

Bridge Repair and Rehabilitation



Compilation 29

American Concrete Institute

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Preface

ACI Compilations combine material previously published in Institute periodicals to provide compact and ready reference on specific topics. The Material in a compilation does not necessarily represent the opinion of an ACI technical committee — only the opinions of the individual authors. However, the information presented here is considered to be a valuable resource for readers interested in the subject.

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Concrete Bridge Construction and Maintenance

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Rehabilitation

On The Cover: Full depth precast concrete bridge deck panels have many advantages for replacing deteriorated deck slabs. They offer speed of construction, low cost, and the ability to build in stages to minimize traffic interruption (see article starting on p. 3).

Construction Procedures for Rapid Replacement of Bridge Decks

by Mohsen A. Issa, Alfred A. Yousif, and Mahmoud A. Issa

Full depth precast or precast prestressed concrete bridge deck panels can be used to replace deteriorated decks.¹⁻¹⁵

These precast elements can be supported on steel rolled beams or plate girders (Fig. 1) as well as precast prestressed or reinforced concrete girders.

This system can be used advantageously to reduce construction time, hence, lowering the cost imposed by the rehabilitation process. Staged construction can be implemented to maintain traffic flow in both directions. Half of the bridge deck can be replaced while the other half remains operational as demonstrated in Fig. 1. Replacement of the deck system using conventional cast-in-place concrete prolongs and interferes extensively with the flow of traffic.

This system has several significant design and construction components:

- 1) Transverse joints between the precast or precast prestressed concrete panels, and the type of material within the joints.
- 2) Connections between the deck and its supporting system, whether steel or concrete, and type of grout.
- 3) Post-tensioning in the longitudinal direction.
- 4) Haunches between the top flanges of the girders and the slab deck.
- 5) Overlay and waterproofing membrane system.

The system

The configuration of the joints between adjacent precast panels should be designed to allow an easier flow of grout, maintaining 1/4 to 1/2 in. openings to allow for any panel misalignment and dimensional growth (Fig. 1).

The grout material must have early high strength to allow for immediate

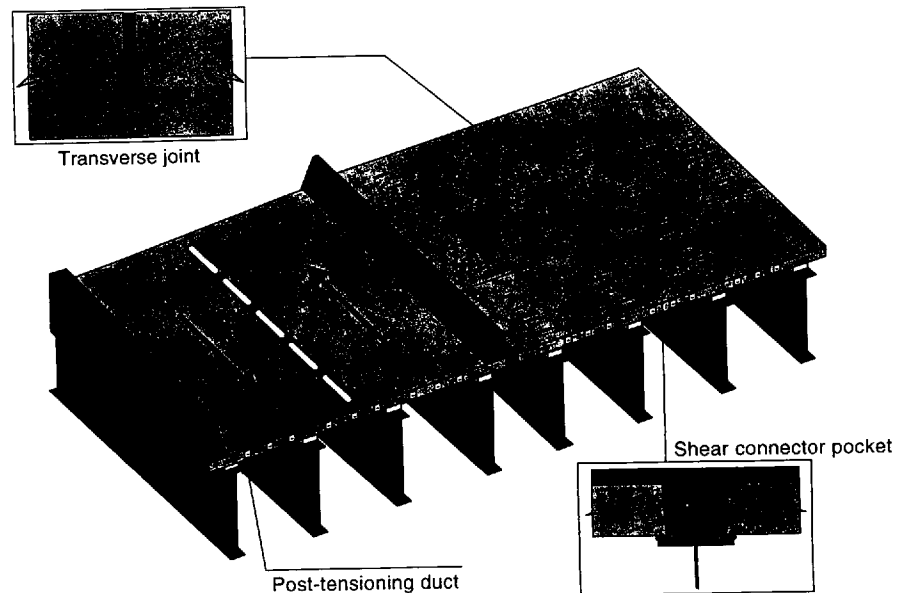


Fig. 1 — Layout of the bridge deck reconstruction system.

