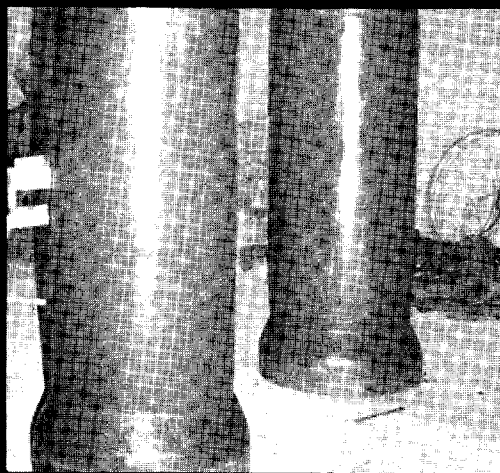
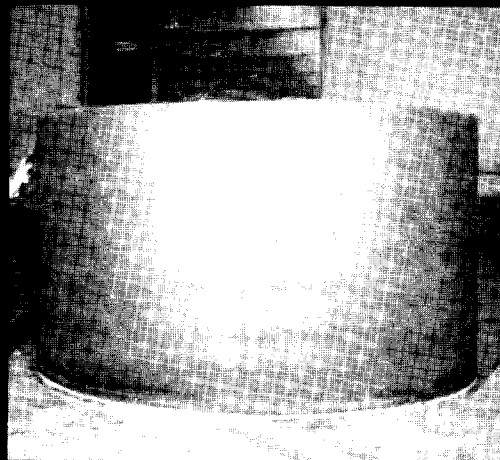
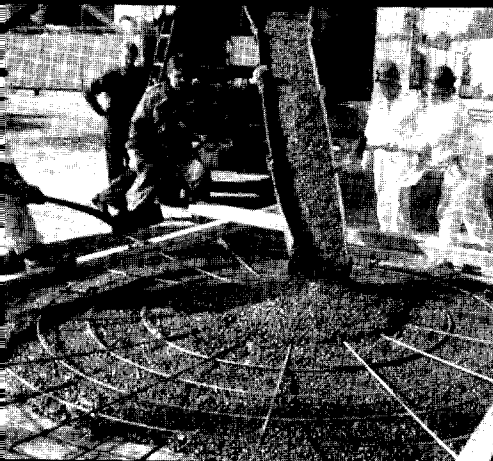


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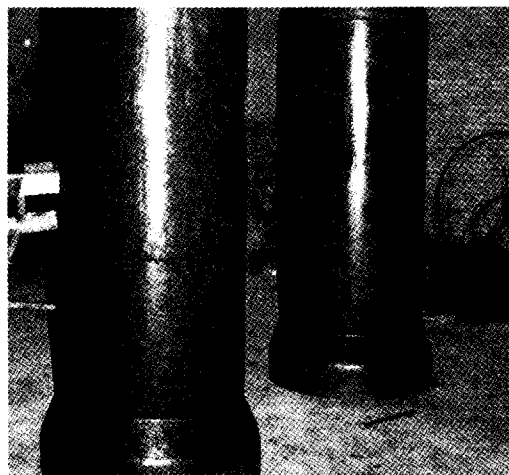
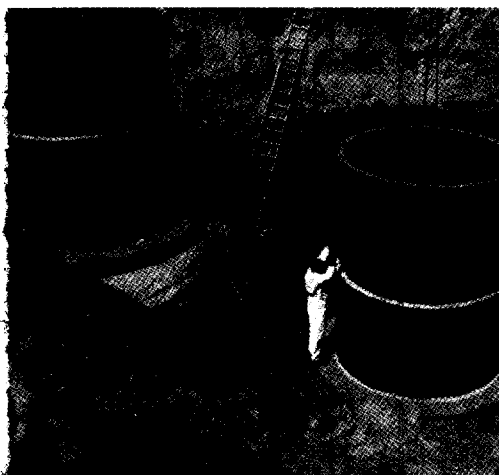
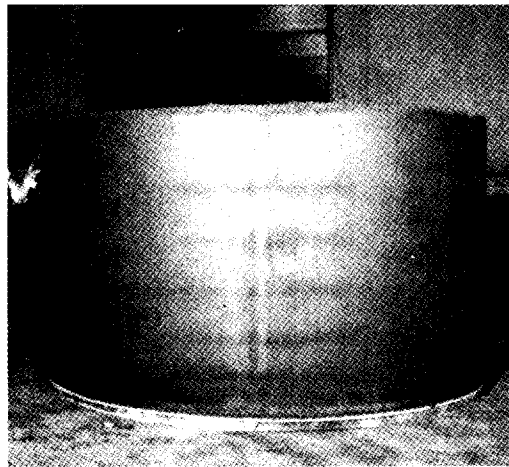


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PREFACE

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

This volume contains the papers which were presented at the ninth symposium, which consisted of two sessions during the ACI fall convention in 1990 in Philadelphia. Arthur M. Dinitz and Jack A. Morrow served as chairmen of the two sessions. At this time, Jack J. Fontana was chairman of Committee 548. I would like to express the gratitude of Committee 548 to these three gentlemen for their efforts in organizing and conducting the sessions. Also the committee wishes to express its appreciation to the authors who made the proceedings possible, as well as to those members who assisted by reviewing the printed papers.

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Chairman
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SP 137-1

Precast Reinforced Concrete Pipe Lined with Polymer Mortar

by M. Kawakami, H. Tokuda,
M. Kagaya, and R. Nasu

Synopsis: Reinforced concrete pipe used as sewer pipe is required to have excellent durability, especially chemical resistance, in addition to high strength and good economy. A precast composite pipe, in which a reinforced concrete pipe is lined with polymer mortar as soon as centrifugal compaction and drainage has been completed, was developed and investigated.

Chemical resistance of the polymer mortar used to acidic water and alkaline water was first studied by the immersion tests and manufacturing process of the composite pipe was introduced secondly. Subsequently, the load tests were carried out and elastic behavior, cracking and breaking strength of the pipes were investigated. Furthermore, a reasonable design method for the composite pipe is discussed and proposed.

Keywords: Chemical attack; compaction; cracking (fracturing); expansive cement concretes; mortars (materials); pipe (tubes); polymer concrete; precast concrete pipe; sewer pipes; strength

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ACI member H. Tokuda is dean of Mining College, Akita University. He received his Dr. Eng. from Hokkaido University in 1973 and has published numerous papers on thermal properties and other properties of concrete.

ACI member M. Kagaya is a research associate, Department of Civil Engineering, Akita University. He received his Dr. Eng. from Hokkaido University in 1988. He engaged in experimental study on vibrating compaction of RCC.

R. Nasu is a managing director and a director of Technical Research Center of Teihyu Corporation, Japan. He received a BS from Kumamoto University in 1956. He has been conducting applied research on the structural behavior and design of reinforced concrete pipes.

INTRODUCTION

The main function of sewer pipe is to smoothly transport sewage and rainfall from one place to another. Sewer pipes are normally buried in the earth, and so require high strength and durability along with chemical corrosion and abrasion resistance. Earthenware pipe, polyvinyl chloride (PVC) pipe and reinforced concrete pipe including jacking pipe satisfy these requirements. Figure 1 shows the fiscal 1988's actual records of sewer pipes in Japan corresponding to the diameters of pipes.

Earthenware pipe and PVC pipe have excellent chemical resistance, but because of their lower strength, particularly their low impact resistance, they are used primarily for diameters less than 300 mm and almost never for sizes greater than 500 mm.

Reinforced concrete pipe, on the other hand, has excellent strength and is ideal for transporting water, but it has some shortcomings when used for sewer pipe. Both concrete and steel are susceptible to chemical attack, especially from acids. The interior surface of sewer pipes is in direct contact with waste water, and concrete pipe will corrode if exposed to an environment (1) that is more acidic than pH 4. The various technologies available for lining the interior surface of reinforced concrete pipe with anti-corrosive materials such as polymer paste, polymer mortar, and polymer-impregnated concrete have been introduced and developed (2).

In this study, a precast composite pipe lined with polymer mortar was tested as follows:

First, chemical resistance of the polymer mortar used to the solutions of hydrochloric acid, sulphuric acid, sodium hydroxide and sodium chloride was clarified by the immersion tests.

Second, centrifugal lining method of polymer mortar was developed

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and introduced as soon as centrifugal compaction and drainage of base core reinforced concrete pipe had been completed. Expansive concrete was used for base reinforced concrete pipe in order to obtain high cracking strength.

Third, cracking and breaking loads were measured by loading tests and those results are discussed and compared with the analysed data.

Finally the optimum design method for the composite pipe considering lining depth of polymer mortar and thickness of concrete composing the base concrete pipe was investigated.

MATERIALS USED FOR REINFORCED CONCRETE PIPE

Plain concrete and expansive concrete were used to make the base reinforced concrete pipe and their mix proportions and physical properties are shown in Table 1 and 2. The expansive additive used was a lime system and the relationship between expansive strain and reinforcing steel ratio are shown in Fig. 2, according to Japanese Industrial Standards(JIS) A 6202.

