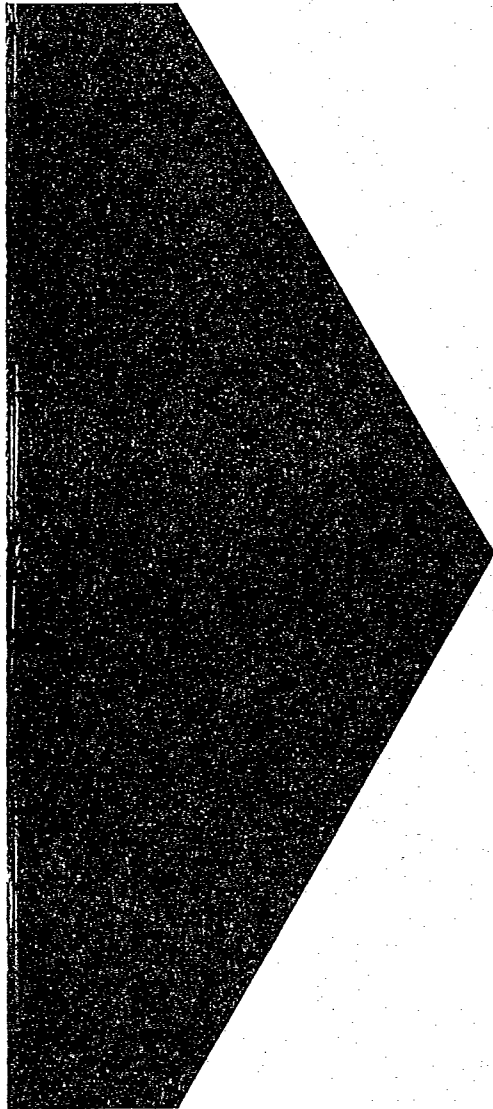


Polymers in Concrete: Advances and Applications



Peter Mendis
Charles McClaskey
Editors

SP-116



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Polymers in Concrete: Advances and Applications

Peter Mendis
Charles McClaskey
Editors

American Concrete Institute, Detroit

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PREFACE

ACI Committee 548 has sponsored six symposia since 1972 to help fulfill its mission to gather and evaluate information on the effects of polymers in concrete. The proceedings of the first five symposia have all been published by ACI and have received world-wide acclaim.

This volume contains papers which were presented at two sessions during the 1988 ACI Annual Convention in Orlando, Florida. The papers combine the development of new concrete polymer materials and the application technology of concrete polymer materials. Mr. Peter Mendis and Mr. Charles McClaskey served as co-chairmen of the symposium, and I would like to express the gratitude of Committee 548 for their efforts in organizing and conducting the sessions as well as compiling the proceedings. The committee wishes to extend its sincerest appreciation to the authors who made the proceedings possible, as well as those members who assisted the chairman by reviewing the printed papers.

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Chairman, ACI Committee 548

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SP 116-1

Application of Styrene-Butadiene Latex Modified Portland Cement Concrete Overlays in Parking Structure Repair and Rehabilitation

by B.D. Irvin

Synopsis: The Styrene-Butadiene latex modified concrete system is an effective technique for repair and rehabilitation of parking garage decks. The system appears to retard corrosion of the reinforcing steel, and has many benefits including ease of placement and workability, superior flexural and bonding strength and deicer scaling resistance. Styrene-Butadiene latex modified concrete can be applied in parking structures during the winter months with few problems.

Keywords: concrete slabs; latex (plastic); parking structures; reinforced concrete; renovating; repairs; resurfacing; styrene-butadiene resins

2 Irvin

Brian Irvin is a Market Development Specialist with BASF Corporation in Chattanooga, Tennessee. He is a member of the American Concrete Institute, subcommittee 548A. He has twenty years experience in all phases of the concrete industry.

INTRODUCTION

Deterioration of parking structures has become a costly concern. The typical method of concrete construction does little to resist the primary cause of deterioration. Consequently, maintenance and repair costs are substantial.

Understanding the reason for this deterioration is the first step in making a decision concerning selection of the repair process.

Parking structure deterioration can be related to some or all of the following causes:

- (A) Poor concrete quality assurance during the construction phase.
- (B) Lack of proper concrete cover for the reinforcing steel.
- (C) Expansion joint leakage
- (D) Freeze-thaw cycle
- (E) Cracking
- (F) Deicing salts

The main cause for deterioration is the use of deicing salts on roadways. Vehicles carry the salts onto the parking structure. Melted ice transfers oxygen and dissolved salts through voids in the concrete to the surface of the reinforcing steel, initiating the corrosion process. The physical presence of corrosion products creates pressure, causing the concrete to crack. The salt laden water can then freely penetrate the reinforcing steel and hasten the destructive corrosion process. Spalling and delamination ensue resulting in total deterioration of the parking structure deck.

In recent years, several repair techniques have been used to attempt to repair parking structure decks. Some of these include the use of superplasticized concrete, silica fume concrete, low water-cement ratio, high air concrete, deck coatings and deck sealers. Each has inherent advantages and disadvantages that are not within the scope of this paper, but one should consider every technique when selecting a repair process.

This paper will address the use of BASF Styrofan 1186, Styrene-Butadiene latex modified concrete overlay as the method of rehabilitation for parking structure decks, specifically the Gateway Center Parking Garage in Pittsburgh, Pennsylvania. Specifications for the repair area and the production, placement, finishing and curing of the latex modified concrete will be presented.

BACKGROUND

Latex modified concrete has been used primarily in the rehabilitation of bridge decks. Over 10,000 bridges have been rehabilitated using a Styrene-Butadiene latex modified concrete overlay. It has proved to be a long lasting and cost effective repair method for this application.

Parking structure and bridge decks are both subject to high concentration of deicing salts. It is expected that the excellent performance record established for latex modified concrete in bridge decks will be duplicated in parking structure repairs.

COST FACTORS

The final decision of a repair technique is usually based solely on economic considerations. In the rehabilitation of a parking structure, the tendency in the past has been to consider only the initial cost of the repair. In such instances, latex modified concrete overlays are precluded.

Styrene-Butadiene latex modified concrete overlays cost somewhat more initially than other repair systems. However, when consideration is given to the extended life of the system and the low maintenance cost, overall costs are lower than those for alternative systems. (bridge deck repair data projects a twenty year life).

A life cycle cost analysis has been prepared by Wiss, Janney, Elstner & Associates to demonstrate this point.

PROPERTIES OF STYRENE-BUTADIENE LATEX

Styrene-Butadiene latex is a dispersion of organic polymer particles in water. It looks very much like milk, and is often referred to as "milk" on the job site. A quantitative indication of the magnitude of these particles is given in figure 1.

Typical properties for latex used in latex modified concrete are listed in figure 2.

PROPERTIES OF LATEX MODIFIED CONCRETE IN THE PLASTIC STATE

In the plastic state, latex modified concrete has a high slump (4 to 7 inches) at a very low water/cement ratio i.e. .30 to .40. This combination of properties is attributable to the effect of the several different components of the latex on the rheology of the concrete mix. The latex modified concrete system provides a concrete that can be easily placed and worked. This system has the added benefit of a long slump life.

PROPERTIES OF LATEX MODIFIED CONCRETE IN THE CURED STATE

Latex modified concrete exhibits many desirable properties in the cured state. Among them are resistance to chloride permeability, resistance to deice scaling and increased bond and flexural strength.

CORROSION REDUCTION WITH LATEX MODIFIED CONCRETE

The role of Styrene-Butadiene latex in improving the resistance of reinforced concrete to chloride ion attack is not completely understood, but it is believed that the low water-cement ratio permitted by the use of latex results in a higher density concrete. Fewer voids are formed during curing and those that do form are smaller than those produced in conventional water-cement ratio concretes.

In addition, the latex is thought to form an elastic film through the matrix of the concrete that helps to reduce formation of hairline cracks during the curing process.

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The end result is a concrete that is much more impervious to penetration by chloride laden water to the reinforcing steel, compared with unmodified concrete.

RESISTANCE TO DEICE SCALING

There is dramatically increased resistance to deice scaling of a horizontal concrete surface exposed to freeze-thaw cycles in the presence of deicing chemicals.

ADHESION OF CONCRETES AND SHEAR BOND STRENGTH

Styrene-Butadiene latex modified concrete increases bond strength (adhesion of the overlay to base concrete). A good bond between the repair material used in the overlay and the base concrete allows the deck to function as one in flexure.

FLEXURAL STRENGTHS

The importance of this flexibility is to avoid stresses between the overlay and base concrete.

BRIEF DESCRIPTION OF THE GATEWAY CENTER PARKING FACILITY

The Gateway Center Parking facility is an enclosed three level underground structure with parking spaces for 750 vehicles. It is open on two sides with ramps to street level with each ramp providing ingress and egress. The structure had been in service 25 years prior to the time when repairs and rehabilitation were undertaken.

The owners of the Gateway Center parking garage chose to use Styrene-Butadiene latex modified concrete because of the ease of placement and workability, superior flexural and bonding strength, deicer scaling resistance, absorbed chloride permeability resistance and long-term savings.

TECHNICAL SPECIFICATION FOR THE REPAIR

The work involved the in-place repair of all existing slabs, walls, columns, ramps and beams and the installation of expansion joints and trench drains. This paper will be concerned only with the deck slab repair.

Workers tested the slabs using the "chain drag" method. A map was made targeting suspect areas. A mechanical scarifier removed a minimum of one quarter inch of concrete from the deck surfaces. The decks were sounded again with hammer and chain drag. In hollow sounding areas, a 30 lb. jackhammer was used to remove the deteriorated concrete. All areas were then resounded and additional concrete was removed where necessary. The slab was removed to full depth in areas where concrete had deteriorated to two-thirds of the original slab depth.



FULL DEPTH CONCRETE REPAIR

Workers sandblasted the reinforcing steel after all unsound concrete was removed. Reinforcing steel was replaced in areas where more than 10% of the section had been lost. The actual usage replacement of reinforcing steel exceeded the original estimate by about five times.

Bottom steel was replaced using thirty-six diameters of lap. Top steel used forty-eight diameters of lap, extending beyond the last point of the lost section. The decks were shored using post shores.