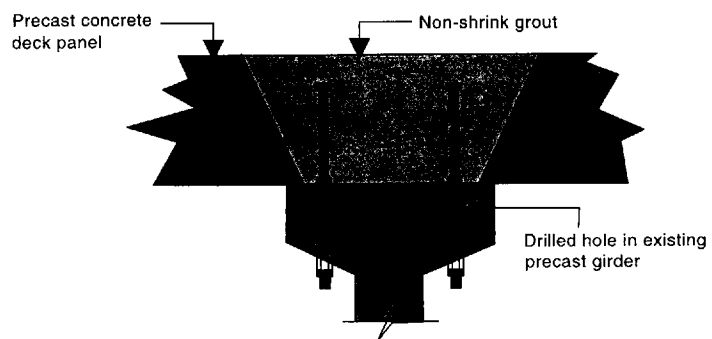


Fig. 2 — Connection between precast panels and precast girders.

post-tensioning, or to permit traffic to resume as usual.

The best method for achieving full composite action between the slab deck and steel stringers (Fig. 1), prestressed or reinforced concrete girders (Fig. 2) is to provide shear connection pockets.

Shear studs or high strength bolts can be used to achieve this objective. However, extensive research has indicated that the shear stud connection is the best solution (Fig. 1).

The number of shear studs depends on the structural design requirements

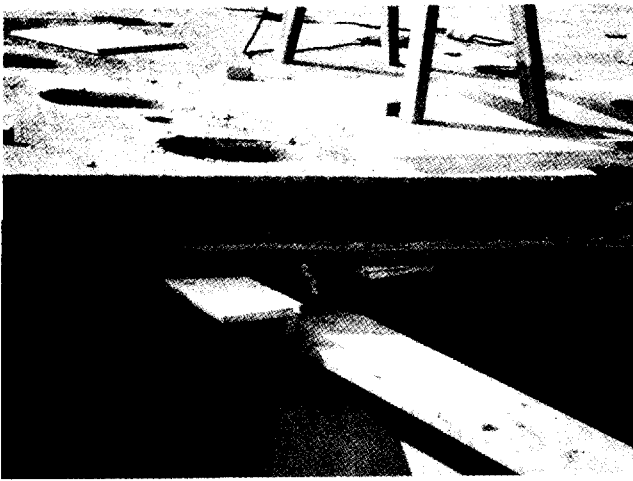


Fig. 3 — Haunches to account for cambering in girders.

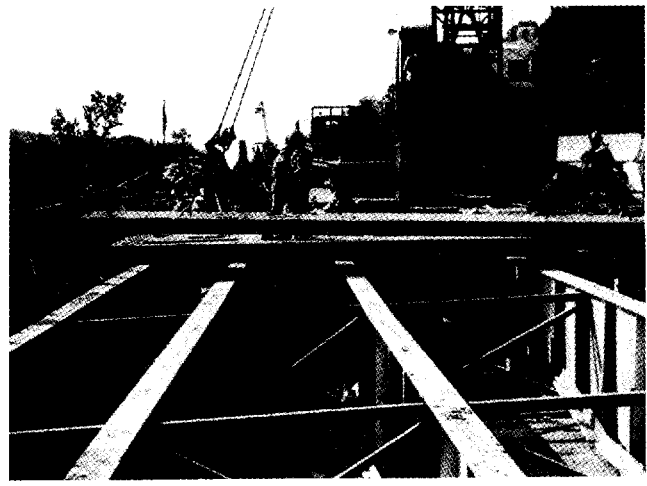


Fig. 4 — Placement of panels on structural steel.

for every specific application. The spacing between shear connection pockets is normally about 2 ft (0.6 m); however, more research is needed to verify this spacing factor.

The configuration of the shear pocket either can be rectangular or beveled. However, the beveled opening is preferred due to a reduction in stress concentration at the corners.

The material used to grout the shear connection pockets may be non-shrink grout with a high early strength to allow for traffic flow in one or two hours.

Post-tensioning the precast panels longitudinally is essential to eliminate the tensile stresses in the transverse joints, i.e., to keep the joint in compression (Fig. 1). The structural behavior of the bridge deck system is manifested in the performance of the joints. Post-tensioning tightens the joints and prevents any leakage through the joints.

Previous research as well as preliminary analysis has indicated that for simply supported spans, at least 150 psi (1.03 MPa) is needed to keep the joints in compression, while for continuous spans, approximately 300 psi (2.07 MPa) is required.¹⁵

A minimum of 1 in. (25 mm) haunch is necessary so that the existing top flange of the steel stringers would not be in contact with the panels. A fully composite system is achieved effectively if these haunches are incorporated into the design and construction of the bridge deck system during rehabilitation (Fig. 3).