PROPERTIES OF POLYMER MORTAR AS THE LINING MATERIAL

The hydrophilic polymer used is composed of epoxy polymer as the principal ingredient and modified polyamine as the hardener. A properly graded chamotte, that is ceramics powder of the maximum size 1.5 mm as fine aggregate was used. Weight ratios of polymer paste and chamotte to polymer mortar were 0.5 and 0.5. The physical properties of polymer mortar are summarized in Table 3.

Chemical resistance of polymer paste and mortar to acidic and alkaline water are shown in Fig. 3 to 6. The weight change of polymer paste was not significantly observed and the absolute values of the weight change were less than 1 %. These pastes were especially stable for alkali.

On the other hand, weight changes of polymer mortars immersed in the solutions of sulphuric acid and hydrochloric acid of 10 % concentration were larger than those in city water of pH 6.6. As polymer mortar was composed of polymer paste and fine aggregate as anti-corrosive materials, it seemed weight changes were caused by swelling and corrosion of polymer mortar. The increases of weight changes of polymer mortars immersed in alkaline water were stopped at six months, however those immersed in acidic water were approximately proportional to the immersion period and they did not stop at two years.

Roughness of the interior surface of this composite pipe was measured by surface trace with needle. Figure 7 shows the surface roughness of the proposed composite pipe, centrifugally compacted concrete pipe using ordinary concrete on site.

According to JIS K 7204, abrasion resistance of polymer mortars and concrete by abrasion wheels were tested. Abrasion loss in weight at 1000 revolutions for polymer mortar and ordinary cement concrete are shown in Fig. 8. From this result, weight loss of this

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polymer lining pipe is about one-third smaller than that of concrete pipe and abrasion level of this pipe is adequate to be used for sewer pipe.

MANUFACTURING PROCESS

Flow chart for manufacturing the proposed polymer lined pipe is shown in Fig. 9. Ordinary reinforced concrete pipe using expansive concrete as structural member is first made by centrifugal compaction. As soon as compaction and drainage have been completed, the polymer mortar lining is performed by use of machine or by hand as shown in Fig. 10. Polymer mortar must be hydrophilic with low shrinkage. Therefore special epoxy polymer as the binder and a properly graded chamotte as fine aggregate were used.

When the designed lining depth was attained, the interior surface was finished as shown in Fig. 11, and this lined pipe was cured by far infrared rays and steam.

Composite centrifugal reinforced concrete pipes of length 1200 mm, internal diameter 1000 mm and total thickness 82 mm as shown in Fig. 12 were made and tested. The lined depth of polymer mortar for all pipes was 5 mm. Three kinds of base concrete pipes in order to investigate the effects of expansive concrete were chosen as follows:

Pipe[1]: Reinforced concrete pipe using expansive concrete only,
 Pipe[2]: Reinforced concrete pipe using ordinary concrete cast in outer half layer and expansive concrete cast in the inner half layer,
 Pipe[3]: Reinforced concrete pipe using ordinary concrete cast in outer three quarter layer and expansive concrete cast in a quarter layer.

STRENGTHS OF COMPOSITE PIPE

Cracking Strength

Cracking load P_{cr} of the composite pipe (6) may be calculated by the following equation,

$$P_{cr} = \frac{\sigma_{bt} + \sigma_{cp} + \sigma_d}{\sigma_{qr}} \quad \text{----- 1)}$$

where

σ_{bt} is flexural tensile strength of concrete,
 σ_{cp} is chemical prestress induced by use of expansive concrete,
 σ_d is flexural stress due to self weight of the pipe,
 σ_{qr} is flexural stress due to the unit applied load in the loading test.

Distribution of chemical prestress estimated by theory of elasticity (3,4) and the results are shown in Fig. 13. Compressive stresses were observed in the layer of expansive concrete and tensile stress was generated in the layer of polymer mortar, the ex-

terior layer of ordinary concrete and the reinforcing steel as the result of restraining the strain of expansive concrete. Table 4 shows the chemical prestress, the stress (5) due to self weight of pipe and the stress (6) generated by the unit load (9.8kN/m).

Breaking Strength

The breaking load P_u (6) based on limit analysis for reinforced concrete pipe can be given by the following equation,

$$P_u = 2(\mu_A + \mu_B)/r - 0.66W \quad \text{----- 2)}$$

where

μ_A is ultimate strength of the cross section at the top and bottom position of the pipe,

μ_B is the ultimate strength of the cross section at the lateral diameter of the pipe,

r is the radius of the pipe,

W is the weight of the pipe per unit length.

Cracking Load and Breaking Load

A load test was carried out according to JIS A 5303. Figure 14 and 15 show the relationships between load and strain, and load and deflection of pipe[1]. From these figures, strain and deflection linearly increased with load before cracking and strains observed at cracks were about 250 to 300×10^{-6} .

Bond strength

Onto the interior surface of the pipe, a square steel plate was affixed with epoxy polymer. Along the edge of the plate a split was cut to the base concrete. The specimen was tested by using a portable uni-axial tension testing machine equipped with a center hole jack. The whole broken surface was in the base concrete and the bond strength was 2.4 and 2.2 MPa at the age of 14 days and 18 months, respectively.

Cracking loads and breaking loads tested were obtained as shown in Table 5. Experimental cracking loads were about 20 % higher than analytical ones. As the lining polymer mortar has very high extensibility, cracks were normally observed at the base concrete as shown in Fig. 16.

On the other hand, breaking strengths calculated by Eq. 2) agreed relatively well with experimental measurements. As the reinforcing steel ratio of the composite pipe is designed to be much smaller than the balanced steel ratio at the ultimate strength of the cross section at the bottom and at the lateral diameter of composite pipe, the ultimate strength depends strongly on yielding strength of the tensile reinforcing steel. And so, theoretical breaking strengths of composite pipes agreed well with experimental ones.

DESIGN OF COMPOSITE PIPE

Characteristics of composite pipe has been investigated from the view point of manufacturing, durability and strengths. With regards to design of composite pipe, lining depth of polymer mortar, depth of expansive concrete and reinforcement should be properly determined by considering total cost of pipe. Lining depth depends on workability, centrifugal compaction, chemical resistance, hydraulic properties, bond strength and extensibility of polymer mortar. Several kinds of polymer mortar were tested in the same conditions and it seemed the polymer mortar described above were the best one. It is recommended that the lining depth of polymer mortar corresponding to the diameters of pipes 400mm to 800mm, 900mm to 1350mm, and 1500mm to 3000mm are 4mm, 5mm and 6mm, respectively, because of workability and the maximum size of the fine aggregate used.

On the other hand, the depth of expansive concrete is determined by the cracking strength related to chemical prestress. Figure 17 shows the relationship between cracking strength and depth of expansive concrete. Judging from this result, the depth of expansive concrete is desirable to be about 0.27 of total thickness of pipe. This depth is corresponding with the critical values of both crack modes at boundary of plain concrete and expansive concrete, and at outer surface of lateral diameter. Ultimate strength is mainly a function of reinforcing steel, especially yielding and arrangement of reinforcing steel. These factors are relatively easy to control, and therefore breaking strength of composite pipe can be precisely designed.

Furthermore, in order to perform the quality control of this proposed composite pipe, loading tests depending on the diameters of the pipes to be the same as for ordinary reinforced concrete pipe are carried out as follows:

There will be one of every 200 pieces for 400 to 900 mm, every 150 pieces for 1000 to 1800 mm, every 130 pieces for 2000 to 2400 mm and every 100 pieces for 2600 to 3000 mm, respectively.

The manufacturing process introduced in this paper is applicable for the sandwiched pipe, in which both exterior and interior surfaces of reinforced concrete pipe are lined with polymer mortar required in the environment of strong chemical attack condition such a hot spring zone. The practical composite pipe was used for the sewer pipe in the acidic hot spring zone as shown in Fig. 18.

CONCLUSIONS

A reinforced concrete pipe lined with polymer mortar applied for sewer pipe was tested. This proposed pipe has a number of advantages compared to the conventional concrete pipe. The results obtained by this study are summarized as follows:

1. Excellent chemical resistance to acidic and alkaline water can be expected.

2. High cracking strength due to chemical prestress by use of expansive concrete is obtained.
3. The interior part would be protected by the lined polymer mortar because of high extensibility of polymer mortar if the deformation of the pipe is large and the concrete part is cracked.
4. As the polymer mortar is lined as soon as centrifugal compaction and drainage of base concrete pipe is completed, manufacturing process flows continuously and this system will be adopted to wide range of diameter from 400 to 3000 mm.
5. The excellent abrasion and flatness of the lined polymer is advantageous from the viewpoint of hydraulic engineering.

ACKNOWLEDGMENT

A part of this research was supported by Grant-in-Aid for Developmental Scientific Research No.63850107 from The Ministry of Education, Science and Culture, Japan.