A low water-cement ratio, patching concrete was used for full depth repair to the top of the scarified surface of the existing slab.

Specification for the low water-cement ratio concrete was as follows:

	1 Cubic Yard	
Cement Factor		= 658 lbs.
Water Cement Ratio		= 0.38 to 0.42
Air Content		= 4% to 8%
Slump		= 1 1/2" to 2 1/2"
Compressive Strength at 28 days		= 4,000 psi

The low water-cement ratio concrete received a light rake finish after placement. The patch was wet cured for seven days, then air dried for seven days. Forms were stripped when the concrete reached 85% of the specified 28 day compressive strength.

STYRENE-BUTADIENE LATEX MODIFIED CONCRETE BONDED TOPPING

The approved mix design for the Styrene-Butadiene Latex Modified Concrete is illustrated in figure 8.

All surfaces were sandblasted and any remaining contaminated areas were re-scarified before placing the latex modified concrete topping.

The prepared surface was sprayed with water until thoroughly wet. Workers used compressed air to remove all standing water immediately prior to placement.

A Styrene-Butadiene latex modified grout was broomed onto all horizontal and vertical surfaces. The grout was prepared by brooming out the coarse aggregate from the latex modified concrete used in the placement.

PRODUCTION AND TRANSPORTATION OF THE LATEX MODIFIED CONCRETE

The latex modified concrete was produced by a concrete mobile unit. The concrete was calibrated prior to each pour. Workers tested the yield, temperature, slump and percentage of air at the onset of each production batch, approximately every 13 cubic yards.

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The work was done in winter months and concrete was produced only when a minimum temperature specification of 40°F could be met. Data showing the strength development at 40°F and 72°F is shown in figure 9.

"Georgia buggies" were used to transport the latex modified concrete to the placement site.

PLACING, FINISHING AND CURING

Prior to each placement, wind barricades were erected. No external heat source was used. The ambient temperature in the parking structure was approximately 45°F and slab temperature was maintained at approximately 45°F under these conditions.

The Styrene-Butadiene latex modified concrete overlay was placed and struck off slightly above grade, then consolidated with hand-held vibrators. It was then bull-floated and a rough broom finish was applied.

Wet burlap was used to cure the overlay for the first 24 hours. The burlap was covered with polyethylene sheets to prevent evaporation of water. After this period, the wet burlap was removed and the overlay was permitted to air cure for an additional 72 hours. After placement, work was restricted within one bay in any direction for 48 hours.

No major problems were encountered in the placement or curing phase of the rehabilitation. The original estimate for the latex modified concrete topping was 2,000 cubic yards. Actual usage was over 3,000 cubic yards.

CONCLUSIONS AND RECOMMENDATIONS

A. The scope of the Gateway Center rehabilitation project was seriously underestimated. This appears to be a common problem which points to the fact that many parking structures are in far worse condition than anticipated at the outset of the repair projects.

It would be deplorable if the structural failure of a parking garage is necessary to call needed attention to this fact.

B. The Styrene-Butadiene latex modified concrete system is cost effective for the repair and rehabilitation of parking structures, as well as bridge decks and marine structures.

C. Styrene-Butadiene latex modified concrete provides a means of protecting reinforcing steel from corrosion brought on by chloride ion penetration.

D. Styrene-Butadiene latex modified concrete can be produced and placed in parking structures during the winter months with few problems.

E. As a recommendation, the old adage, "an ounce of prevention is worth a pound of cure," should be given special consideration when constructing new parking facilities. A coordinated approach involving an overlay of Styrene-Butadiene latex modified concrete in the original construction phase should be thoroughly investigated.

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Life Cycle Cost Analysis
Wiss, Janney, Elstner & Associates

Comparative Life Cycle Cost Analysis for Different Surface Protection Methods

The following economic analysis was originally prepared in 1984 by R.G. Tracy of Tracy Material Consultants, Kalamazoo, Michigan. The prices have been updated to 1988 values by Wiss, Janney, Elstner & Associates, Inc., Northbrook, Illinois.

Problem Assumptions:

- 100,000 square foot, 10-year old, two-way flat plate structure
- Slab repairs are required on 20% of the surface
- Latex modified concrete (LMC) is to be used for patching in all cases as follows:

Type 1 repairs (shallow) to 10,000 sq ft @ \$8.50/sq ft	\$85,000
Type 2 repairs (deep) to 8,000 sq ft @ \$12.00/sq ft	\$96,000
Type 3 repairs (full depth) to 2,000 sq ft @ \$20.00/sq ft	<u>\$40,000</u>
Total Slab Repair Costs	\$221,000

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LMC OVERLAYS 20-year expected life		DECK COATINGS 8-year expected life		SURFACE SEALERS 4-year expected life	
Slab repairs to 20,000 sq ft	\$221,000	Slab repairs to 20,000 sq ft	\$221,000	Slab repairs to 20,000 sq ft	\$221,000
Plus overlay cost 100,000 sq ft @ \$4.00/ sq ft	<u>\$400,000</u>	Plus coating cost 100,000 sq ft @ \$3.00/ sq ft	\$300,000	Plus sealer cost 100,000 sq ft @ \$.75/ sq ft	\$75,000
TOTAL INITIAL COST	\$621,000	TOTAL INITIAL COST	\$521,000	TOTAL INITIAL COST	\$296,000
		Plus replacement of coating after 8 and 16 years, assume 4% annual inflation	\$972,480	Plus repeat sealer application after 4, 8, 12 and 16 years, assume 4% annual inflation	\$450,938
Plus maintenance costs, assume 1/2% (500 sq ft) additional repair annually @ \$10.50/ sq ft first year, (\$5,000) 4% inflation for future years, 20 years total	<u>\$162,588</u>	Plus maintenance costs, assume 1% additional concrete and coating re- pair(1,000 sq ft) @ \$13.00/ sq ft first year, (\$12,500) 4% inflation for future years, 20 years total	<u>\$360,459</u>	Plus maintenance costs, assume 2% additional concrete patching (2,000 sq ft) @ \$10.50/ sq ft first year, (\$21,000) 4% inflation for future years, 20 years total	<u>\$524,091</u>
TOTAL INVEST- MENT FOR 20 YEARS SERVICE	\$783,588	TOTAL INVEST- MENT FOR 20 YEARS SERVICE	\$1,853,939	TOTAL INVEST- MENT FOR 20 YEARS SERVICE	\$1,270,091
Net present value of investment assuming 10% discount rate	\$682,381	Net present value of investment assuming 10% discount rate	\$973,003	Net present value of investment assuming 10% discount rate	\$668,671



1. COMMON PIN	→ 1 MM ←
AREA = $7.8 \times 10^{13} \text{ \AA}^2$ ($\text{\AA}$ = ANGSTROM)	
AREA OF 2000 $\text{\AA}$ DIAMETER LATEX PARTICLE	
= $3.1 \times 10^6 \text{ \AA}^2$	
NUMBER OF LATEX PARTICLES REQUIRED TO COVER THE PIN HEAD = 25,000,000	
2. ONE POUND LATEX OF 2000 $\text{\AA}$ DIAMETER PARTICLES	
SURFACE AREA = 18,000 YARD	
3. ITEM	PARTICLE DIAMETER $\text{\AA}$
LATEX	2,000
CEMENT	20,000 +
SAND	50,000 +
AGGREGATE	50,000,000 +

Fig. 1--Latex particle size

POLYMER TYPE:	STYRENE-BUTADIENE COPOLYMER
TOTAL SOLIDS, % BY WT.	47.0
pH	10.5
VISCOSITY, cps	85
SURFACE TENSION, Dynes/cm	40
PARTICLE SIZE (ANGSTROMS)	2000
FREEZE THAW STABILITY	PASS 2-CYCLE
WEIGHT (Lb./GALLON)	8.4