An overlay is essential in order to provide a smooth ride. Following the bridge deck replacement process, the top surface of the deck and grouted openings are rough. As a result, an

overlay is necessary to eliminate the rough surface and protect it from the wear and tear of traffic.

Types of overlays include: bituminous asphalt wearing surface, latex modified concrete, silica fume concrete (microsilica), lowa low dense slump, and epoxy overlays. It is normally good practice to provide a waterproofing membrane system prior to the application of the bituminous asphalt wearing surface. This procedure helps reduce the possibility of any leakage through the deck.

The field investigations conducted throughout the past several months included discussions with bridge engineers from various departments of transportation.

General descriptions and observations of the rehabilitation techniques encountered during the investigations were as follows:

- Most bridges are rehabilitated based on two-phase construction, where only half of the bridge width is considered at a time;
- A longitudinal joint is necessary to achieve a crown between the two phases of construction (Fig. 1);
- The surveyors observe the conditions prevalent to prepare what is necessary for the work to follow;
- During normal traffic flow, carpenters prepare the formwork while work continues on the structural steel, bearings, and parapets;
- Normally, weekend closures are necessary because the work commences at 7 p.m. on Friday night and concludes by Monday 6 a.m.;
- For the two end closures, high early strength concrete would be placed on Sunday night. By Monday morning,

the cast-in-place concrete reaches a minimum strength f_c' of 1000 psi (7.0 MPa), where the bridge will be open to traffic. In this case, the formwork would be designed to hold the concrete and traffic load;

- Curing time normally slows down the job;
- Longitudinal joints are used between precast panels where the installation of full width panels is not possible, such as at sections of the bridge where an exit is required. The weight of the crane is a big factor to achieve full width panels. As a result, two of the panels are installed with a longitudinal joint in between;
- Due to the presence of high tension wires on one side of a bridge in Seymour, Connecticut, the crane could not be used on that side to lift and erect the panels. As a result, a technique was adopted where the panels were slid on roller skates horizontally in order to properly place the slab units.

Construction procedures

The construction procedures adopted by contractors on several rehabilitation projects were very effective in terms of scheduling, materials, and actual work techniques. Moreover, the most important factor observed in construction was the effectiveness of time scheduling and how the work was accomplished within the time limits imposed on the workers to maintain traffic on the specified date.

The field investigation included the observation of several bridges nationwide. Work on the Seymour Bridge in Connecticut was reported to include 22 weekend closures of the bridge to ac-

comply with the replacement of the deck panels within the two-day periods.

The construction procedure is described in more detail by the following major steps in the replacement process: **Shut down and switch traffic:** traffic is switched to alternate routes to accommodate motorists.

Saw cutting: existing slabs are saw cut approximately 0.5 in. (13 mm) from the top flanges of the beams and close to the shear connectors.

Demolition: the deck is then ripped out while demolition of the existing deck commences.

Formwork: as the panels are removed, the carpenters work on the forms except for the haunch pockets (zero deflection).

Preparation for panel placement: top flanges of the beams are cleaned prior to the placement of the precast panels.

Placement of precast panels: precast panels shipped from the precast yard are lifted by cranes and set in order on the supporting system; each panel is properly aligned in accordance with the deck grade (Fig. 4).

Leveling: as each panel is placed, it is adjusted with the use of leveling screws to perfectly align the slab units adjacent to one another. The leveling screws are turned as a result of any improper alignment. This process is extremely successful as it is performed without any difficulties.

Haunches: haunches (Fig. 3) are then tied. Haunches of 1 in. (25 mm) minimum are provided to account for any cambering in the girders.

Post-tensioning ducts: one set of post-tensioning tendons are run through the sheath ducts from the live end to the dead end. The number of post-tensioning lines (number of sheath ducts) is equal to the number of lines of shear pockets less one. For example, one sheath duct runs along the centerline between adjacent lines of shear pockets. At the transverse joints between adjacent precast panels (female to female type of joint), the sheath duct at the junction is normally taped with duct tape prior to grouting.

Grout shear keys: after tightly securing the ducts, the transverse joints between precast panels are grouted using polymer grout (Fig. 5). Polymer is used in order to achieve a strength of 7000 psi (48 MPa) in one hour or 5000 psi (34 MPa) in 45 minutes. The setting speed is important in preparing the deck for post-tensioning. As the mixture is poured, a finger vibrator is used to fill the joints more efficiently.