REFERENCE

1. Kawakami, M., Tokuda, H. and Kagaya, M., "Durability of Resin Concrete Exposed to strong Acid Hot Water," Proceeding of Third International Congress on Polymers in Concrete, Koriyama, Japan V.2(1981), pp. 524-537.
2. Kawakami, M., Tokida, H and Nasu R., "Coating Technologies for Sewer Pipe," ACI Concrete International; Design & Construction 11(11)(1989), pp. 86-88.
3. Kawakami, M., et al., "Inherent stress of Multi-Layer Cylinder," Journal of Structural Engineering, 34A(1988), pp. 1-7.(in Japanese).
4. Kawakami, M., Gamski, K., Tokuda, H. and Kagaya, M., "Chemical Prestress and Strengths of Reinforced Concrete Pipes Using Expansive Concrete," Materials and Structures of RILEM, 22(1989), pp. 83-90.
5. Timoshenko, S. P. and Goodier, J. N., "Theory of Elasticity," McGrawHill(1951), pp. 71-75.
6. Kawakami, M., et al., "Strength of Polymer Composite Pipe," Proceedings of Fourth International Congress on Polymers in Concrete, Darmstadt, FRG(1984), pp. 169-174.

TABLE 1 — MIX PROPORTION OF THE USED CONCRETE

Concrete	G _{max} (mm)	Slump (cm)	W/(C+E) (%)	s/a (%)	Unit weight (kg/m ³)					
					W	C	S	G	Ex	Admixture
Plain	20	3±2	35	37	158	450	679	1164	--	3.2
Expansive	20	3±2	31	40	149	430	733	1108	50	3.4

TABLE 2 — PHYSICAL PROPERTIES OF THE PLAIN AND EXPANSIVE CONCRETE

Concrete	Compressive strength (MPa)	Flexural strength (MPa)	Modulus of elasticity (GPa)	Poisson's ratio
Plain	58.1	5.4	34	0.2
Expansive	63.2	7.9	34	0.2

TABLE 3 — PHYSICAL PROPERTIES OF POLYMER MORTAR

Property	results (unit)	Remarks
Compressive strength	36.6 (Mpa)	JIS K 7208
Tensile strength	13.6 (MPa)	JIS K 7113
Flexural strength	20.6 (MPa)	JIS K 7203
Modulus of elasticity	2.74 (GPa)	JIS K 7203
Poisson's ratio	0.31	
Coefficient of thermal expansion	53 x10 ⁻⁶ (/°C)	JIS K 7113
Extensibility	3.42 (%)	JIS K 7113
Setting shrinkage	8.8 x10 ⁻⁴	

TABLE 4 — STRESS DISTRIBUTION OF COMPOSITE PIPE

kind of pipe	Chemical prestress (MPa)			Stress due to self weight (MPa)			Stress due to unit external load (MPa)		
	(S1)	(S2)	(S3)	(S1)	(S2)	(S3)	(S1)	(S2)	(S3)
Pipe[1]	-2.08	-1.78	—	0.28	0.72	—	1.62	0.73	—
Pipe[2]	-2.69	1.73	-2.50 1.85	0.28	0.72	0.03	1.62	0.73	0.006
Pipe[3]	-3.01	0.58	-2.90 0.64	0.28	0.72	0.31	1.62	0.73	0.75

(S1): inner surface at bottom

(S2): outer surface at lateral diameter

(S3): boundary of plain concrete and expansive concrete

(-): compressive stress

(+: tensile stress

TABLE 5 — CRACKING LOAD AND BREAKING LOAD OF COMPOSITE PIPE

kind of pipe	Cracking load (kN/m)		Breaking load (kN/m)	
	Experimental	Theoretical	Experimental	Theoretical
Pipe[1]	63.7	55.8	87.4	91.3
Pipe[2]	58.8	45.5	91.1	91.3
Pipe[3]	60.4	58.0	87.8	91.3

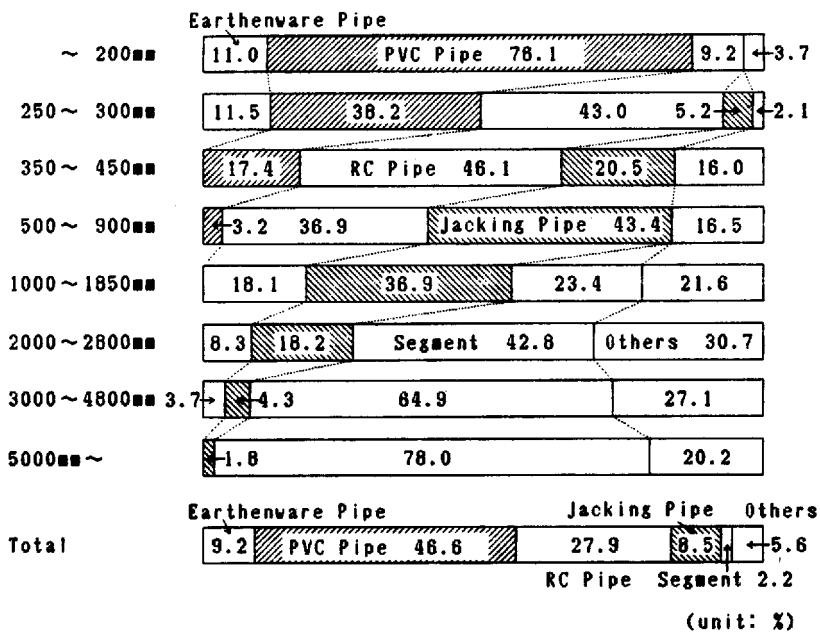


Fig. 1—Share of sewer pipe in Japan

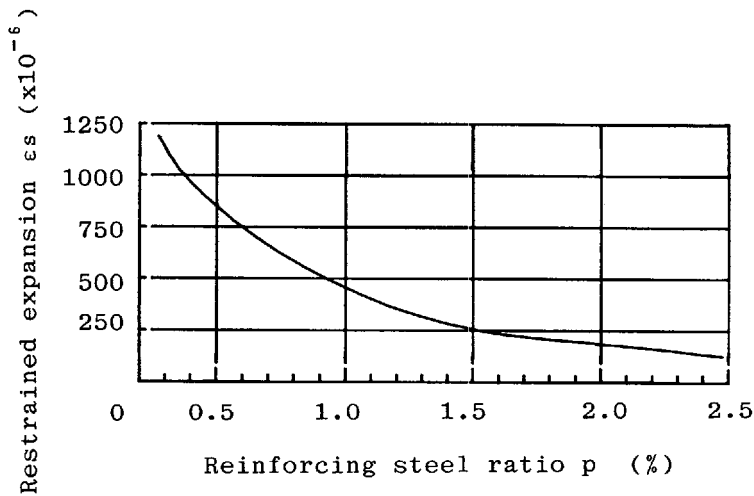
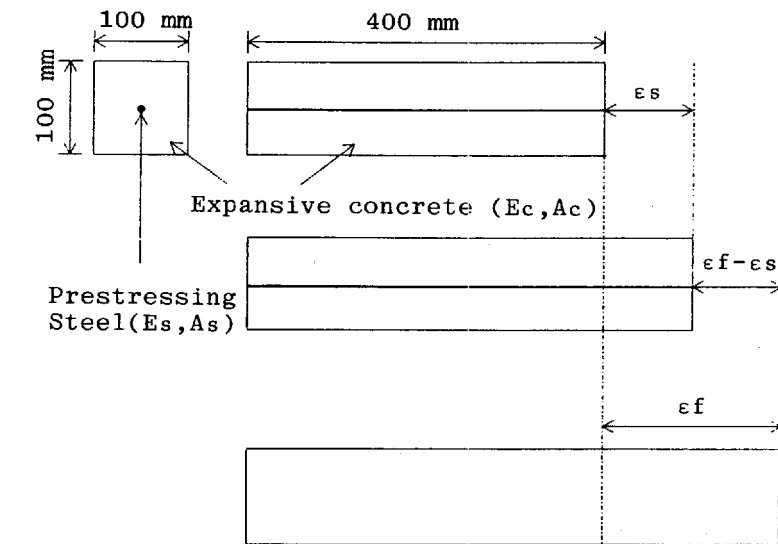


Fig. 2—The prismatic specimen to measure the restrained expansive strain ϵ_s and reinforcing steel ratio p