Fig. 2--Typical latex properties

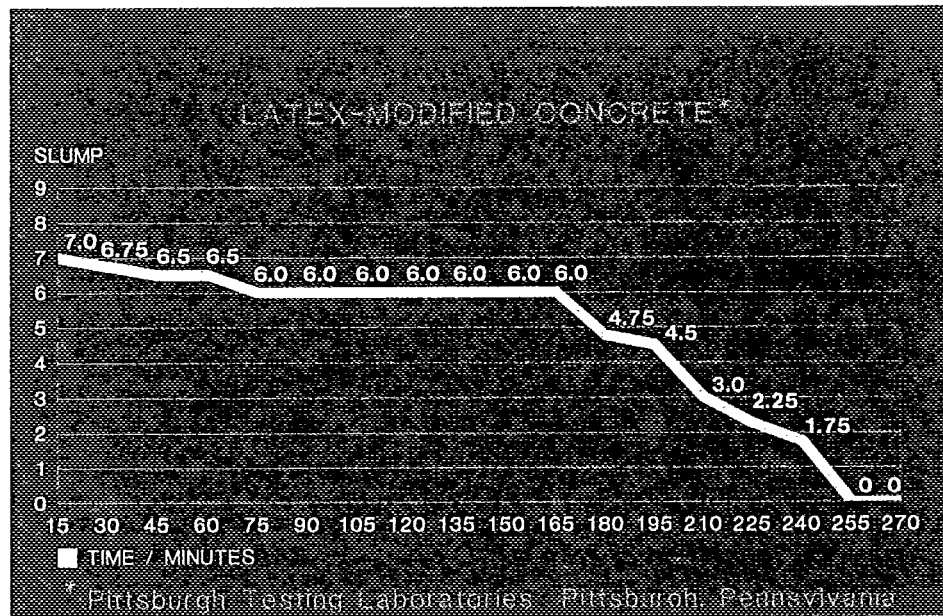


Fig. 3--Slump loss

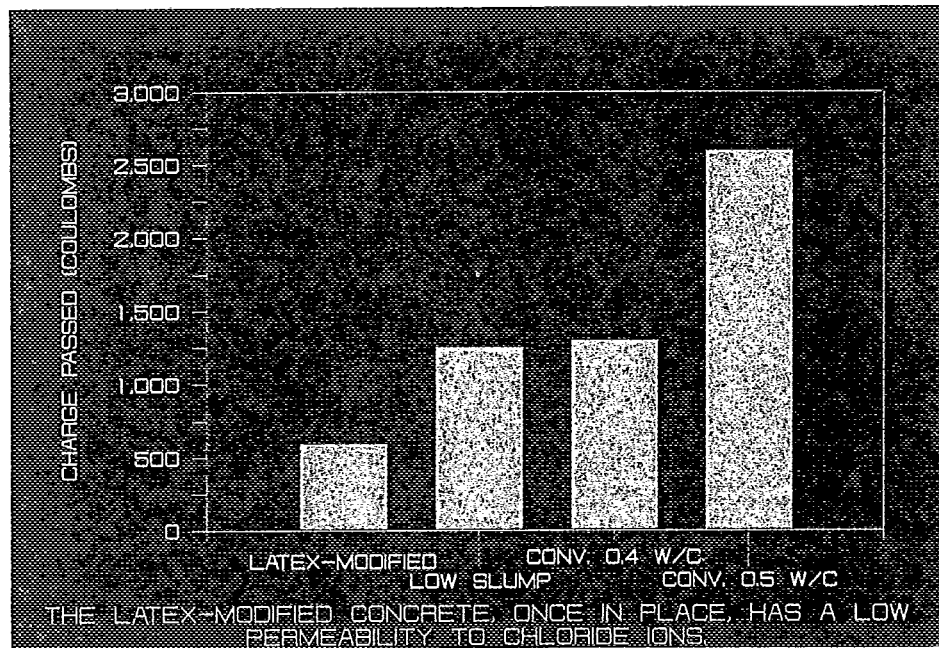


Fig. 4--Chloride permeability

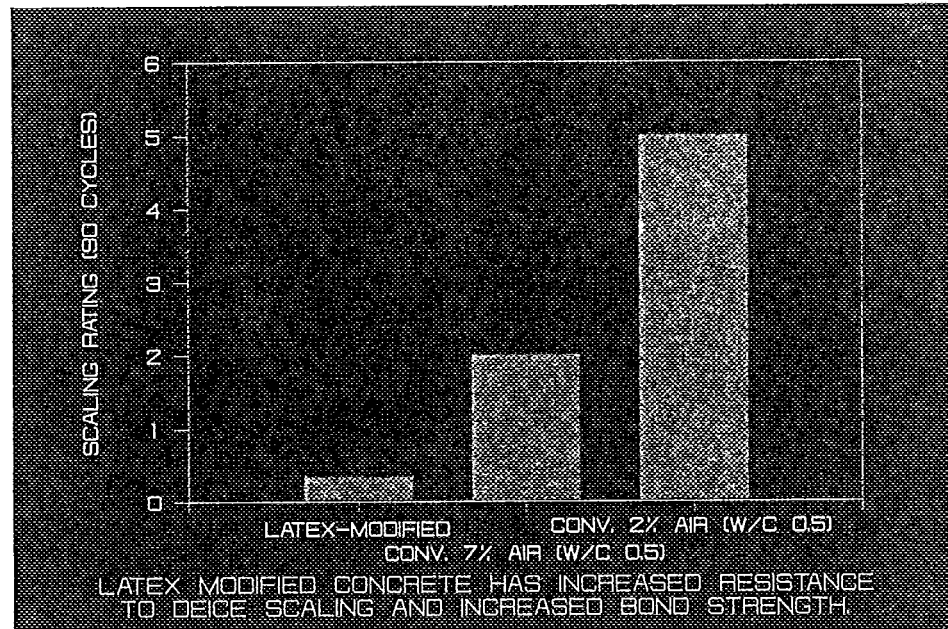


Fig. 5---Resistance to deice scaling

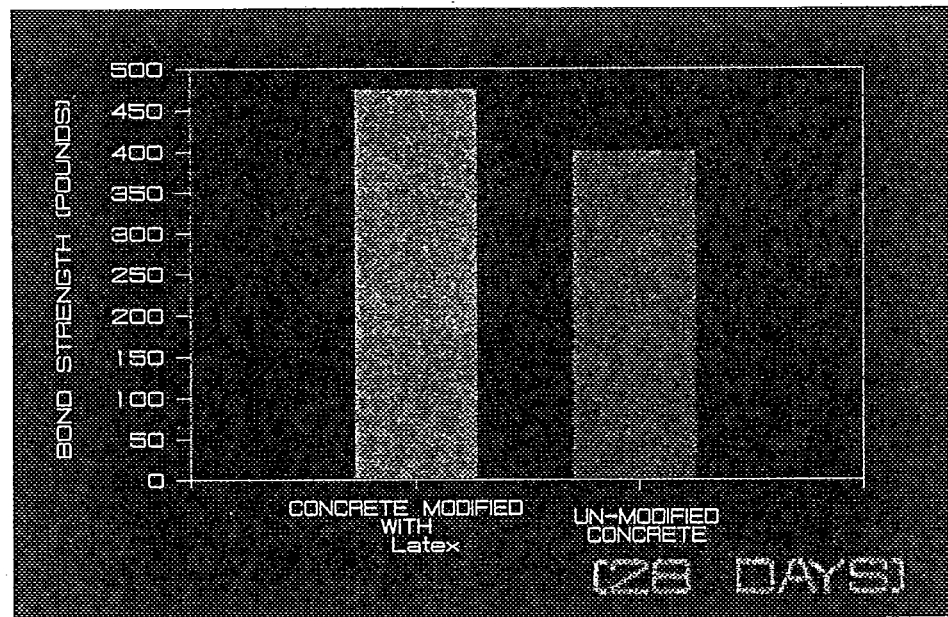


Fig. 6---Adhesion of concretes

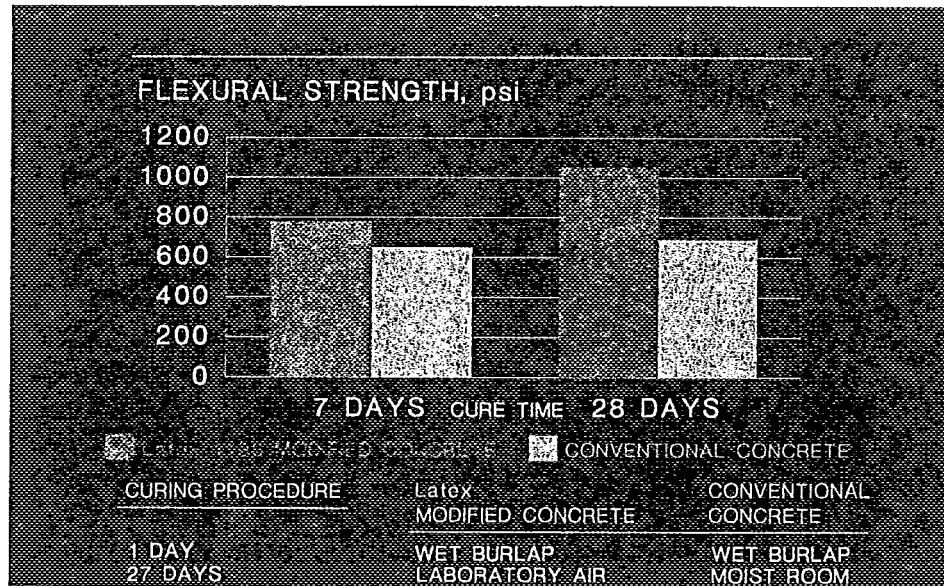


Fig. 7--Flexural strengths

THE APPROVED MIX DESIGN FOR THE LATEX-MODIFIED CONCRETE WAS AS FOLLOWS:

1 CUBIC YARD

CEMENT:	PORTLAND I/II	= 658 lbs.
LATEX:		= 206 lbs.
COARSE AGGREGATE:	(#8 STONE) (SHIPPINGPORT, PA. SAND & GRAVEL)	= 1,054 lbs.
WATER:	(CITY)	= 115 lbs.
FINE AGGREGATE:	(NATURAL SAND) (SHIPPINGPORT, PA. SAND & GRAVEL)	= 1,725 lbs.

Fig. 8--LMC bonded topping

	40°F.	72°F.		40°F.	72°F.
1 DAY	1580	2750	14 DAY	4200	5200
	1620	2830		4220	5300
	1390	2780		4180	5260
AVERAGE	1530	2790	AVERAGE	4200	5250
4 DAY	4060	4540	28 DAY	4780	6050
	3540	4330		4690	6000
	3810	4350		4710	6050
AVERAGE	3800	4410	AVERAGE	4730	6030
7 DAY	3660	4440			
	3780	4620			
	4060	4550			
AVERAGE	3830	4530			

Fig. 9--Compressive strength

SP 116-2

Computer-Aided Design of Precast Polymer Concrete Line-Drainage Channels

by U. Zanke

Synopsis: A program for the evaluation of discharge in Line-Drainage-Systems was developed at the Hydraulics Institute at the FH Kiel/Eckernforde. Complex hydraulic situations are treated such as lateral inflow varying along the drainage channel with respect to the shape of the drainage area. Sudden changes in the cross sections and flow hindering structures are to be treated. Additionally, sand may be accumulated in the channels and the maximum possible accumulation can be known for a given runoff. The programs were checked against hydraulic investigations.

Keywords: computer programs; drainage; economics; polymer concrete; precast concrete; safety; structural design

16 Zanke

About the Author

Dr. Ulrich Zanke is a Professor of Civil Engineering at the University of Hannover, W. Germany. His research and development includes, experimental hydraulics, sediment transport, and coastal engineering and over 16 years of experience with Civil engineering projects.

INTRODUCTION

Drainage troughs are installed for the drainage of liquids from firm surfaces. Typical areas include, parking areas, athletic fields or airport surfaces, from which precipitated water must be drained. Another important area is floor surfaces in facilities such as, the chemical and food processing industry, in which, in addition to water, industrial liquids must also be removed reliably.

As a rule the inflow comes from the side, but depending on locality, an additional amount of water must also be accommodated from above. The surfaces to be drained do not always have the same width so that the time inflow into the channels can vary over their length. If one must figure on an increased inflow of solids, purification systems, which partially block the flow cross section, must be estimated. Furthermore back-up (damming) effects can take place from connecting sewage systems. In such cases drainage channels must be designed safely as well as economically. For this purpose empirically sized precast polymer concrete channels of different nominal widths and cross-sectional shapes as well as various floor constructions (without drop, step-wise slopes, continuous slopes) have been utilized. Using this method, the upper parts of longer channels, which experience less loading, are designed to be smaller.