Shear studs: in the meantime, the crew prepares the shear pockets for placing the shear studs. Headed 3/4 or 7/8 in. (19 or 22 mm) studs are welded to the top flanges of the steel. These studs are tested and inspected on the site for any defects.

Closure placement: mild steel reinforcement is placed in the closure placement at both ends of the span where cast-in-place concrete is required.

Longitudinal joints: mild steel reinforcement is provided in the longitudinal joints as shown in Fig. 5.

Post-tensioning: one hour after the grouting of the shear keys, the slab units are post-tensioned longitudinally to secure the tightness in the joints.

Clean-up: the spans are then cleared of any debris accumulated in the joints or pockets. The lifting inserts or devices used to carry the slab units are cut off to prepare the deck for post-tensioning and grouting.

Grout sheath ducts: after post-tensioning, the sheath ducts are grouted. A mechanical gage is normally used to limit the grout pumping gage pressure to a maximum of 150 psi (1.03 MPa) during the grouting operation.

Grout pockets and haunches: the shear connector pockets and girder haunches are then grouted with a non-shrink grout. This grout should have a one hour compressive strength of 500 psi (3.4 MPa).

Concrete cast in place: high early strength concrete is then placed in the closure placement.

Check concrete strength: the cast-in-place concrete is then checked for a minimum strength of 1000 psi (6.9 MPa). This process is necessary to ensure safety of the traffic.

Weekend closures are the best possible solution for completing the work without major interruptions in the flow of traffic. However, the point that should be stressed is the scheduling aspect of these projects, because all of the work proceeded with extreme efficiency and responsibility.

Construction sequencing

Precast deck panels can also be installed on precast concrete girders using the procedure outlined in the section describing installation on steel supporting systems. The procedure would include a connection that is presented in Fig. 2.

A recent nationwide investigation did not reveal any applications where



Fig. 5 — Grouted transverse joints and layout of longitudinal joint.

precast concrete girders were used as supporting systems for full depth precast panels. The only application where precast panels were installed on a concrete supporting system was observed at the Dublin 0161 Bridge in Ohio. The existing deck supporting system for this bridge was a concrete arch with crossbeams, and connected to the deck by means of tie downs. This tie down system attributes towards the non-composite nature of the system. This system is not typical of the proposed system; however, it is introduced for the interest of observing the rehabilitation process utilizing precast panels on concrete supporting systems.

The general sequence of construction for the Dublin Bridge was divided into two phases as outlined below:

Phase I

- 1) Close the bridge.
- 2) Removal of the asphalt concrete wearing surface from the concrete deck.
- 3) Cut and remove the sidewalk and railing on the south side.
- 4) Cut and remove the deck in panels between the arch frame members.
- 5) Repair surfaces of frame members as required.
- 6) Install leveling pads and elastomeric bearings.
- 7) Install longitudinal stress panels in groups between expansion joints. The full depth precast panels consisted of panel lengths that varied from about 9.5 ft (2.9 m) to 12 ft (3.7 m), panel width of 28 ft (8.5), along with a varying depth. Set each succeeding panel so as to compress the gaskets at the duct interfaces. Epoxy mortar material was used for the joints between the adjacent precast panels.
- 8) Install longitudinal strands in the ducts and post-tension in the long direction for the design prestress level.

9) Install the anchor dowels and tie downs in the panels. Dowel holes in the existing structure.

10) Proceed to the next panel group.

11) Construction loads equivalent to the design service live load of the finished structure may be placed on each completed group of panels (longitudinally stressed), except that no loads will be allowed in a 3 ft (0.9 m) wide longitudinal strip along either edge of the group of panels.

Phase II

12) Repeat steps 1 through 11 for the north side of the structure. After each group of panels is joined and stressed longitudinally, prepare the longitudinal joint between the adjacent group of panels on the north and south sides. Install the transverse strands and stress and grout them.

13) Install the permanent expansion joint armor angles, and the compression seals.

14) Open the bridge to full traffic.