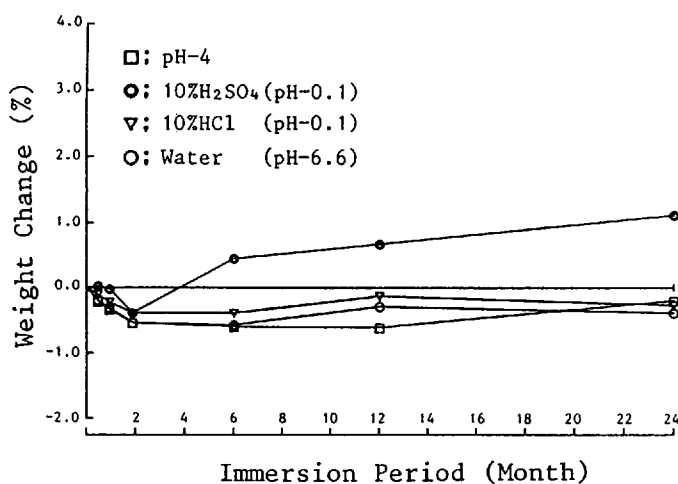


Fig. 3—Weight change of polymer paste immersed in acidic water

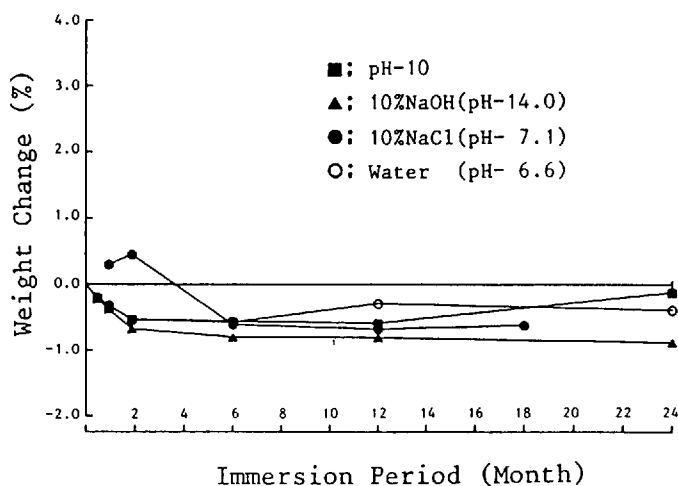


Fig. 4—Weight change of polymer paste immersed in alkaline water

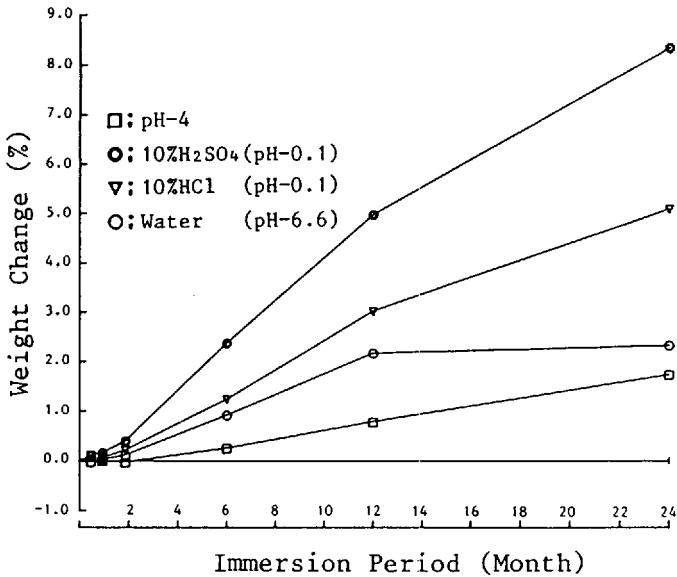


Fig. 5—Weight change of polymer mortar immersed in acidic water

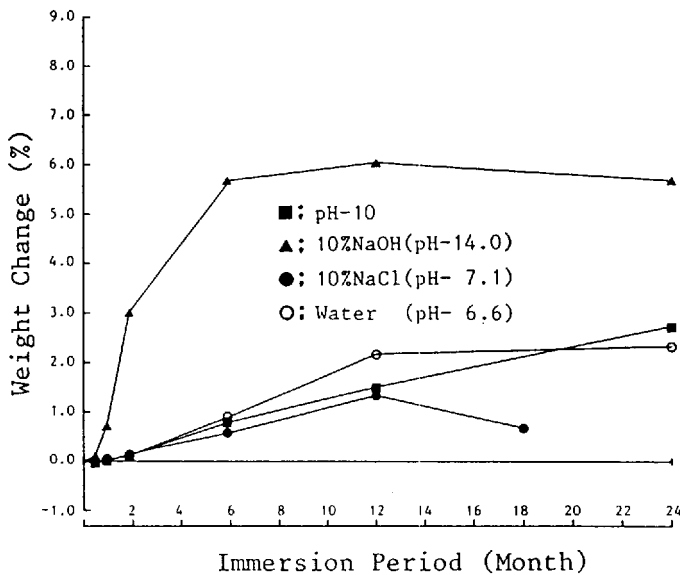


Fig. 6—Weight change of polymer mortar immersed in alkaline water

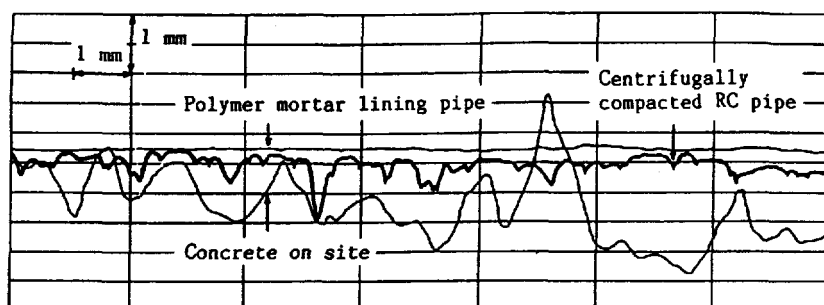


Fig. 7—Interior surface roughness of composite pipe

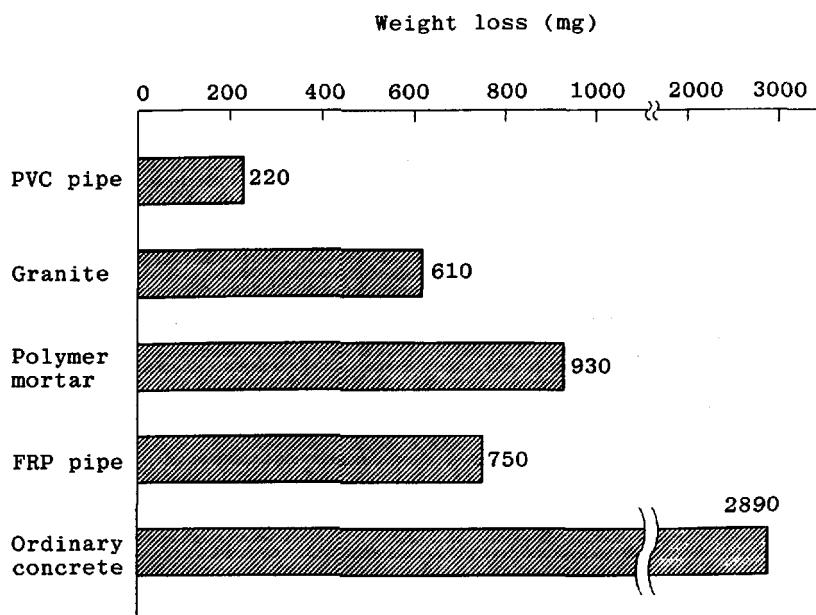


Fig. 8—Abrasion test results

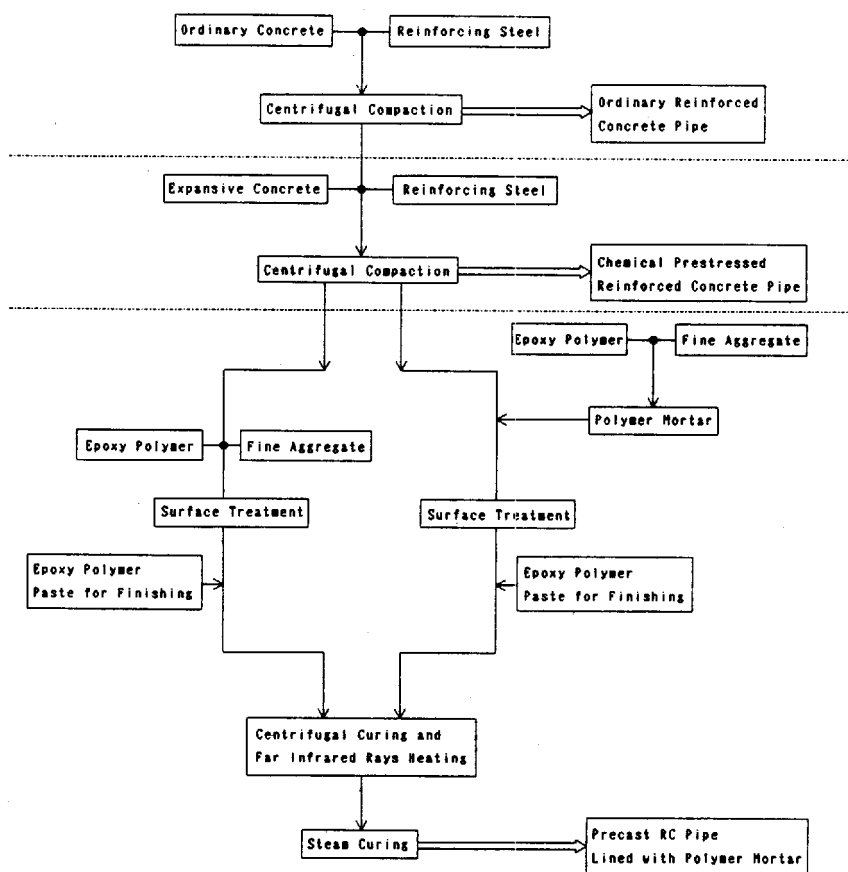


Fig. 9—Manufacturing process for composite pipe

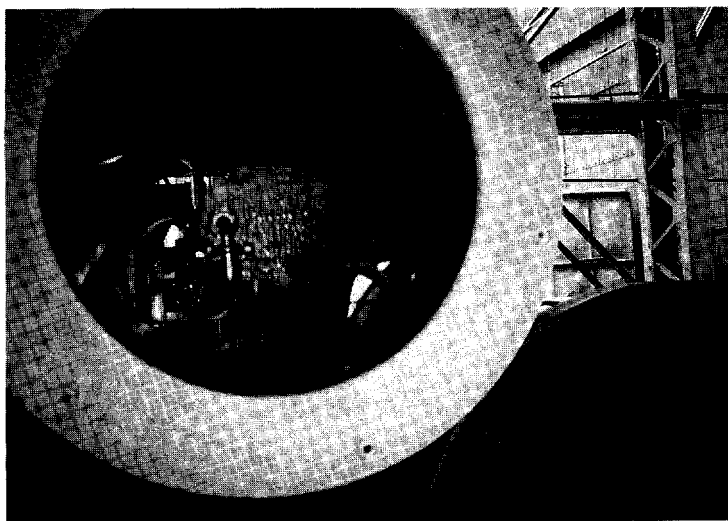


Fig. 10—Placing polymer mortar



Fig. 11—Surface finishing treatment

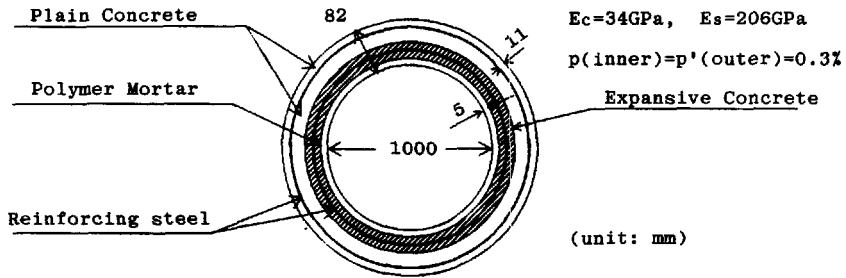


Fig. 12—Cross section of a composite pipe with diameter 1000 mm

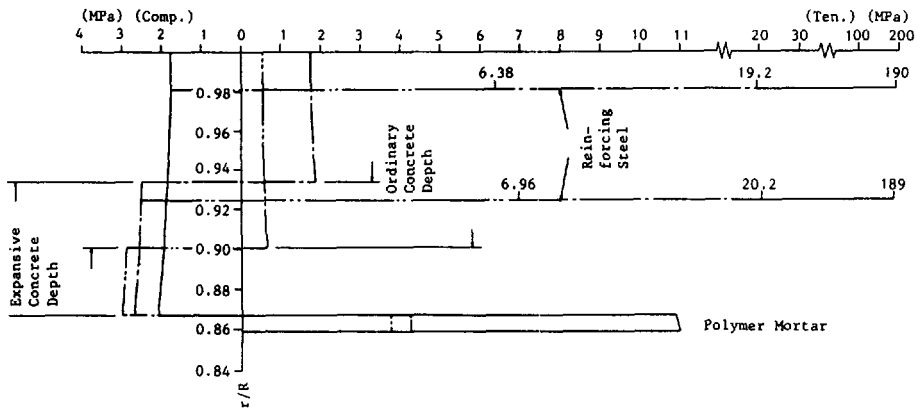


Fig. 13—Distribution of chemical prestress

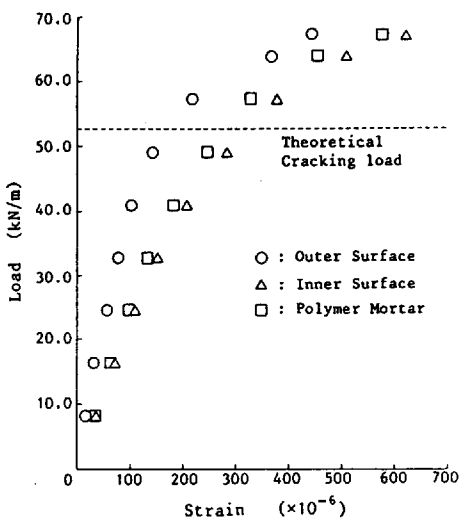