Until now the designing of such drainage lines presented problems in practice because of the time required to carry out the complex calculations and the potential for error inherent in these calculations.

SOLUTION METHODS

Because of the described situations, a simple calculation of the water levels and thus the carrying capacity of the channels is not adequate using conventional flow formulas. To achieve the design goals of safety and economy, two methods can be considered; the hydraulic experiment and the calculation of the float (level) line differential equation.

This paper presents the results of the hydraulics experiment and compares these with the calculated results obtained using special case "sand accumulation". To calculate the sand accumulation an additional iteration loop was inserted into the calculation. The following method can be used for a small inflow of sand:

The shear stress is calculated for every flow condition and is compared with the critical shear stress (SHIELD (1936). ZANKE (1982)). If the actual shear stress is too small, a slight aggradation of the bottom occurs. In the opposite case, erosion occurs until a firm bottom layer is attained. This control loop is iterated long enough until the hydro-mechanical and the sedimentological parameters are in tune. In this way one obtains step by step the sand bottom along the ducts. Attention must be paid here to the differences in roughness of the channels or the sand bottom, which have a distinct influence on the water levels. Corresponding laboratory tests were conducted in the above-named investigations at the TU of Graz so that comparison conditions with the calculation were available. Measurements and calculation are compared in figure 5. Taking into account the uncertainty, with which one can determine today the boundary "at rest/movement" on the bottom, the result can be considered to be quite good.

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HYDRAULIC TEST

For the comparatively simple case of a prismatic channel with side inflow remaining constant laboratory investigations were conducted, e.g., at the Technical University of Graz, with a channel section 10 m long (NEMECZEK, 10..). Although one can set up a sizing diagram from such tests, a diagram thus obtained can be used only for the tested channel type and the inflow variations used in the test. For other channel types, line lengths or inflows with different distributions, the results cannot be used. The same is true for the case of potential built-ins, such as channel cleaning facilities or supports for cover grates. These built-ins interfere with the water outflow, depending on degree of immersion. For the case of a production-caused change in wall roughness the tests must be repeated.

This means that for arbitrary configurations of inflow and channel type appropriate new test series must always be made.

NUMERICAL MODEL

In the subject case calculations are considerably more flexible. The hydromechanical principles are known. They can be derived from the impulse expression or taken from the literature (e.g. CHOW (1959), SCHROEDER (1966), HENDERSON (1966)). Depending on the type of simplifications that must necessarily be made, one obtains outwardly slightly different solutions which, however, hardly differ from each other as far as the results go. Thus one obtains, e.g., according to CHOW/HENDERSON the differential equation:

$$\frac{dh}{dx} = \frac{J_s - J_v - \frac{2Q}{gA^2} \frac{dQ}{dx}}{1 - Fr^2}$$

h = depth of water

x = running length

J_s = required slope

J_v = loss in slope

Q = throughflow at position x

= flow cross section

g = gravitational constant

Fr = FROUDE number

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The equation cannot be solved fundamentally. It must be converted into a suitable differential form and solved numerically. Local flow losses result from the named built-ins and cross section discontinuities. However, these flow losses again depend on the (at first still unknown) solution of the differential equation. This is also true for the wall friction losses, which themselves again occur as an implicit equation (can be solved only by a series of trial calculations). Thus several implicit equations are immixed within each other. Because of the calculation expenditures and the susceptibility to errors a manual calculation makes no sense.

For the simplest case, the drainage channel with constant side inflow, a computer program had already been established in 1979 by the Ecole Polytechnique Toulouse. Since at that time programs were still dependent on large-scale computers, only the sample calculations of specific cases was left here for practical applications.

SOLUTION

Because of the sufficiently-high-performance desk computers available today there is opened up a new dimension for the sizing of the channels named a CAE-solution (Computer Aided Engineering) becomes possible because then even very complex and high computer-time programs can easily be carried out at every desk.

This, however, creates a new problem. Until now the test- or calculation results were weighted and evaluated by experienced engineers. Thus errors or solutions, although correct mathematically but physically impossible, were filtered out. Programs such as the mentioned older program of the TU Toulouse were operated by the originator himself and thus had to consist of only the essential calculations. The work of interpretation can be taken over within limits by a program system at a desk computer. For example, it is possible to a large extent to intercept meaningless input values. It is also possible to filter out wrong secondary solutions up to a certain extent with justifiable expenditures. Thus program break-downs and useless solutions are largely avoided. The subject programs were equipped in this sense. A complete replacement of the subject-competent interpretation, a so-called system of experts, however, would require program expenditures where

the actual computer program sinks completely into the background. Whether such expenditures are economical, depends on the individual case.

This means that for a CAE-solution, in addition to the actual simulation of the flow processes, a more or less considerable program scope must be available to the user in every case as quasi intelligent filter.

As a near-practical solution we established in the present case a program system with which the following work steps can be handled:

- input and manipulation of the hydraulic and geometric situations with checking of the logic
- calculation of the associated water levels and flow velocities
- calculation of the degree of flow utilization of the channels
- calculation of the maximum drainage amounts for given geometric conditions.
- calculation of the end state of aggradation for given boundary conditions.
- graphic display of the results

Furthermore an appended data management such as the access to channel configurations already used before and their alterations is obvious.

TEST FACILITY

A test carrier was set up (figure 2) for assuring the calculated water levels as well as for the empirical determination of the loss coefficients of changes in cross section, partially immersed structural parts as well as for channel branch connections. The carrier has a length of 20 m and can be tilted to 5%. The "to be tested" channels with cross sections up to ca. 30 x 40 cm were placed on the carrier as finished elements of 1 m length and were cemented to each other. Side inflow occurred from a slightly sloping, height-adjustable plane made of aluminum plates. To achieve a quiescent inflow such as from a large rained-on area we covered a strip ca. 20 cm wide with mats made of rubberized coir fibers. Water flowed unto these mats from ca. 100 adjustable nozzles (see figure 3). The outflow from the mats is quiet so that the inflow at the last 30 cm is nearly as in nature. The roughness of the inflow surface can be changed with suitable cover layers.

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QUALITY OF THE CALCULATIONS

As an essential result it was found that the deviations between measurement and calculation for the water levels lie within the range of the test accuracy of the test stand (ca. ± 1 mm). An example is shown in figure 4.

SUMMARY

Desk Computers can be used today to solve hydraulics problems which previously could only be solved by hydraulic experiment. In many cases the calculations were not made because of the complexity of the experiments and limited access to institutions with facilities suitable for carrying out these experiments. The result was conservative design of drainage systems that met the requirement of safety, but may have not been the most economical solution.

A major part of the software development effort has concerned defining boundaries so that solutions that are mathematically correct, but physically impossible to achieve are filtered and rejected. Therefore, a knowledgeable user with a thorough understanding of the hydraulics problem to be solved can use the computer aided design program and achieve a practical solution which meets the design criteria of safety and economy.

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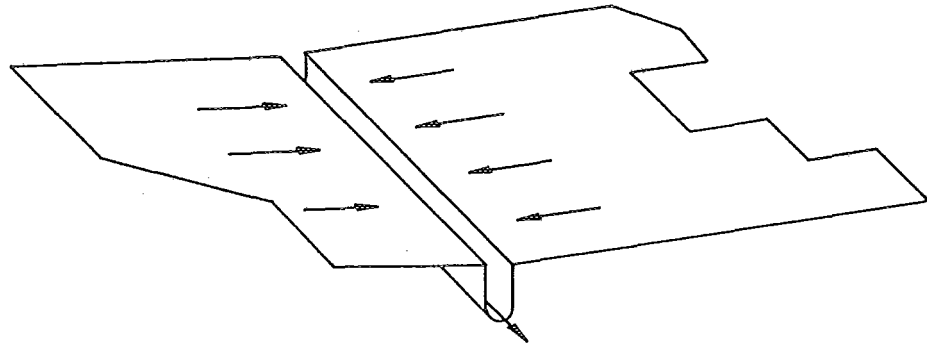