Conclusions

Durability and reduced need for maintenance, ease and speed of construction, along with maintaining traffic without interruption of traffic flow are advantages in using precast or precast prestressed concrete deck panels in bridge deck rehabilitation. Full depth precast or precast prestressed concrete deck panels give the contractor the opportunity to prefabricate all the required slab units prior to demolition of the old deck, and also give the owner the chance to widen the deck to meet current safety requirements in an efficient manner. By adopting proper construction techniques, it is possible to maintain full daytime traffic flow during the rehabilitation process. A half width of the bridge can be closed during the night and a specific portion of the old deck is removed and replaced the same night to be ready for traffic the next day. Alternatively, weekend closures also are very effective as demonstrated in the description of the Seymour project.

Acknowledgments

This study was funded by a contract awarded to the University of Illinois at Chicago by the Illinois Department of Transportation and the finan-

cial support is gratefully acknowledged. Appreciation of the authors is also extended to Ralph Anderson, Iraj Kaspar and Salah Khayyat, bridge engineers, Illinois Department of Transportation, for support throughout the project; Chien H. Wu, Chairman, Department of Civil and Materials Engineering, University of Illinois, Chicago, for financial support for the students working on the project; the structural and maintenance engineers of the Connecticut Department of Transportation, Ohio Department of Transportation, and all the other departments of transportation and authorities for expending valuable time in assisting in the research and investigation; to Michael Culmo, transportation supervising engineer, Connecticut Department of Transportation, for assistance during the visit to the Seymour Bridge construction project; and to Dave Poling, Ohio Department of Transportation, for assistance on the Dublin Bridge in Columbus, Ohio.

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Selected for reader interest by the editors.



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Water-jetting helps new concrete bond to old

Improving Concrete Bond in Repaired Bridge Decks

by Johan Silfwerbrand

Repairing a concrete bridge deck should result in a composite concrete deck whose strength is at least as high as the original monolithic deck. The strength of the composite deck depends on the bond between the old and the new concrete, and consequently, knowing about the factors affecting the bond is important (Table 1).

Repairing a concrete bridge deck consists of the following phases:

- Removal of unsound concrete
- Cleaning of exposed concrete surface and reinforcement
- Replacement of corroded reinforcing steel
- Replacement of removed concrete
- Curing

ACI Committee 546¹ emphasizes that chipping tools must be selected that will not damage the surrounding concrete.

The use of chipping hammers was once the most widespread method of concrete removal if the deterioration was deep (roughly 15 mm [$\frac{1}{2}$ in.] or more).² In recent years, the introduction of the water-jet technique (also called hydrodemolition, hydrojetting or hydroblasting) has

changed the prevalent method of concrete removal. A thorough discussion of Swedish experiences with the water-jet technique is given by Ingvarsson and Eriksson.² Besides a high efficiency, several important advantages add to the success of the method: water-jetting leaves a rough and clean surface, no microcracks are introduced into the remaining concrete, the reinforcement is undamaged and cleaned from rust, and only deteriorated concrete is removed, leaving the sound concrete intact.

The "Swedish Regulations for Concrete Structures"³ does not permit shear transfer at interfaces of composite concrete structures unless the reinforcement or the compressive forces cross the interface. The composite member may be formed of a precast part and a cast-in-place part or two cast-in-place parts placed at different times. Similarly, in the absence of tie reinforcement or dowels, European and American concrete codes prescribe comparatively low values for the permissible shear stress at the interfaces (0.22 to 0.62 MPa [32 to 90 psi] dependent on characteristic compressive concrete strength⁴ and 0.55 MPa [80 psi]⁵ respectively). Consequently, reinforcement crossing the interface is needed as soon

Table 1 — Factors that may influence concrete bond

Old concrete	Concrete strength Aggregate/gradation Salt content Age
Concrete removal	Technique Depth
The interface	Microcracks Roughness Surface preparation Prewetting Other treatment (bonding agents, etc.) Surface moisture at time of place overlay
Concrete overlay	w/c Cement content Aggregate/gradation Cement type Fibre content Admixtures Slump Thickness Compaction method Compaction time Curing Time since placement
Weather conditions	Temperature and humidity after concrete removal

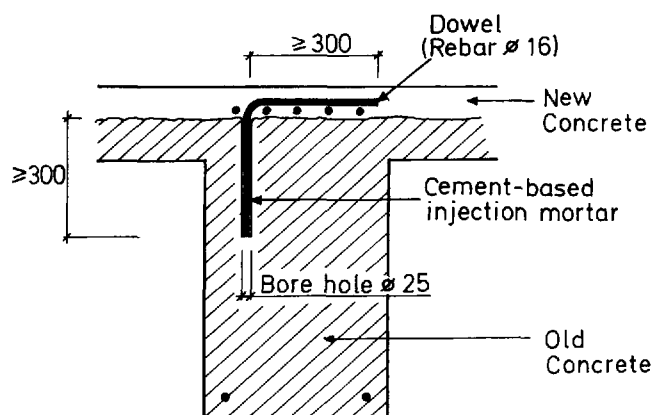


Fig. 1 — Dowel arrangement according to the practice of the Swedish National Road Administration.