Fig. 14—Relation between load and strain

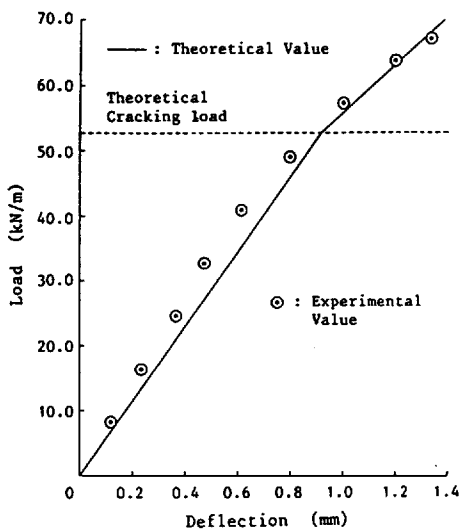


Fig. 15—Load-deflection curve

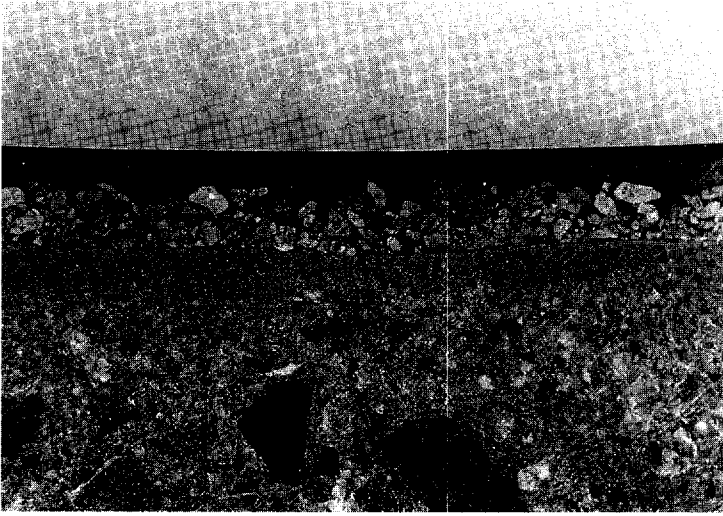


Fig. 16—Typical crack of the concrete part

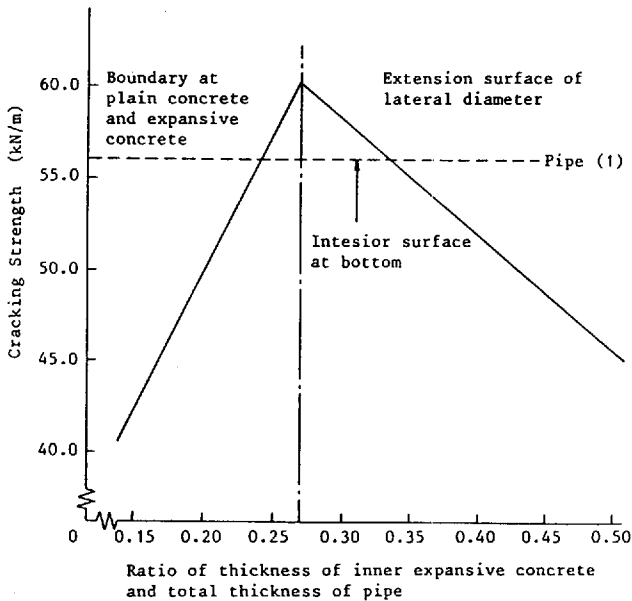


Fig. 17—Cracking strength and thickness of expansive concrete

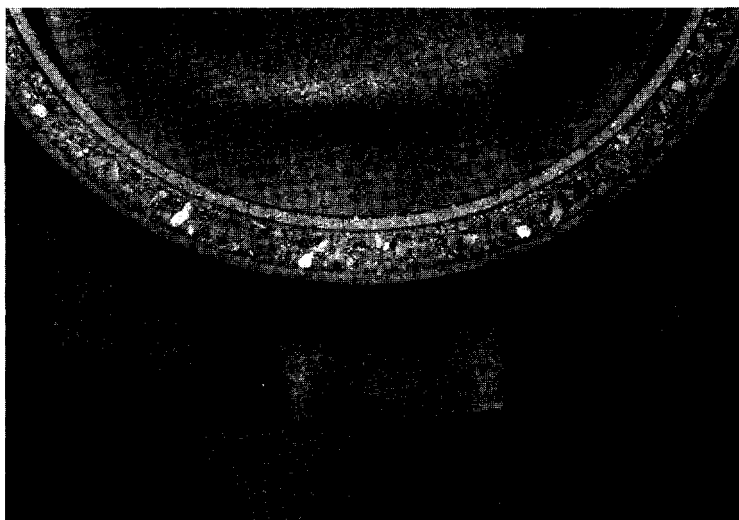


Fig. 18—Polymer mortar lining pipe at both exterior and interior part

SP 137-2

Steel-Fiber Reinforced Polymer Concrete

by J.J. Fontana

SYNOPSIS:

Steel-fiber reinforced polymer concretes were developed for utilization in large precast vaults. The data obtained indicate that polyester resin concretes reinforced with steel fibers will provide a ductile composite allowing structural engineers to design vaults capable of withstanding HS-20 loadings. Epoxy resin concretes exhibit higher flexural strengths, however they do not carry loads beyond the yield point of the composite and are considered to brittle for this application. Several vaults were cast and have been placed in service.