Fig. 1--Typical situation of a surface drainage

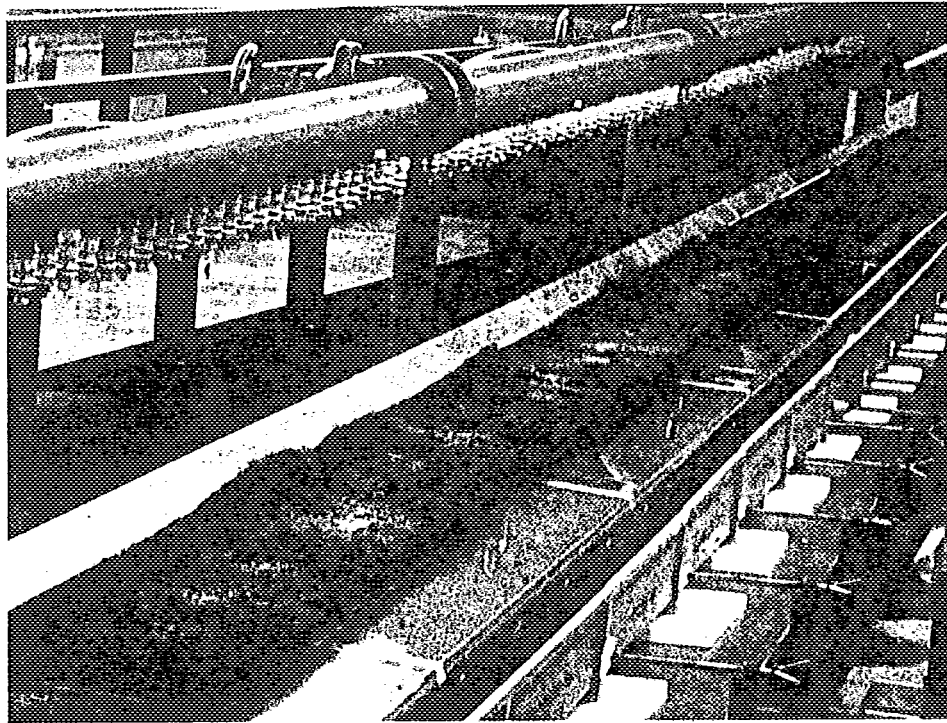


Fig. 2--Test facility

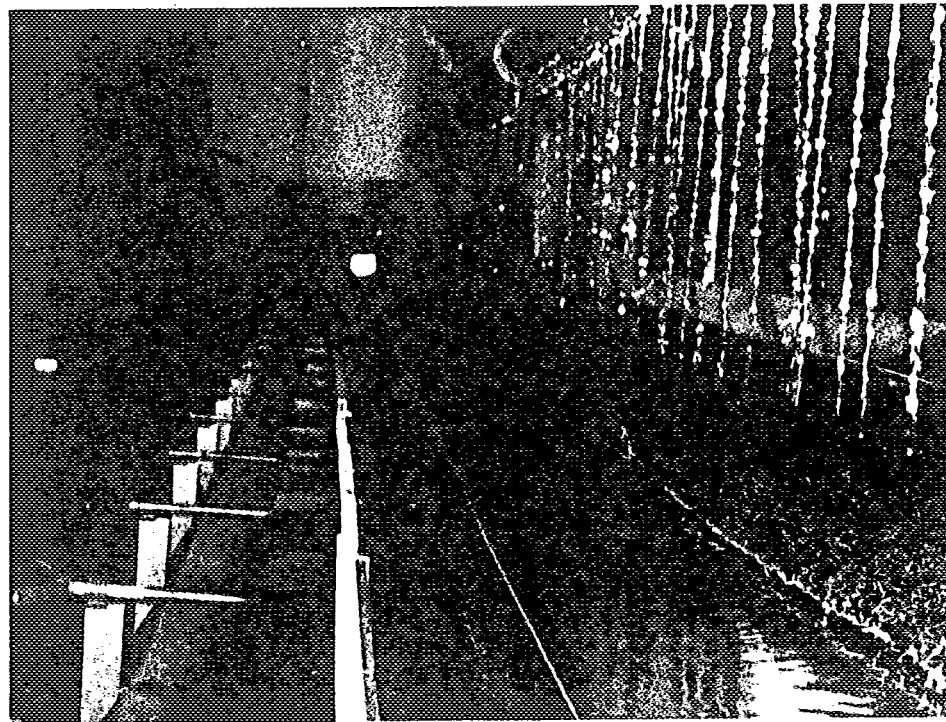


Fig. 3--Simulation of precipitation of large areas

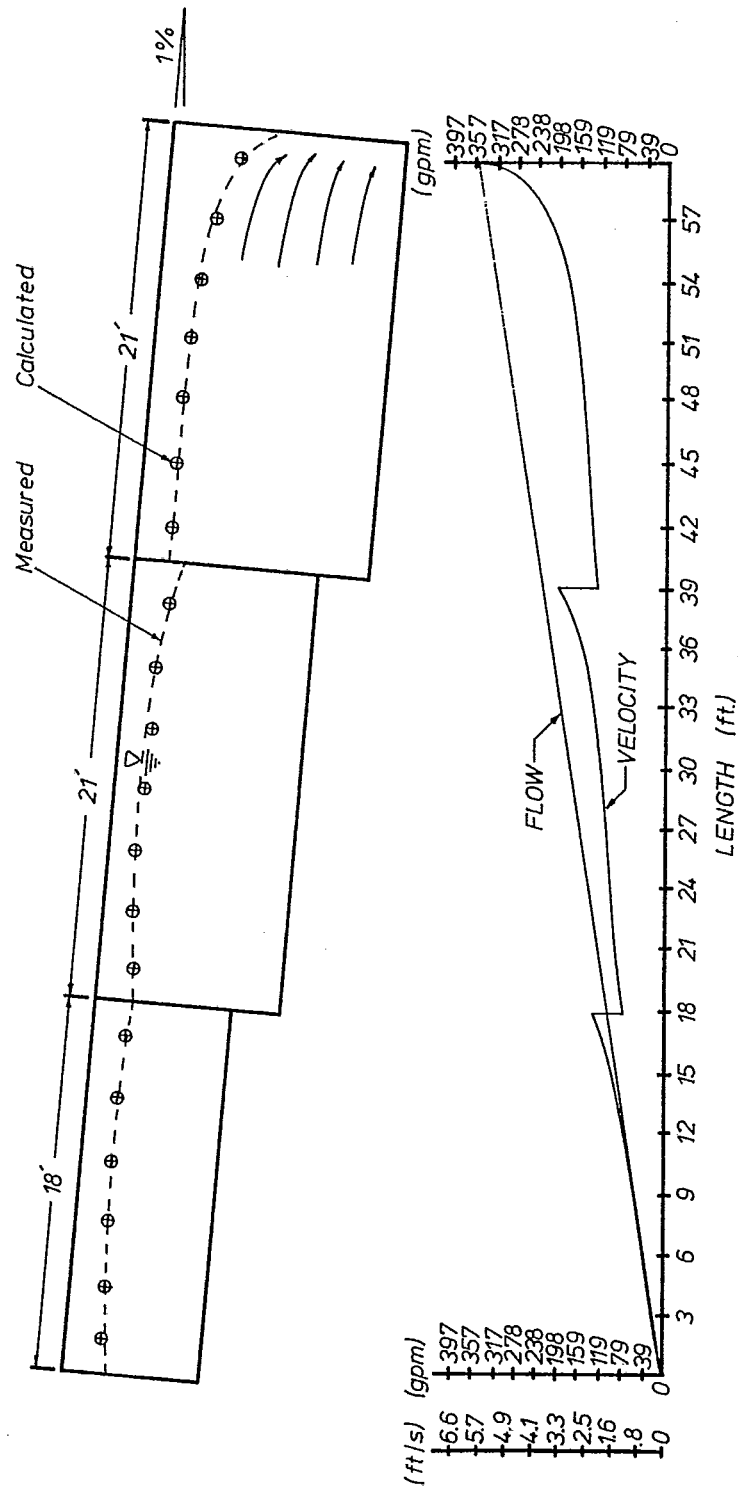


Fig. 4--Calculated result and comparison of measurement and calculation

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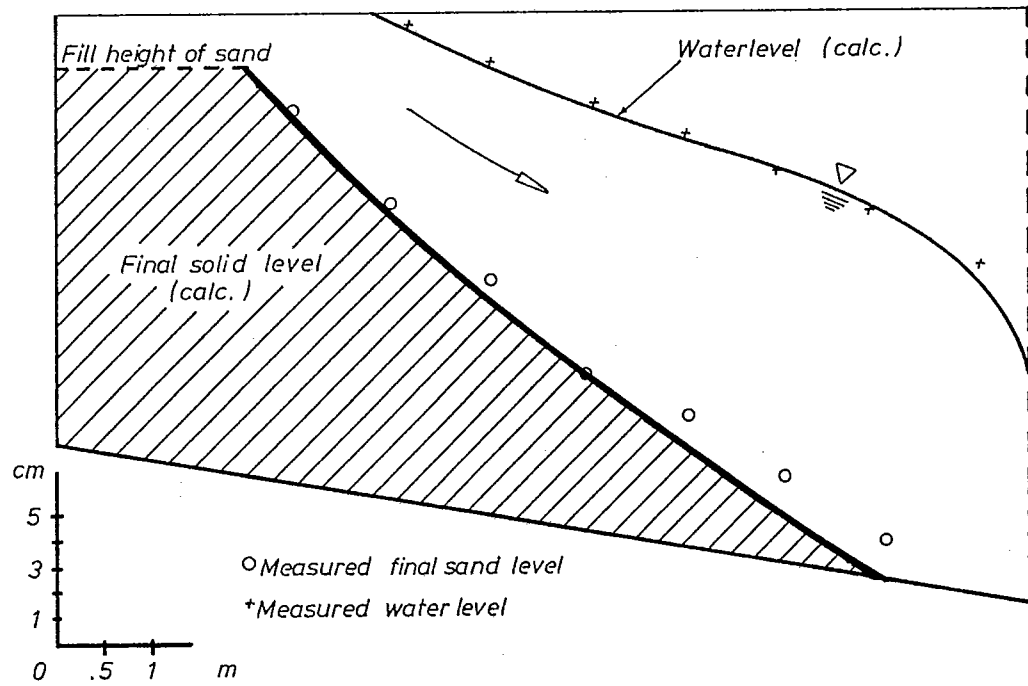


Fig. 5--Measured and calculated sand accumulation end state

SP 116-3

Cold Weather Polymer Concrete Repair to Palisades Dam, Idaho

by W.G. Smoak

Synopsis: Acrylic polymer concrete was used to repair absorptive aggregate "pop outs" on the concrete stilling basin apron of Palisades Dam, Idaho. This work was accomplished during December 1986 when ambient air temperatures were below -10 °F (-23 °C). Ice dikes were used to direct gate leakage away from the repair areas. Hot forced air heaters, gas-fired weed burners, and incandescent electrical lights were used to provide heat to the repairs.

Keywords: absorption; aggregates; cold weather construction; concrete dams; methyl methacrylate; polymer concrete; popouts; repairs

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INTRODUCTION

Palisades Dam and powerplant is a principal feature of the Bureau of Reclamation's multi-purpose Palisades Project.

The dam is a 270-ft (82-m) high zoned earthfill structure located on the South Fork of the Snake River, about 45 mi (72 km) southwest of Jackson, Wyoming. The dam crest length is 2,100 ft (640 m) long. Construction of the dam was completed in 1957, and the powerplant was placed in operation in 1958.

The dam outlet works is controlled by a fixed wheel gate at the entrance to a 26-ft (8-m) diameter outlet tunnel. This tunnel carries water through the left dam abutment to a steel manifold section and ultimately through regulating gates to the stilling basin. The outlet works capacity is 33,000 ft³/s (156 m³/s).

During an inspection of the outlet works stilling basin in the fall of 1986, the presence of numerous circular "pop outs" was detected on the concrete apron downstream of the regulating gates. These defects were generally circular with varying diameters but were uniformly 1 to 2 in (25 to 50 mm) deep. There was significant concern that under the high-velocity flow conditions experienced on the apron during spring runoff, these defects could become source sites for cavitation.

