonded to the top of the model vault with an epoxy adhesive. Quality control testing of the material used to make the model vaults indicated that the polyester FPC system had a flexural yield strength of 3580 psi, a flexural modulus of elasticity of 2.45 x 10^{6} psi, and a shear strength of 800 psi. The epoxy adhesive used to
bond the cover to the vault had a shear strength of 1000 psi.

The tests on the quarter-scale vault were conducted by pulling a vacuum on the model over a period of 6 hr. The forces created by the vacuum simulated the external forces that a typical vault would be subjected to while in service. The vacuum was applied through fittings located in the cover of the model. A series of strain gauges were attached to the exterior of the vault to monitor its behavior while under test. Dial indicators were used to measure the centerpoint deflections of the walls.

The maximum vacuum that could be achieved was 170 mm of mercury resulting in a peak load of 1643 psf. Both model vaults were subjected to a complete load-unload cycle and the following results were observed.

1. There was no visible cracking of the FPC model. It should be noted that any significant cracking would have made it impossible to achieve and maintain a vacuum.

2. The measured strains and deformations in the box indicated linear behavior with complete recovery. The deflection at the center of the largest sidewall increased from 0.0015 in. at a vacuum of 600 mm (446 psf) to 0.0053 in. at a vacuum of 200 mm (1560 psf). The measured deflection on the unloading side was 0.0019 in. at a vacuum of 600 mm (446 psf).

Typical stress-strain curves for the load cycle on the end and side walls are presented in Figures 3 and 4.

The tabular solutions contained in Reference 3 were used to perform an analysis of the model vaults. Assuming that all sides are fixed, Case 80 indicates that the maximum bending moment (which occurs at the center of the long support) and the center deflections are:

\[ M = 0.069p_0a^2 \]

\[ \Delta = 0.022p_0a^4/\pi t^3 \]

where: \( p_0 \) = the applied uniform load
\( a \) = length over which the load is applied
\( t \) = thickness of the member
Therefore at the peak pressure of 11.41 psi (1643 psf) the predicted values are:

\[ M = 0.069 \times 11.41 \times 17.5^2 = 241.1 \text{ in.-lb/in.} \]
\[ \Delta = 0.022 \times 11.41 \times 17.5^4 / 2,450,000 \times 1^3 \]
\[ = 0.0096 \text{ in.} \]

The maximum stress in the plate is therefore:

\[ f = 6M/t^2 = 6 \times 241.1/1^2 = 1,447 \text{ psi} \]

This stress is below the measured flexural yield stress of 3580 psi so that elastic action, as observed, would be expected. In addition, the observed deflection of 0.0053 in. was found to be significantly less than the calculated deflection of 0.0096 in.

One of the model vaults was next modified by passing a 6 in. diameter pipe through the center of the end wall to simulate a gas main, see Figure 5. The pipe was bonded in place using an epoxy adhesive. A second vacuum test was performed and the results were found to be very similar to those obtained for the first test. For example, the center deflection at a vacuum of 200 mm (1560 psf) was found to be 0.0054 in. as compared with 0.0053 in. for the first test.

The model was then loaded to failure with a concentrated load applied through a 9 in. diameter pipe centered around the penetration. The measured load-deflection curve is shown in Figure 6. Note that the yield load was about 7500 lb at a deflection of about 0.046 in. (after correcting for the initial setting of the test apparatus). The ultimate load was 12,500 lb. It is interesting to observe that no catastrophic failure occurred after the ultimate load had been reached. Instead the load dropped off slowly as the deflection increased. It was found that at a deflection of 0.77 in. the sidewall of the model could still support a load of 5000 lb.

A yield line analysis [4] was performed and the predicted collapse load of the plate was calculated to be 21,600 lb for fixed boundary conditions and 10,800 lb for simply supported boundary conditions. These computations used the actual location of the loads in the experiment and the yield lines did not intersect the penetration so that stiffening of the plate by the steel pipe was not a factor. The fact that the measured collapse load was closer to the simply supported boundary conditions indicates that the polyester FPC may
not be sufficiently ductile to fully develop the plastic hinges along the plate boundary.

DESIGN CALCULATIONS

The above methodology was used to design two gas-regulator vaults, one for the Brooklyn Gas Company (BUG) and one for the Consolidated Edison Company of New York (ConEd). The BUG vault had interior dimensions of 7 ft by 6 ft by 7 ft high. The cover of the vault contained a clear opening of 2 ft 9 in. square in one corner to receive a cast iron manhole. The ConEd vault had interior dimensions of 6 ft 1 in. by 4 ft by 2 ft 8 in. high. A steel butler-type cover was used on the ConEd vault so that it was not necessary to design a FPC cover.

The calculations for the design of the BUG vault are presented below. The design was based on the following assumptions:

1. The top of the manhole is located 1 ft below grade.
2. The design live load will be governed by the loading conditions for the concrete truck or the NYC uniform live load, whichever is more severe.
3. The FPC is assumed to have a minimum flexural yield strength of 3000 psi. One half of this value, or 1500 psi, will be used as the maximum allowable design stress.