Keywords: Compressive strength; coupling agents; epoxy resins; fiber reinforced concrete; fibers; flexural strength; loads (forces); polyester resins; polymer concrete

JACK J. FONTANA has conducted research and development in the field of concrete polymer materials for almost 29 years at Brookhaven National Laboratory. He is presently involved in the manufacture of latex modified concrete and epoxy concrete materials for the repair and overlay of portland cement concrete structures.

INTRODUCTION

Polymer concrete has primarily been used to repair reinforced portland cement concrete and to overlay bridge decks, parking garage decks, and industrial floors. Broader utilization of polymer concrete in load-bearing elements is presently taking place in machine tool bases, gear pump base plates, and utility vaults.

For many applications polymer concrete can be designed to utilize its high compressive strength; however many applications require a material with high bending strengths as well. The addition of steel-fiber reinforcement greatly improves the flexural strength and ductility of polymer concrete. This is considered necessary for many applications.

A ductile polymer concrete can be utilized for engineering applications that were previously not considered feasible. One of these applications is the manufacture of large precast polymer concrete utility vaults. Polymer concrete utility vaults will provide a longer, maintenance free service life because their durability and physical properties are superior to those of portland cement concrete. The higher strengths will allow design engineers to reduce the wall, floor, and ceiling thicknesses of the vaults thus making them lighter for easier transportation and installation. In addition, the cost of polymer concrete vaults can now be competitive with portland cement concrete.

MATERIAL EVALUATIONS

The effect of silane coupling agents on the compressive strength of polyester resin concretes has been previously reported.^{1,2,3} Experience has indicated that the use of silane coupling agents promotes the adhesion between the resin and silica aggregates, thus increasing the strength of the composite.

In Table 1, four mix designs have been tabulated. Two of these were made with silane coupling agents and two

without. The polyester resin concretes with the silane coupling agent had compressive strengths that were 16 to 24 percent higher than equivalent samples without silane.

The flexural strength of several different resin concretes was measured on 2 inch by 2 inch by 12 inch beams per ASTM-C78. The data in Table 2 indicate that the type of resin used has a pronounced effect on the measured strengths. The epoxy resin concrete was 8 to 19 percent higher than the orthophthalic and dicyclopentadiene polyester resin concretes, respectively. Different resins will yield different results in almost every case. The polyesters in this case were low elongation resins with a modulus of 150,000 to 250,000 psi. The epoxy resin had a moderate elongation with a modulus of 90,000 to 110,000 psi.

The orthophthalic polyester was used to study the effect of the aspect ratio of steel fibers on the flexure strength of resin concrete. Three different aspect ratios were investigated: 10, 20, and 60 (Table 3). The first two are ribbed fibers and the latter a hooked-end fiber. The measured flexural strengths do not indicate any significant changes when the aspect ratio was changed. However, there was a significant difference in the flexural strength when the quantity of fibers was increased from 4.8 to 9.1 weight percent. Although the hooked-end fiber content by weight was only 5.4 percent the actual number of fibers was 1.8 times higher than the ribbed fibers. In all cases the steel fibers pulled out of the resin concrete matrix during the bending test. This is the reason why these samples exhibit such a ductile behavior.

The aggregate mix design was optimized and the coarse aggregate was changed from a silica river gravel to a crushed granite. These changes produced composites with much higher flexural strengths (Table 4). The epoxy resin concretes have a much higher flexural strength than those of polyester resin concretes but the ductility of the epoxy composites is much lower than that of the polyester concretes. The area under the load-deflection curve (Figure 1) beyond the yield point is at least ten times the area under the yield point for the polyester resin concrete. This indicates that the composite is very ductile. The load deflection curve of the epoxy resin concrete indicates that these composites cannot maintain a load beyond the yield point and, therefore, these composites are considered brittle.

In the polyester resin concrete the load is transferred to the steel fibers and at the yield point they begin to carry most of the load. The steel fibers pull out of the

resin concrete matrix as they are loaded until the ultimate stress is reached. At this point the composite can no longer carry any load. In the epoxy resin concrete the load is also transferred to the steel fibers; however, the bond of the epoxy resin to the steel fibers is so good that they cannot pull out of the matrix and break just beyond the yield point of the composite. This brittle behavior can result in a catastrophic failure and is unacceptable for some engineering applications.

Most of the design processes for the large resin concrete vaults were based upon simplifying assumptions that are acceptable when applied to ductile materials. Such discontinuities in box geometries as corners, lifting lugs, and pipe penetrations result in stress concentrations that can easily be 5 to 10 times the average stress values used in the design. When a ductile material is so stressed, it will yield locally and redistribute the stress so that at ultimate failure, the stress distribution approaches that assumed in design. A brittle material however, cannot redistribute the stress and will fail when the peak stress around the stress concentrations reaches the allowable limit.

DESIGN OF PRECAST POLYMER CONCRETE VAULTS

The procedure used for the design consider the vaults as a series of plates: cover, sidewalls and foundation slab. Each of the components was analyzed using classical plate theory with the loading resulting from soil pressures, highway loading and dead weight. The highway loading used in this case was AASHTO HS-20. This was considered to be the most severe of the options available.

Once the design was completed, a quarter-scale model of a vault was built. The center-to-center dimensions were approximately 2 ft. by 1.5 ft. by 1.5 ft with a wall thickness of 1 inch. The first tests of the box were conducted by evacuating the box thereby subjecting it to a uniform pressure. A peak pressure of 1643 psf was achieved. The box was subjected to several complete load-unload cycles without suffering any damage. The typical design pressure for this box was about 1000 psf so that the test subjected the box to pressures 60% higher than design.

The test indicated that there was no observable cracking since any cracking would have made it impossible to achieve the vacuum especially beyond the first load cycle. The measured strains and deformations in the box indicated linear behavior with complete recovery. The

deflection at the center of the larger plate increased from 0.0015 inch at a vacuum of 600mm Hg to 0.0053 inch at a vacuum of 200mm Hg. The measured deflection on the unloading side was 0.0019 inch at 600 mm Hg.

The box was then modified by inserting a 6-inch diameter pipe through the center of the large wall to represent a pipe penetration in actual practice. The pipe was bonded to the wall with an epoxy mortar. On retesting, the center deflection was 0.0054 at 200 mm Hg which compared favorably with the first test.

The box was loaded to failure with a concentrated load applied through a 9-inch diameter pipe centered around the penetration. Strain gauges were placed every 3 inches in the X and Y axis from the center of the box. The measured load-deflection curve is shown in Figure 2. The yield load was 7500 pounds at a deflection of 0.064 inch. The ultimate load was 12,500 pounds. Catastrophic failure was not observed, but the load dropped off slowly as the deflection increased. For example, the load corresponding to a deflection of 0.77 inch was 5000 pounds. Since concentrated loads scale as the square of the geometric scale factor, the experimental data seemed to agree with the design loads within the experimental parameters.

CONCLUSIONS

All of the experimental data indicate that steel-fiber reinforced polymer concrete vaults can be cast with thinner walls than portland cement concrete vaults. Several of the polymer concrete vaults were cast and placed in service. After approximately 18 months of service the utilities report that the polymer concrete vaults are performing as good as was expected.

REFERENCES

1. L.E. Kukacka et al, Introductory Course on Concrete Polymer Materials, Brookhaven National Laboratory, BNL 19525, December 1974.
2. Yoshihiko Ohama, Effect of Water Content of Aggregates Used and Coupling Agents on Durability of Polyester Resin Concrete, Journal of the College of Engineering of Nihon University, Series A, Vol. 19, March 1978.
3. Jack J. Fontana, The Use of Silane Coupling Agents in Polymer Concrete Composites, Proceedings of conference "Progress in Coupling Agent Technology and Market Development", Cleveland, Ohio, May 1990.
4. T. Broniewski et al, Long life strength Polymer Concrete, First International Congress on Polymer Concretes, Session E, Paper 5, May 1975.