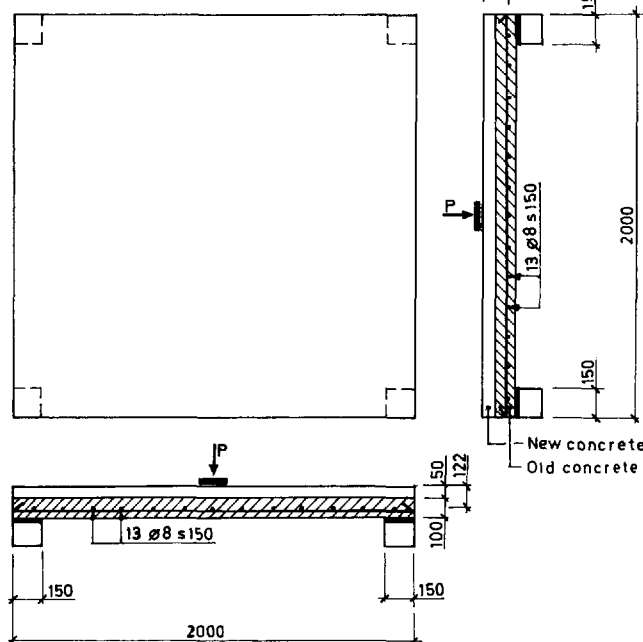
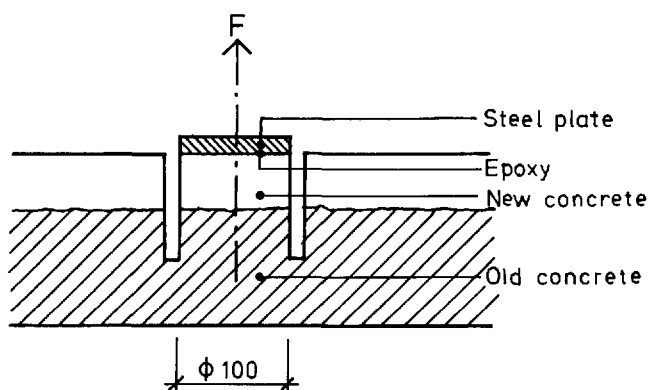


Fig. 2 — (above) Test slabs.

Fig. 3 — (left) Pull-off test diagram (measurements in mm, 1 mm = 0.394 in.).

as the applied design shear stress exceeds these small values.

The shear capacity in composite concrete bridge decks is provided by dowels connecting the two concrete layers (Fig. 1). However, to provide the entire bridge deck with dowels would be very expensive. If the shear capacity can be ensured without dowels for well-defined conditions, then repairing bridge decks will be more economical.

Previous research

Bond tests on composite concrete structures with water-jetted interfaces are scarcely dealt with in testing literature. Some bond tests on cores from an overlaid precast bridge deck are reported by Tayabji,⁶ who obtained shear stresses of about 3 MPa (440 psi).

New tests

Pull-off tests were conducted to find out how the water-jet technique affected the bond between old and new concrete in comparison with other methods of concrete removal. Traffic, weather, deicing salts or other environmental factors

may influence the bond strength, but they will hardly change the rank of methods of concrete removal. For simplicity, testing was limited to pull-off tests on concrete slabs stored indoors without exposure to traffic or environmental factors.

Test slabs

The test group consisted of five composite slabs and two monolithic slabs. The composite slab overlays were cast seven months after the concrete base layer. One of the monolithic slabs was cast from concrete used for the base layer, the other was cast from concrete used for the overlay.

The slabs were 2 m (6.6 ft) square. At testing, the total thickness of all slabs was 150 mm (5.9 in.). The ratio between the overlay thickness and the total thickness was 1/3 for all of the composite slabs (Fig. 2).