Cover Design

The cover is 7.33 ft by 6.33 ft (allowing for centerline to centerline dimensions). The moment equations from Reference 3 for both a uniform load and a concentrated load applied at the center are:

\[ M_{\text{UNIF}} = 0.06p_o a^2 \]

and

\[ M_{\text{Conc}} = 0.1432P \]

The moments due to the uniform loading of 1 ft of soil (110 psf), the weight of the cover (estimated at a thickness of 4 in. = 50 psf), and either the uniform live load of 600 psf or the concentrated wheel live load of 20,800 lb are:
The total design moment is therefore:

\[ M_{\text{total}} = M_{\text{soil}} + M_{\text{pc}} + M_{\text{20800}} \]

\[ = 279.4 + 127 + 2978 = 3384.4 \text{ in.-lb/in.} \]

The required cover thickness is:

\[ t_{\text{cover}} = \left[ \frac{6M_{\text{max}}}{f_{\text{allow}}} \right]^{1/2} = \left[ 6 \times \frac{3384.4}{1500} \right]^{1/2} = 3.7 \text{ in.} \]

A cover thickness of 4 in. was used to account for the manhole cutout. Note that the cover can be analyzed as two plates each with a free edge along the cutout, fixed edges along the other three boundaries, and carrying half the applied loading. The 4 in. thickness is adequate for this case. At the request of BUG the cover thickness was increased to 5 in.

Wall Design

The wall is 7.33 ft square (center to center) resulting in a b/a ratio of 1.00. The bending moment caused by a uniform load is therefore:

\[ M_{\text{unif}} = 0.0513p_a^2 \]

The top and bottom of the wall are 1 ft and 8.5 ft, respectively, below grade so that the soil pressures at the top and bottom of the wall (from Eq. 1 and 2) are:

\[ p_{\text{top}} = 110 \times \frac{1}{2} + \frac{81,000}{2(5+2)(6+2)} = 778 \text{ psf} \]

and

\[ p_{\text{bottom}} = 110 \times \frac{8.5}{2} + \frac{81,000}{2(5+17)(6+17)} = 548 \text{ psf} \]

For an average pressure of 663 psf the maximum bending moment is:

\[ M_{\text{wall}} = 0.0513p_a^2 = 1,913 \text{ in.-lb/in.} \]

The required wall thickness is:

\[ t_{\text{wall}} = \left( \frac{6 \times 1913}{1500} \right)^{1/2} = 2.77 \text{ in.} \]
At the request of BUG a wall thickness of 4 in. was used to allow for installation of expansion bolts for the ladder and internal piping.

Foundation Slab

The foundation slab has the same dimensions as the cover so that the same moment equations apply. The weight of the vault is:

\[
W_{\text{VAULT}} = 150[(6.5)(7.5)(5)/12 + 2(7.5)(7.5)(4)/12 + 2(6.5)(7.5)(4)/12 + (6.5)(7.5)(5)/12] = 16590 \text{ lb.}
\]

The uniform load on the slab is (from Eq. 5):

\[
P_{\text{FOUND}} = 110(1) + (16,590 + 81,000)/(6.5 \times 7.5) = 2112 \text{ psf}
\]

This value is the required soil bearing pressure.

The maximum bending moment is:

\[
M_{\text{FOUND}} = 0.06 p_o a^2 = 5364 \text{ in.-lb/in.}
\]

The required slab thickness is:

\[
t_{\text{FOUND}} = (6 \times 5364/1500)^{1/2} = 4.63 \text{ in.}
\]

A thickness of 5 in. was selected to account for the cutouts for the sumps. A diagram of the vault is shown in Figure 7.

FIELD APPLICATIONS

Upon completion of the design study two vaults were fabricated for use by BUG at a cost of approximately $8,320 per vault. A breakdown of the costs are as follows: design and fabrication of formwork = $2,620, casting of FPC vault (materials and labor) = $5,450, and shipping charges = $250. The vaults were installed in the field in mid-1990. As of July 1994, the vaults have been in service for approximately 4 years with no reports of any problems.

Two vaults were also fabricated for use by the Gas Division of ConEd. These vaults were fabricated at a cost of approximately $2,100 per vault and were
installed in mid-1990. A breakdown of the costs are as follows: design and fabrication of formwork = $985, casting of FPC vault (materials and labor) = $830, and shipping charges = $250. They have been in service for approximately 4 years with no reported problems. ConEd has since had 4 other vaults fabricated at a cost of $800 to $1,000 per vault. These vaults have been in service for approximately 2 to 3 years without any problems.

As discussed, the costs identified above include all materials and labor to construct the molds, all materials and labor to precast the vaults, and all delivery charges. It should be noted that these costs are reflective of those costs associated with the manufacture of a prototype vault. It is anticipated that the cost of manufacturing the vaults will decrease as the volume of production is increased.

SUMMARY

A methodology for designing precast, fiber-reinforced polymer concrete (FPC) vaults has been developed and experimentally verified in the laboratory. The approach used in the design considered the vault to be made up of as a series of plates: cover, sidewalls, and foundation slab. Each plate was subjected to loads resulting from soil pressure, live loading, and dead weight and was analyzed using classical plate theory. The design approach was experimentally verified by testing two quarter-scale model vaults. Upon completion of the laboratory evaluation, two vaults were designed and fabricated for use as underground, natural gas-regulator stations. The vaults were placed into service by the Brooklyn Union Gas Company and the Consolidated Edison Company of New York. These vaults have been in service since mid-1990 with no reported problems.

ACKNOWLEDGEMENTS

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REFERENCES


2. Fontana, J. J. Steel-Fiber Reinforced Polymer Concrete. Polymer Concrete, ACI-SP137, D. Gerry Walters, Ed. American Concrete Institute, Detroit, MI, 1993, pp. 21-27.


CONVERSION FACTORS - INCH-POUND TO SI (METRIC)

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Fig. 1—Typical flexure load versus deflection curve for polyester resin FPC

Fig. 2—Loads acting on a typical underground vault
Fig. 3—Stress-strain curve for a uniform pressure applied to the sidewall of the quarter-scale model.

Fig. 4—Stress-strain curve for a uniform pressure applied to the end wall of the quarter-scale model.
Fig. 5—Typical quarter-scale model after being tested to failure
Fig. 6—Load versus centerpoint deflection curve for a concentrated load applied to the end wall of the model.

Fig. 7—Plans for the BUG FPC gas-regulator vault.