TABLE 1 — EFFECT OF SILANE COUPLING AGENT ON
THE COMPRESSIVE STRENGTH OF POLYMER CONCRETE

RESIN CONTENT wt. %	SILANE CONTENT wt % of resin	COMPRESSIVE STRENGTH psi
10.3	0	7795
8.4	0	8163
9.4	1.0	10101
9.4	1.0	9425

TABLE 2 — FLEXURAL STRENGTH OF SEVERAL RESIN
CONCRETE SYSTEMS WITHOUT STEEL FIBERS

TYPE OF RESIN	FLEXURE STRENGTH psi	TANGENT MODULUS 10 ⁶ psi
DICYCLOPENTADIENE POLYESTER	2704	3.25
ORTHOPHTHALIC POLYESTER	2988	2.27
EPOXY	3216	2.77

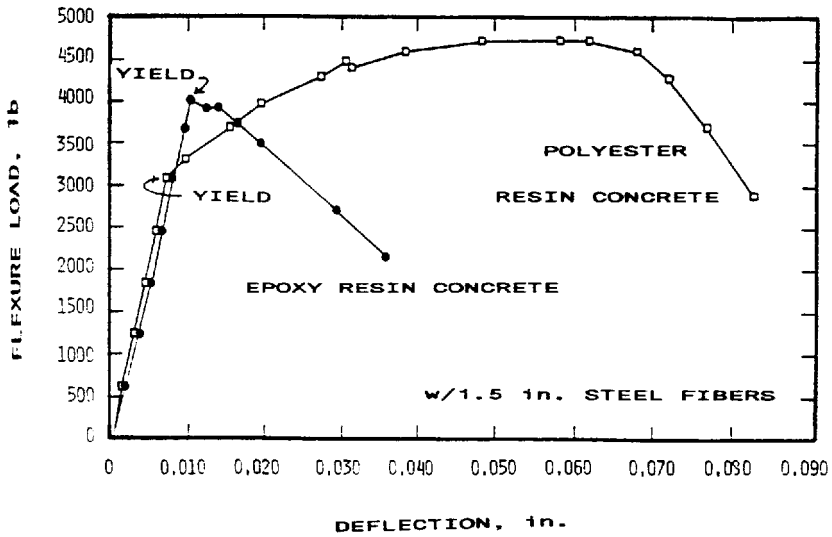


Fig. 1—Load-deflection curves of steel-fiber reinforced polymer concrete

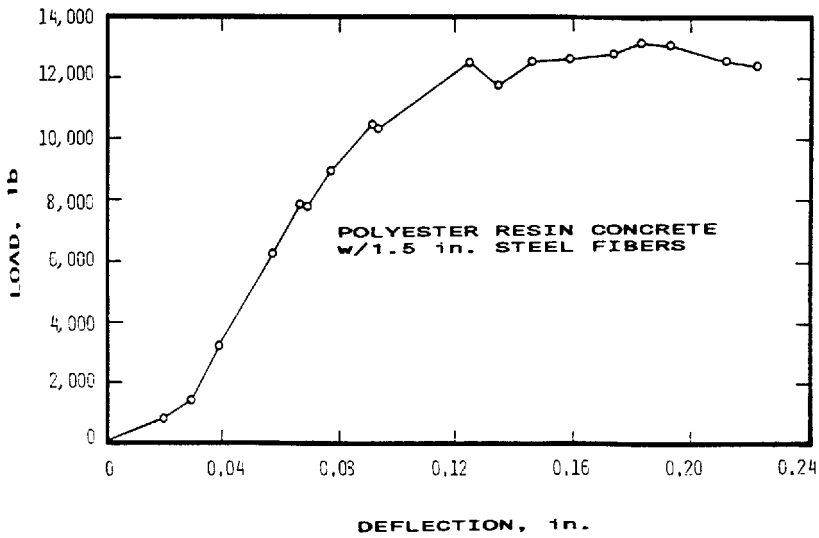


Fig. 2—Load-deflection curve of quarter scale model utility vault

SP 137-3

Thin Polymer Wearing Surfaces for Preventive Maintenance of Bridge Decks

by P.D. Carter

Synopsis: This paper will discuss the lessons learned from 66 Alberta bridge sites where thin polymer wearing surfaces have been placed by contractors on existing concrete bridge decks. The repair is intended to be a retrofit surface membrane designed as preventive maintenance for use on existing decks where deterioration has not progressed too far. It is intended to reduce the rate of deterioration caused by the penetration of chlorides and water so as to extend the service life of the bridge deck. The climatic exposure conditions of the decks vary from deck temperatures of 45°C to -45°C and average freeze thaw cycles from 69 to 135 per year. Case histories will be presented from the 87 000 m² of installed polymer wearing surface systems. Permeability, skid resistance, and CSE test data will be reported as a measure of the success of the systems in stabilizing rebar corrosion in salt contaminated existing decks. Observations will be made on the cause and prevention of typical localized failures and their frequency of occurrence at typical sites. Comments on life expectancy and cost will be presented, based on actual performance in a severe climate.

Keywords: bonding; bridge decks; corrosion; cracking (fracturing); deicers; deterioration; maintenance; permeability; plastics, polymers and resins; rebars; repairs; service life; skid resistance

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INTRODUCTION

Since 1985 Alberta Transportation and Utilities (AT&U) has waterproofed 66 bridge decks with thin polymer wearing surfaces. These systems are similar in concept to the membranes being installed on the decks of parking garages. This paper reports on the performance of these systems and the lessons learned from their installation.

BACKGROUND

Many previously installed Iowa method concrete overlays that had been used for deck rehabilitation were heavily cracked. These surface cracks appeared to be mostly caused by long term drying shrinkage, but they propagate when subjected to vehicle loads in cold conditions so that each year more cracks were observed. Chloride ions were expected to migrate easily through the cracks. Copper sulphate electrode (CSE) and chloride content testing were done on many of these repaired decks with the conclusion that rebar corrosion was continuing to develop and would likely reduce the service life of the repairs, thereby requiring a second major rehabilitation within the expected 50 year lives of the structures. While investigating possible methods of waterproofing or extending the service lives of these previous repairs, it was noted that some thin epoxy wearing surfaces had been installed by government bridge crews in the 1960's. These sites were inspected and tested by CSE, chloride content, bond strength, and electrical resistivity, and favorable results were found wherever the epoxy had survived the exposure to traffic and snow plows. At the Bow River Bridge on Highway 1X a coal tar epoxy had been used in 1963 to waterproof a lightweight aggregate concrete deck designed to reduce dead load on an older steel through truss span. Seventy percent of the epoxy system was intact after twenty years, and many of the failed areas appeared to have resulted from thermal incompatibility in local areas where too much depth of epoxy had been applied. Many of these local failures occurred within two years of application and stabilized afterward. At a second site, the Battle River Bridge on Highway 21, a thin epoxy-silica wearing surface was 95% intact after twenty years service. This had been later covered by a thin asphaltic chip coat, which probably extended its life by protecting the epoxy from ultraviolet radiation and surface abrasion.

In 1984 a test bridge was selected and divided into eight equal sections of 139.4 m² each for the purpose of field testing some of the membrane wearing surface systems available at that time. Proposals were considered from

contractors known to have expertise in this type of work. Several parking garage membrane systems were included in the tests. These softer, more flexible materials performed poorly under the bridge exposure conditions where higher loads and speeds coupled with snowplowing and regular sanding of the highway during winter conditions resulted in rapid abrasive wear. Another parking garage system that rapidly failed was a coal tar epoxy seeded with an extremely hard brittle slag aggregate. With a MOH hardness of nine this aggregate was too brittle and would often disintegrate under the impact of tires, leaving tiny holes in the membrane surface. After six years exposure to 4000 vehicles per day, most of this system has worn away, exposing the concrete deck. Two of the other test sections involved silica sands embedded in different epoxy resins. Both of these systems are now highly polished along the wheel lines, apparently due to poor wear resistance of the silica. One of these systems also experienced some debonding that was attributed to the well graded fine aggregate filling voids and creating a brittle epoxy-aggregate composite material that performed differently than the concrete deck when subjected to live load and thermal stresses. The most durable of the eight test sections involved a flexibilized epoxy resin and a poorly graded basalt aggregate with a relatively high aluminum oxide content. This section also had the best bond strength and electrical resistivity values.

PURPOSE OF POLYMER WEARING SURFACES

The intended purpose of the systems in Alberta was to economically slow down the rate of deterioration of existing reinforced concrete bridge decks containing relatively high chloride ion contents while also sealing surface cracks created by long term drying shrinkage and live load stresses. Another feature of the repair was to not reduce the load carrying capacity of the bridge by increasing its dead load. Over 90% of AT&U's bridges have black steel in the deck's top rebar mat. 70% of the bridge system was originally designed for HS-20 loading and is more susceptible to cracking when subjected to the higher present day legal loads. AT&U had rehabilitated over 100 bridge decks with Iowa method high density concrete overlays since 1977, and many of these were heavily cracked and relatively permeable to water and chloride ions. For economic reasons the repair was intended to be of a preventative maintenance nature where a minimum of concrete removal would be required. CSE tests were used to predict this optimum time for repair of each deck. Other bridge decks were selected for polymer overlay due to their poor air void systems and high susceptibility to scaling and freeze thaw deterioration. Another factor in selecting suitable repair sites was design strength and live load carrying capacity. Polymer wearing surfaces add very little dead load to a structure. Improvement in skid resistance of the repaired deck was another minor factor involved in the site selection and prioritization process.

DESCRIPTION OF THIN POLYMER SYSTEMS

The thin wearing surfaces done since 1985, have usually consisted of two seeded layers plus a tie coat resulting in an average thickness of 6 mm. A seeded layer is a layer of liquid seeded with aggregate. It is applied to a uniform thickness by squeegee and roller so as to seal the deck surface. The

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aggregate must be quickly introduced to this layer before the resin begins to gel and increase in viscosity. The aggregate sinks into and displaces the resin. Regardless of the amount of aggregate used, the thickness of the layer is governed by the amount and smoothness of the liquid resin that is applied. Excess aggregate is removed after the resin has hardened. The tie coat consists of a thin layer of pure resin used to seal pinholes and voids in the composite layer.