The decision was made to repair the apron during December of 1986. Since it was certain that additional defects of this type would continue to occur, a simple, economical repair system was needed for future use by dam maintenance personnel.

Upon request for assistance by the Palisades Project Office, the cause of these defects was determined to be the presence of some quantity of low quality, absorptive aggregate in the concrete. This aggregate, which was limestone, absorbs and retains moisture until freezing expansion causes a conical failure in the concrete with the weak aggregate particle at the bottom center of the defect.

The Bureau of Reclamation has been active in research and development of concrete-polymer materials since 1967, and its

Concrete and Structural Branch of the Division of Research and Laboratory Services had major responsibilities in the activities and in the development of concrete-polymer materials and procedures. A repair procedure utilizing commercial polymer concrete was recommended. The preparation of the damaged concrete and application of the repair materials were performed by Bureau maintenance personnel located at the dam. The necessity to repair the concrete during very low-temperature winter conditions placed unique requirements upon materials and personnel. Ambient air temperature in the vicinity of the dam was as low as -14°F (-25°C), but the stilling basin concrete was $+6^{\circ}\text{F}$ (-14°C) at the time of the repair.

REPAIR PROCEDURE AND MATERIALS

Maintenance personnel first constructed sand dikes to shield the repair area from water leaking from the regulating gates. The extremely low ambient temperatures soon turned the dikes into very effective ice dams which prevented additional water from entering the repair areas.

The perimeter of each damaged concrete area was sawcut to a depth of about 2 in (50 mm). The interior concrete was then removed to a uniform 2-in depth and sandblasted clean of debris. Figure 1 shows one of the repair sites that had been prepared and was ready to receive PC (polymer concrete). Snow and ice are apparent on the sand dike.

A total of 28 areas, varying in size from 1 ft^2 (0.1 m^2) to about 5 ft^2 (0.5 m^2) were prepared in a similar manner prior to mixing the PC.

The spillway apron was completely open to the elements, but a protected mixing area was obtained by covering one of the slide gate bays with a tarpaulin as shown in figure 2. This area was heated with a hot forced air heater. Even so, mixing was delayed when it was found that the bearings of the rotary drum mixer were frozen and required thawing, figure 3, before the mixer could be used.

A low-temperature version of PC containing increased promoter-catalyst concentrations had been obtained from the manufacturer, Rescon Technology Corporation. All the resin primer, PC components, and the aggregate supply were stored in a heated area until the beginning of mixing. Polymer concrete mixing and placement proceeded as follows:

Four or five of the repair areas were selected and surface heated with a gas-fired weed burner. Simultaneously, the primer resin and catalyst were mixed: 2 oz (59 mL) of catalyst to 1 qt (0.96 L) of resin. Workmen then brushed the primer onto the heated repair areas, as seen in figure 4, while the PC was being mixed. A normal batch of PC was

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prepared with 66 lb (30 kg) of PC powder, 66 lb of dry, 3/4-in (19-mm) maximum size aggregate, and approximately 1 gal (3.8 L) of PC resin.

Workmen transported the mixed PC to the individual repair areas in small buckets to permit the minimum possible exposure to the cold ambient temperatures. The PC was then tamped into the repair area and troweled to a finished surface as quickly as possible, figure 5.

Protection and curing of the PC repairs were accomplished in two ways. If the PC volume in the repair was relatively large and could be expected to develop a polymerization exotherm, the repair was simply covered with 3- to 4-in (75- to 100-mm) thick fiberglass roll insulation placed in plastic bags. For smaller areas, electric light fixtures containing 300-watt incandescent bulbs and large reflectors were placed directly over the fresh PC as shown in figure 6. These lights were left in place for about an hour and then replaced with fiberglass insulation to retain the heat. The light fixtures were then placed over other fresh patches.

Placement and finishing of PC in all the repair areas was completed in about half a day. By early afternoon, all the repair areas had become hard, but the insulation was left in place overnight.

RESULTS AND INSPECTION

Maintenance personnel inspected the repairs several times during the winter of 1986-87. All repair areas were bonded to the old concrete, and no cracking was apparent in the PC. In May of 1987, following the spring runoff, a detailed inspection was performed by the author. There was no evidence of erosion/abrasion, cracking, or disbonding of the repairs, but several new "pop outs" were apparent on the concrete and will be repaired in the future. The absorptive aggregate "pop outs" on the concrete will most likely be a long-term continuing problem in the outlet works stilling basin at Palisades Dam. A simple, economical repair technique was developed to meet these future repair needs.



Fig. 1--Typical repair area after deteriorated concrete has been removed.

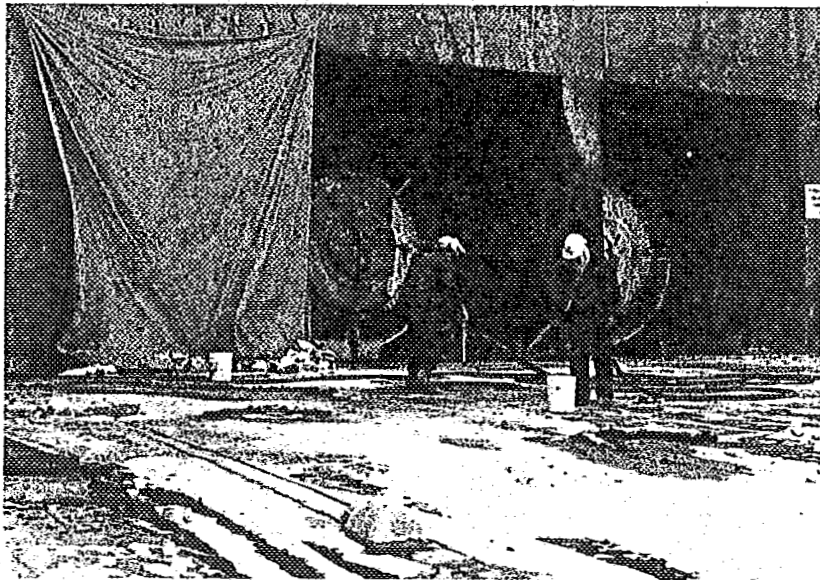


Fig. 2--One of the gate bays was covered with a tarpaulin to provide a semi-protected mixing area -- snow and ice are visible on the concrete apron

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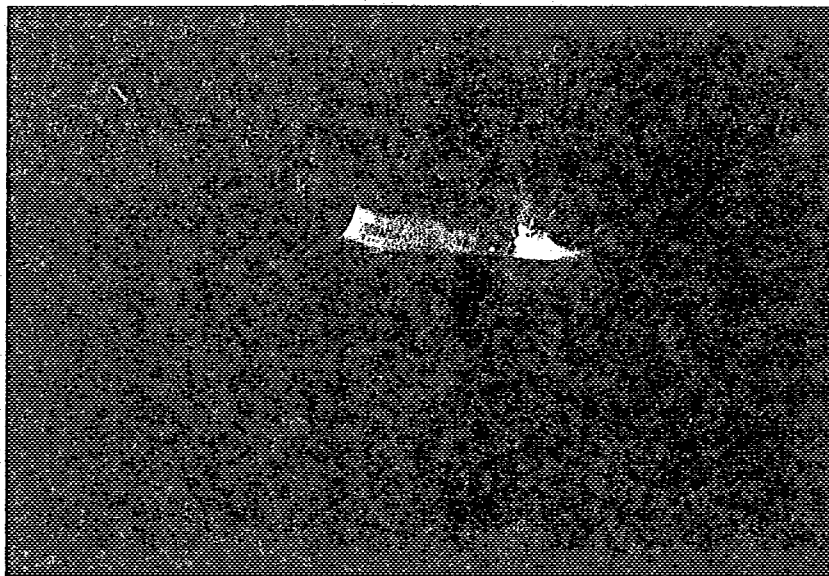


Fig. 3--A weed burner was needed to thaw the bearings of the rotary drum concrete mixer before PC could be mixed

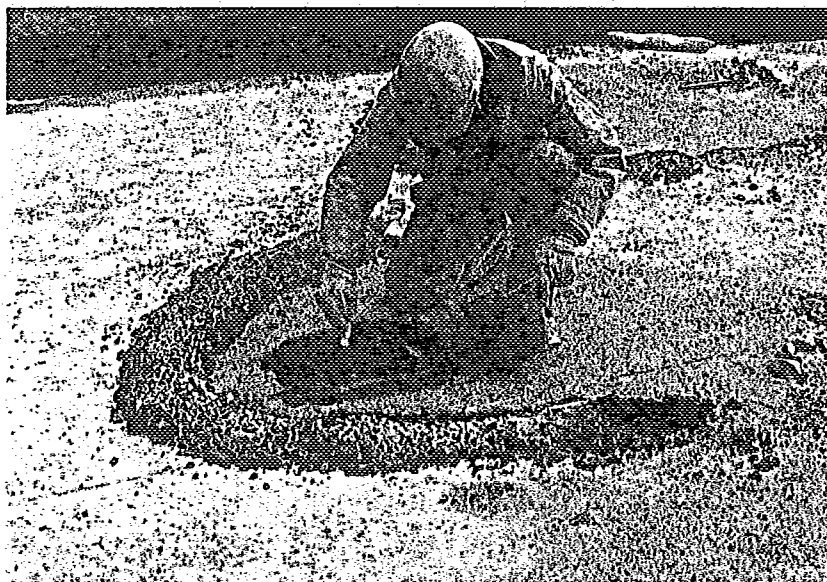


Fig. 4--PC primer was brushed into the prepared repair area immediately before the PC was placed



Fig. 5--Workmen carried only small quantities of PC from the mixing area in an attempt to prevent the mix from becoming cold prior to placement -- a steel trowel was used to finish the repair

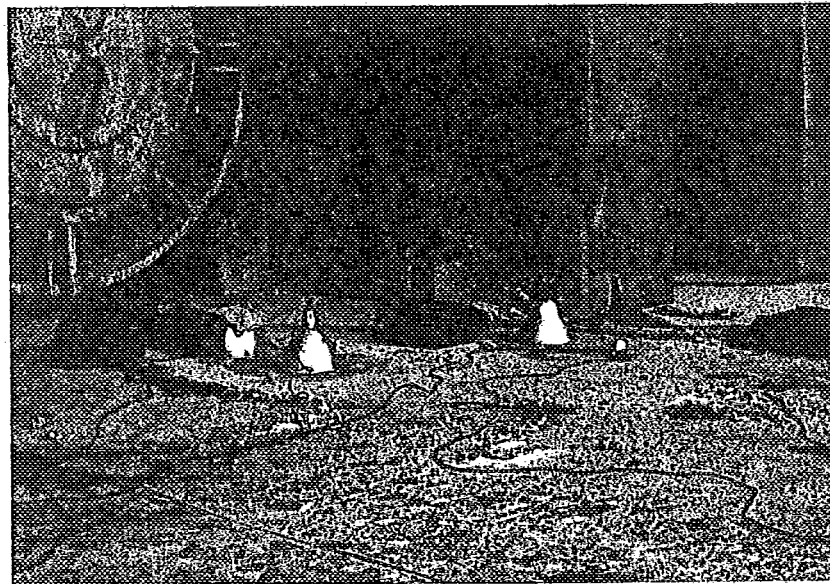


Fig. 6--Incandescent light fixtures and fiberglass insulation was used to heat and protect the repairs until curing had occurred

SP 116-4

Sandwich Beams with Polymer Concrete Facings

by K.J. Fridley, D.L. Wheat, and D.W. Fowler

Synopsis: A summary of the results of an experimental program to investigate the structural behavior of composite sandwich beams having polymer concrete (PC) top and bottom facings is presented. The possible application of PC as a facing material is of interest since it may provide an effective use for the material. A total of 18 different cross-sectional configurations were investigated. Polyester-based PC facings were used in conjunction with three separate core materials: sand-lightweight portland cement concrete, expanded polystyrene foam, and rigid polyurethane foam. Various combinations of core and facing thicknesses were used. Load-deflection responses, strain and stress distributions, modes of failure, and ultimate strengths of the beams were monitored. The experimental results were compared to sandwich beam theory and, in general, compared well. Overall, the results of the study suggest that the application of PC as a facing material holds high potential and warrants further attention.

Keywords: beams (supports); compressive strength; deflection; facings; loads (forces); polymer concrete; sandwich structures

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Dan L. Wheat is an Associate Professor of Civil Engineering at the University of Texas at Austin. He teaches courses in structural engineering and performs research in the areas of polymer concrete and wood engineering.

David W. Fowler is a Professor of Civil Engineering at the University of Texas at Austin where he teaches courses in structural engineering and conducts research involving polymer concrete. He has been active in polymer concrete research since 1969.

INTRODUCTION

Effective use of polymer concrete (PC) as a structural material requires that its positive characteristics, namely strength and durability, be optimized with respect to cost (1). The application of PC as a facing material for composite sandwich members such as beams and panels is of interest since it may meet this requirement and provide an effective and economical use for the material. The structural behavior of composite sandwich beams having PC facings is the subject of this paper.

MATERIALS

Polymer Concrete

A polyester-based PC was used as the facing material for each of the sandwich beams tested in this study. Since the PC was cast in thin sheets, the mix design was governed by ease of workability. The gradation and dry unit weights of the aggregates are listed in Table 1(a). The basic PC mix proportions are given in Table 1(b). The combination of benzoyl peroxide, the initiator, and dimethyl-para-toluidine, the promoter, insured polymerization with a working time of approximately 50 minutes.

Sand-Lightweight Portland Cement Concrete

Sand-lightweight portland cement concrete was used as a core material for 6 of 18 sandwich beams. The gradation and dry unit

weights of the aggregates are listed in Table 2(a). The mix proportions for the sand-lightweight concrete are shown in Table 2(b). Type III cement was used to provide early strength.

Expanded Polystyrene Foam

Expanded polystyrene foam, which was used for one-third of the beams, is a rigid, closed cell, plastic material with density dependent properties. A unit weight of 2 pcf was chosen for this study due to availability and strength considerations.

Rigid Polyurethane Foam

Rigid polyurethane foam, which was used in the 6 remaining beams, is likewise a closed cell plastic material with density dependent properties. A unit weight of 2 pcf again was chosen for this study.

TEST BEAMS AND PROCEDURES

Test Beams

A total of 18 cross-sectionally different composite sandwich beams were constructed and tested (4). The beams were constructed with a length of 5 ft (to be supported on a 4-ft simple span), a width of 6 in., and a depth of 3 to 5 in. depending on core and facing thicknesses. Table 3 summarizes cross-sectional details of the 18 beams and the relevant facing and core mechanical properties which were determined as part of the study.

The actual construction of the sandwich beams followed two separate procedures depending on the core material. For the beams with concrete cores the facings were cast directly on to precast portland cement concrete cores. The concrete cores were previously oven dried and had a methacrylate bond enhancer applied to the bonding surfaces to insure proper PC-portland cement concrete bond. A second fabrication procedure was required for the beams with foam cores. The facings were precast and adhered to presized cores with an epoxy adhesive. The reason for the second procedure was an incompatibility of the polyester resin with the foam materials, that is, the polyester resin would dissolve the polystyrene and polyurethane foams.

Instrumentation

Each of the beams with concrete cores was instrumented with a set of nine 60-mm electronic strain gages as shown in Fig. 1. The set of six strain gages shown in Fig. 1(a) was attached at

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the beam's mid-span to measure the flexural strains. The two strain gages at the extreme fibers were mounted on the exterior faces as close as possible to the side of the beam. The three strain gages illustrated in Fig. 1(b) were placed halfway between the load point and the support and measured the shearing strains produced in the concrete core. A single rosette gage was not used due to lack of sufficient gage length available in such gages, that is, a gage length of 60-mm was required due to aggregate size. The five strain gages adjacent to the PC-portland cement concrete boundaries (four shown in Fig. 1(a) and one in Fig. 1(b)) were mounted as close as possible to the interface without having their backings overlap.

The beams with polystyrene and polyurethane cores were instrumented as shown in Fig. 2. Six 60-mm electronic strain gages were attached to each beam at mid-span as shown in Fig. 2(a) and were used to measure the flexural strains produced in the facings during loading. The two gages at the extreme fibers were mounted on the exterior faces as close to the side of the beam as possible. The strain gages mounted adjacent to the PC-polystyrene and PC-polyurethane boundaries were placed as close as possible to the interface without having their backings overlap the boundary. A dial gage with 0.001-in. accuracy was used to measure the relative movement between the facings -- an approximate measure of the shear strain in the core assuming no slip in the facing-core bonds -- at a point midway between the load point and the support and was placed as shown in Fig. 2(b). The dial gage was mounted on the extreme fiber surface of the top facing. A rigid aluminum arm was mounted to the extreme fiber surface of the bottom facing directly opposite of the dial gage and extended up to the gage's deflection plunger. Any reading of relative facing movement recorded by the dial gage was assumed to be the result of shear deformation in the core.

Deflection measurements for all 18 beams were taken at the load points and mid-span using six 0.001-in. dial gages. The dial gages were placed on both sides of the beams at each deflection point. Each set of readings was averaged to provide the deflection at that point.

Testing Procedures

Each beam was loaded to failure as shown in Fig. 3. A double-channel support beam was placed on the loading table of a 120,000-lb deflection-controlled loading machine to provide a 4-ft simple span. The support beam was designed so that deflections under maximum expected loads would be negligible and not affect the test. The load was transferred from the test machine's loading head to the third points of the sandwich beams by a spreader beam. The beams were supported on and loaded through 1.5-in. rollers across the width of the beams. A neoprene bearing pad was placed between each of the rollers and

the beams to prevent local crushing. Constant rates of cross-head motion of 0.02 in./min for the beams with concrete cores and 0.075 in./min for the beams with polystyrene and polyurethane cores were used to load the beams. Deflection and strain readings were taken at 200-lb load increments (P/2 applied at each load point) for the beams with concrete cores and at 100-lb increments for the beams with polystyrene and polyurethane cores. The type of failure and the load at which the failure occurred were recorded.

TEST RESULTS

The load-deflection responses, stress and strain distributions, modes of failure, and ultimate strengths were determined for each of the 18 beams listed in Table 3. Selected results which best represent the observed behaviors are presented. Also, the results presented herein are compared to conventional sandwich beam theory. The theories are not discussed in this paper except where modifications or additions were made. Full discussions of sandwich beam theory can be found in refs. 2 and 4.

Load-Deflection Responses

Three typical load-deflection curves-- one for each core type-- are illustrated in Figs. 4, 5, and 6. The vertical axes represent the total applied load measured using the loading machine indicator. The horizontal axes of Figs. 4(a), 5(a), and 6(a) represent the deflection of the load points relative to the supports, while the horizontal axes of Figs. 4(b), 5(b), and 6(b) represent the mid-span deflection relative to the deflections at the load points. Thus, the curves of Figs. 4(a), 5(a), and 6(a) represent the load-deflection responses caused by flexure and shear combined, and the curves corresponding to Figs. 4(b), 5(b), and 6(b) represent the responses of the beams to the pure moment in the center third of the span (between the load points).

The measured responses for the beams with concrete cores were linear to approximately 70 percent of ultimate at which point flexural cracking occurred in the core inside the constant moment region. The deflections increased with smaller increases in load after cracking until the tensile PC facing finally ruptured. The increase in deflections after cracking was more noticeable in the beams with thin facings, such as SC1-1 and SC1-4 (1/2-in. facings). This is expected since the flexural rigidities of the beams with thick facings are less affected by the core rigidity than those with thin facings and, therefore, are not as dependent on the core behavior. Not seen in the load-deflection curves for the beams with concrete cores were any discontinuities at initial cracking. A small drop in load was

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observed when cracking occurred along with a relatively smaller reduction in mid-span deflection. As load continued to be applied, the deflection increased such that essentially no difference or discontinuity between the cracked and uncracked deflections could be seen when the response was plotted. This behavior was visually observed but no accurate measurements could be made since the magnitudes of the deflections were so small.