In 1988, six bridges in low traffic areas were repaired with a cheaper Class C system that had only one seeded layer. This system had a non-seeded primer layer. Subsequent permeability tests indicated better waterproofing of this system than the Class B system with two seeded layers and no primer.

In 1989 the Class B design was changed with the tie coat being placed first, as a primer layer, followed by the two seeded layers. Three of the 66 bridges were repaired with a different process consisting of a screeded layer of premixed polymer mortar. AT&U is presently evaluating this type of repair for possible standardized use. The main advantage of the screeded systems is the speed with which they can be placed on large bridges, resulting in labour cost savings and reduced down time, time lost to public use. A possible disadvantage is reduced bond strength due to entrapped air voids at the bond line on rough deck surfaces.

FIVE YEAR WARRANTY

Experience has shown that the premature failure of a thin polymer wearing surface usually involves either the material or the workmanship. System design and site exposure conditions are less likely to be the factors responsible. Such failures might occur after the usual two year performance and maintenance bond period associated with contract work. Requiring a longer performance and maintenance bond period might have reduced competition by eliminating smaller contractors that had already demonstrated the expertise to correctly install the systems. Another potential problem was envisaged where a contractor would not cooperate with the resin manufacturer, and neither party would take responsibility for the resulting failures.

On government work where contracts are awarded to the low bidder, a resin manufacturer might reduce his price by not supplying the necessary technical advice on-site to the contractor. A system was needed to prevent the above problems and to encourage cooperation between the material supplier and the contractor. For this reason a five year warranty signed by both parties was required. Bids would not be considered valid unless accompanied by a jointly signed warranty agreement. Bankruptcy of either party would leave the other party wholly responsible. The warranty covers failure of the wearing surface exposed to normal traffic use. It does not cover failures in the deck below the membrane. Warranty repairs are required at the end of the five year period that commenced when the work was approved by AT&U, except that large failures, i.e. more than 5% of the deck, must be repaired within 60 good weather days of notification, regardless of how soon the accumulated 5% has developed. This has only occurred twice.

Regardless of the warranty, the contractor's work is subject to the approval of the owner's inspectors. The contractor is only compensated for work that is approved and accepted.

COSTS

The unit cost of the polymer wearing surfaces has varied from \$55 to \$75 per square metre depending mostly on the design, the size of the job, and the location. This cost includes the five year warranty. These prices are about half the cost of various types of concrete overlays or membrane and asphalt repairs used in Alberta. Based on present experience the concrete overlays may require less maintenance in the first ten years, but will eventually have rebar corrosion problems. The membrane and asphalt systems will require some maintenance and sealing of asphalt within ten years. The polymer wearing surfaces will likely require an additional average \$5 per square metre of maintenance in the first ten years. Eventually they can be covered by a thin asphaltic chip coat at a cost of about \$2.50 per square metre. The replacement cost of the typical concrete bridge deck is about \$350 per square metre for a service life of approximately 35 years. The wearing surface cost is only about 20% of a complete replacement cost, but should extend the service life of the deck by about 70% of the complete replacement service life.

CRACK REPAIRS

The 1985 to 1987 contracts involved crack repairs for flexural cracks, i.e. those identified as opening due to live load stresses. These were distinguished from shrinkage cracks by:

1. straightness; shrinkage cracks tend to be more curved, randomly oriented, and changing in direction;
2. location in relation to negative movement areas where the deck surface opens under live load; and
3. presence of white stains on the underside of the deck which indicates that the cracks reflect through the full deck thickness.

The flexural cracks were repaired by routing, debonding and filling with a so-called flexible epoxy, which probably performed as designed until exposed to very low temperatures. In 1988 band-aid repairs were introduced. These involved debonding the wearing surface over the crack, then strengthening the wearing surface with tensile reinforcement centered over the crack.

Prior to repair, many of the 1986 and 1987 repair sites were heavily cracked. Battle River on Highway 14 had about 600 lineal metres of flexural cracks and probably 6000 lineal metres of shrinkage cracks. After four years service, about 100 lineal metres of cracks have reflected through the polymer wearing surface. The Peace River bridge repaired in 1986 had 3000 lineal metres of crack repairs and about 70% of these have reflected through the repaired wearing surface. The extreme cold, as well as the flexibility and live load deflection of this deck, make crack repairs appear futile.

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AT&U is presently considering the possibility of not repairing cracks when the wearing surface is installed. Future repairs could be done to the cracks that eventually reflect through the waterproofing system. The work would likely involve routing the cracks and caulking the routed groove with a urethane material recessed about 3 mm below the surface level. A narrow routed width, say 6 mm, would reduce the impact on the edges of the repair and lessen the possibility of traffic damage to the wearing surface.

OBSERVATIONS - LESSONS LEARNED FROM FAILURES

The comments expressed in this section are based on the experience of the systems installed in Alberta and may not always agree with the experiences in other locations.

1. When failures have occurred, they have usually been the result of human error or workmanship rather than material. If the in-place material properties are defective, the usual cause is improper mixing or measuring of the resin components. These errors may be magnified if the resin components are not of similar volumes; that is, a measuring or mixing error will be worse if the A and B components are blended in a ratio of 4 to 1 than if they are 1 to 1. The material manufacturer could reduce these problems by using different coloured components so that such problems become visibly obvious to the workers and inspectors. Small batches of resin, such as for mortar patches, have a higher than normal risk of human errors in measuring or proportioning. Some misbatched resins become darker in colour as they age and therefore become obvious.
2. The moisture content of the deck is a big factor in achieving proper bond strength to the deck. The deck will not have a uniform moisture content. The gutter areas will dry slower, as will other low areas that have absorbed extra moisture due to ponding and poor drainage. Moisture in the deck may react with the resin to prevent bond strength development.
3. When deck patches are required in conjunction with the wearing surface repair, they should be wet cured to reduce shrinkage and debonding, then allowed to surface dry in order to avoid debonding of the wearing surface. Patching should be done prior to surface preparation. In Alberta the maximum allowable depth of polymer mortar patches is 15 mm. Deeper patches develop cracks around their perimeter at low temperatures. Drying times for cementitious patches are mostly dependent on temperature, humidity and patch thickness.
4. The more the surface is prepared and roughened, the greater the mechanical bond of the wearing surface will be, and this will relate to the time the system will last in service. Extra surface preparation is an economical way to extend service life. Shotblasting provides a more uniformly prepared surface than sandblasting. Water blasting increases the potential of weak bond strength and isn't allowed.

5. In cold climates thicker wearing surfaces will fail quicker than thinner ones due to the different thermal coefficients of expansion of concrete and wearing surface. In these failures the wearing surface may come off in sheets with a thin layer of concrete attached to the underside. The shear strength of the concrete may be weaker than the bond strength of the resin.
6. Thin wearing surfaces cannot be expected to smoothen a macroscopically rough deck surface. Deck patching must be done carefully to avoid leaving or creating bumps. Bumps in the end product will shorten the life of the wearing surface, especially if snow plows are in use.
7. Aggregate type and gradation is important for wear resistance and for flexibility, toughness, and crack bridging ability of the wearing surface, especially when high modulus of elasticity resins are involved.
8. The technique used in seeding aggregate in the broom and seed method is also very important in preventing surface ripples, wicking and porosity, and resin rich areas that are not thermally compatible with concrete. Proper techniques allow the individual aggregate particles to spread out and fall downward into the liquid resin, while dust and fines are carried away in the air. Especially in hot weather, the aggregate should not be introduced too quickly, but should be slowly and evenly built up on the surface until no wet spots are visible.
9. Neither the static mixer nor the paddle mixer are foolproof. Since the viscosity of the resin components is affected by temperature, both hot or cold conditions may affect the mixing process. Constant vigilance is required and routine calibration checks must be done. Improperly mixed resin will defeat the best efforts of an otherwise correct application.

CASE HISTORY - A DISASTER

The Crowsnest River Bridge on Highway 3 is an example of many things that can go wrong in a contract polymer wearing surface repair. An invitational contract was tendered in 1985, and only specialized contractors were invited. The contractors had convinced AT&U that they had the expertise to do the work, while AT&U was in the early learning stages, and much of the 1985 specification referred to the 'manufacturers instructions'. The contractor provided a five year warranty, but it quickly became apparent that there were problems. The site inspector rejected the surface preparation twice, and the final surface preparation was still a compromise. The contractor did not notify the manufacturer until the job was well underway, with the result that there was no input from the manufacturer and the contractor's workmanship was poor. Among other things, the weather was too warm, and this combined with slow and improper seeding techniques, resulted in glassy resin-rich areas and porous areas. Due to improper levelling of the liquid resin, the final surface varied in thickness, sometimes from 4 to 10 mm on one 75 mm diameter core. The 200 litre resin barrels had been stored unprotected in the sun and rain, so that water had contaminated the material. It also appeared that air bubbles from improper use of paddle mixers were common. After three years a significant amount,