The measured responses for the beams with foam cores were essentially linear to failure. Some nonlinearity, however, was observed in approximately the last 10 percent of the load range for beams SC2-3, SC2-6, SC3-2, SC3-3, and SC3-6, and was probably the result of excess shear deformations in the cores. That is, the cores allowed large relative moments between the two facings thus forcing the facings to act like two independent beams.

Three theoretical curves were plotted for each beam -- two corresponding to Figs. 4(a), 5(a), and 6(a) and one to Figs. 4(b), 5(b) and 6(b) -- using sandwich beam theory. One of the theoretical curves in Figs. 4(a), 5(a), and 6(a) represents the predicted load-point deflection considering both flexure and shear. The second curve corresponding to Figs. 4(a), 5(a), and 6(a) is the load-point deflection predicted by considering only flexure. Both of these theoretical curves were plotted to an applied load which either the tensile facing was predicted to rupture or the core was predicted to fail in shear. The theoretical curves shown in Figs. 4(b), 5(b), and 6(b) are the predicted mid-span deflection relative to the load-point deflections, that is, they consider only flexure. These curves were plotted to an applied load where a rupture of the tensile PC facing was predicted.

Two items should be noted concerning the theoretical load-deflection curves for the beams with concrete cores (Fig. 4). First, the theoretically predicted shear deflections are so small compared to the flexural deflections that the two theoretical curves plotted in Fig. 4(a) coincide, thus only the one curve.

The second item concerns the nonlinear portion of the theoretical curves in Fig. 4. Since conventional sandwich beam theory is a linear analysis, an incremental procedure was used to predict the nonlinear portion of the load-deflection curves. The nonlinearity was caused by a reduction in flexural rigidity after cracking in the concrete core and was accounted for by considering the beams as doubly-reinforced portland cement concrete sections. The polymer concrete facings were assumed to act as compressive and tensile reinforcement for the concrete cores. The ACI (5) method for determining an effective moment of inertia of a cracked section was incorporated into the sandwich beam theories. The following ACI equation was used to determine an effective moment of inertia after cracking was predicted:

$$I_e = I_g (M_{cr}/M_a)^3 + I_{cr} [1 - (M_{cr}/M_a)^3] \quad (1)$$

where I_e is the effective moment of inertia after cracking, I_g is the gross moment of inertia of the uncracked section, I_{cr} is the moment of inertia of the cracked section, M_{cr} is the cracking moment, and M_a is the maximum moment in the member. The effective moments of inertia then were used to determine effective flexural rigidities. Theoretical deflections after cracking were determined using the conventional sandwich beam procedures except that the initial flexural rigidities were replaced by the determined effective rigidities.

The theoretical load-deflection curves for the flexural zone predicted the observed deflections quite well (Figs. 4(b), 5(b), and 6(b)). This indicates that sandwich beam theory accurately predicts flexural behavior. Therefore, in Figs. 4(a), 5(a), and 6(a) the difference between the observed deflections of the load points and those predicted by theory considering only flexure must be due to shear.

The shear contribution in the beams with concrete cores (Fig. 4(a)) can be assumed negligible since the theoretical flexural deflections accurately predict the observed load point deflections; actually, the predicted deflections are overestimated for three of the six beams without even considering shear. Also, the shear deflections predicted by theory were so small compared to the flexural deflections that the difference could not be plotted, thus the two theoretical curves in Fig. 4(a) coincide as previously mentioned. For example, the theoretical deflection caused by shear in beam SC1-2 (Fig. 4) at a total load of 1500 lbs is 0.00032 in. which is less than one percent of the corresponding flexural deflection.

Shear is much more important in the beams with foam cores. The predicted load-point deflections considering only flexure account for approximately 15 percent of the total deflections of the beams with polystyrene cores and approximately 10 percent for beams with polyurethane cores. The additional deflection caused by shear is shown by arrows in Figs. 5(a), and 6(a). The importance of shear in the beams with foam cores was predicted by theory. The total load-point deflections were accurately predicted when both flexure and shear were considered as shown in Figs 5(a) and 6(a).

Strain and Stress Distributions

Flexurally produced strain and stress distributions for beams SC1-4 (1/2-in. facings with 3-in. concrete cores) and SC2-3 (1-in. facings with 2-in. polystyrene core) are illustrated in Figs. 7 and 8. The plotted distributions represent the strain

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gradients at loads corresponding to 40 percent of ultimate, 70 percent of ultimate, and impending failure. The stress distributions were found by multiplying the strain readings by the appropriate moduli of elasticity.

The strain distributions in the portland cement concrete include the tensile strains up to cracking; only the compressive strains are included after cracking. Cracks in the concrete cores were observed at a tensile strain of approximately 0.0002 in./in. The strain distributions of the beams with concrete cores were approximately linear and passed through the geometric centroids of the sections prior to cracking. However, the neutral axes were forced upward, towards the compressive facing, once flexural cracking occurred in the cores to compensate for the loss of tensile capacity in the portland cement concrete. The neutral axes moved upward in all of the beams with concrete cores until the tensile facing ruptured, but greater movement was observed in the beams with thinner facings. The strains in the PC facings at rupture ranged from 0.00057 to 0.00061 in./in.

The strain distributions through the expanded polystyrene foam and the rigid polyurethane foam were assumed to be linear and in line with the measured strain distributions of the facings. The assumption was made necessary since no feasible way existed to read directly the strains from the low modulus foam materials. With the above assumption, the strain and stress distributions of the beams with foam cores were found to be approximately linear and pass through the geometric centroids of the sections throughout loading. The neutral axes did not move as was the case with the beams with concrete cores.

Shear Stresses

A comparison between observed and theoretical shear stresses caused by shear forces corresponding to loads of 40 percent of the ultimate loads is presented in Table 4. The observed stresses were determined by multiplying strain values by the shear moduli of the cores. The theoretical stresses are average stresses through the thickness of the cores as predicted by sandwich theory rather than maximum values. An average was used since the observed strains were average strain measurements which yield average stresses. A positive percent difference in Table 4 indicates that the observed shear stress was greater than the predicted value.

Modes of Failure

The observed modes of failure were classified as either flexural or shear. Flexural failure was defined as the rupture of the tensile facing at maximum load and was the most prevalent. All of the beams with concrete cores were predicted, and likewise

observed, to fail in flexure; however, flexural cracking occurred in the cores prior to rupture of the tensile facings. The first cracks in the cores were observed to be distributed fairly evenly between the load points (zone of constant moment). Continued cracking was observed until the tensile facings ruptured at the location of a core crack somewhere between the load points. Figure 9 illustrates the ultimate cracking pattern for beam SC1-4 (1/2-in. facings and 3 in. core).

Beams SC2-1, SC2-2, SC2-4, and SC2-5 with polystyrene cores, and SC3-4 with a polyurethane core also were observed to fail in flexure with SC2-1, SC2-4, SC2-5 so predicted. The tensile facings in these beams ruptured at a point between the two load points. Regardless of the core material, when the tensile facing ruptured it was a sudden and brittle failure; there was no yielding or cracking observed in the PC at any point.

Shear failure was defined by large relative movements between the two facings and was observed only in the beams with foam cores. The foam cores tore apart and no longer resisted movement between the facings. Beams SC2-3, SC2-6, SC3-2, SC3-3, SC3-5, and SC3-6 were observed to fail in shear. Shear failure was predicted in SC2-2, SC2-3, SC2-6, and all of the beams with polyurethane cores.

Beam SC3-1 failed in shear and flexure almost simultaneously. The core began to allow significant relative movement between the facings which indicated the beginning of a shear type failure. However, before any further shear action was observed the tensile facing ruptured at mid-span.

Ultimate Strengths

Observed and theoretical ultimate moments and ultimate shears are listed in Table 5. The observed ultimate moments are listed for only those beams that failed in flexure as are the observed ultimate shears for those that failed in shear. The number in parentheses next to the observed and theoretical ultimate moments are the corresponding shear forces, in pounds, producing the moments. A positive percent difference indicates an observed ultimate strength greater than the theoretical prediction. Conversely, a negative percent difference indicates a lower observed ultimate strength. Observed values for both moment and shear are given for beam SC3-1 since it failed in flexure and shear simultaneously. Also, percent differences for both ultimate moment and ultimate shear are given for SC3-1, the first corresponding to moment.

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CONCLUSIONS

The overall results of this investigation suggest that the application of PC as a facing material for composite sandwich members holds high potential. Relatively small volumes of PC can be used which fully utilize the positive characteristics of the material. The following specific conclusions can be made for the composite sandwich beams investigated in this study and are based on the results reported in this paper.

Beams with Concrete Cores

Extensive flexural cracking occurred in the concrete cores prior to the total collapse of the beam. Due to the cracking, the ACI method for determining an effective moment of inertia was used in conjunction with conventional sandwich beam theory to predict accurately the behavior of sandwich beams with concrete cores.

The flexurally-produced strain distribution in beams with concrete cores were linear to failure. The neutral axes of the beams initially coincided with the geometric centroid of the entire beams but as cracking occurred they moved upward (towards the compression face) in the sections. The movements were less for beams with thick facings.

Beams With Polystyrene Cores

Conventional sandwich beam theory adequately predicted the behavior of the sandwich beams with polystyrene cores. Shear must be considered when the core material is of low modulus and strength, such as the polystyrene. The deflections of the beams with polystyrene cores were dominated by shear; approximately 85 percent of the total deflections were caused by shear throughout loading.

Flexural failures as well as shear failures were predicted and observed in the beams with polystyrene cores. The beams with the 1-in. thick facings failed in shear. The thicker facings increased the moment capacity of the beams to a level that surpassed the shear capacity.

The neutral axes of the beams with polystyrene foam cores were observed to coincide with the geometric centroids of the entire beam. No movements of the neutral axes were observed at any point during loading.

Beams With Polyurethane Cores

Basically, the same conclusions can be made for the beams with polyurethane cores as were made for the beams with polystyrene cores. However, shear had more of an effect with the polyurethane. Approximately 90 percent of the total deflections of the beams with polyurethane cores were caused by shear versus 85 percent with the polystyrene cores. Also, shear failures were more common with the polyurethane cores; only the beams with 1/2-in. facings failed in flexure.

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