The concrete used for the base layer of the composite slabs had a water cement ratio of 0.56, a cement content of 275 kg/m³ (464 lb/yd³), and a maximum aggregate size of 32 mm (1 1/4 in.). At the time

of the pull-off testing, the cube strength was 52 MPa (7540 psi). Corresponding figures for the concrete used for the overlays are 0.51, 420 kg/m³ (708 lb/yd³), 12 mm (1/2 in.), and 56 MPa (8120 psi).

Surface preparation and curing

Initially, the base layer thicknesses varied between the different slabs. After concrete removal or sandblasting all of the composite slabs had equal base layer thicknesses.

Approximately 20 mm (0.8 in.) of concrete was removed from four of the slabs with base layer thicknesses of 120 mm (4.7 in.) using water-jetting or hand-held pneumatic hammers. Each method was used on two of the slabs. These slabs were not sandblasted. The fifth slab was originally cast 100 mm (3.9 in.) thick. It was sandblasted prior to placement of the overlay.

Loose particles and dust were thoroughly removed from all surfaces using high pressure air and vacuum cleaners.

Following the recommendations of the Swedish National Road Ad-

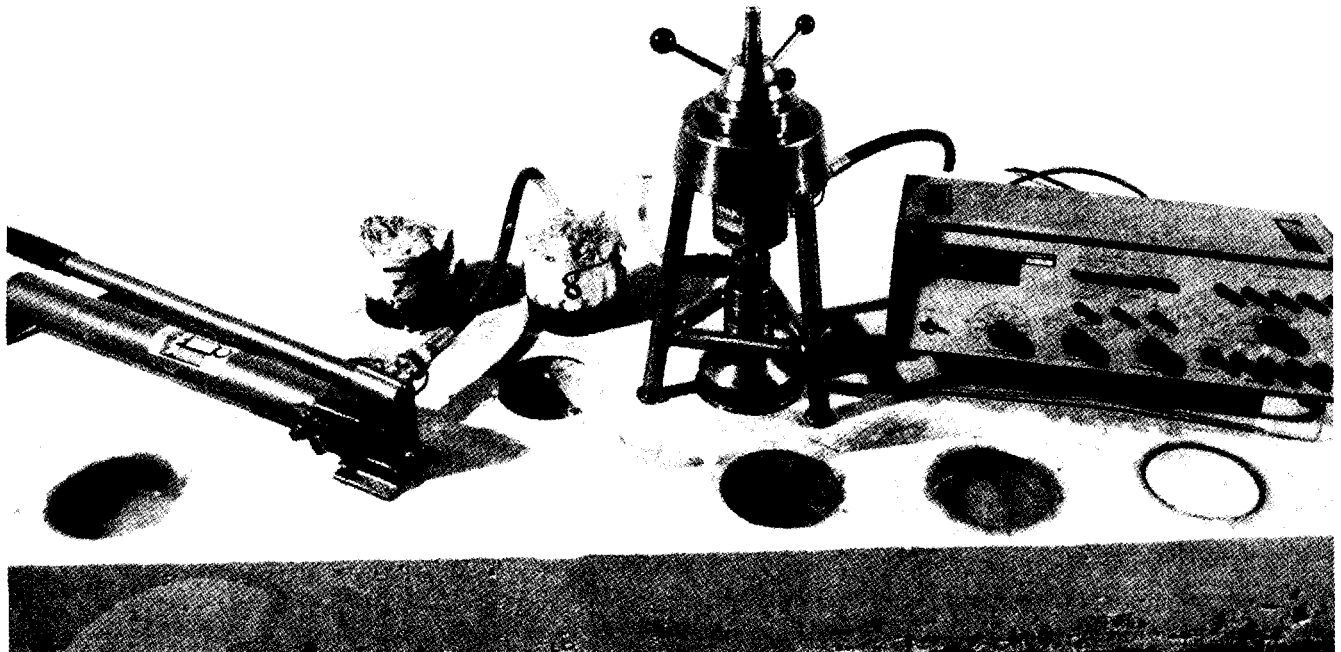


Fig. 4 — The pull-off test equipment.

ministration,⁷ all of the composite slabs were wetted and kept moist for 48 hours before casting the overlay. They were all superficially dry when the overlay was placed. No bonding agent or other treatment was used.

After both the base layer casting and the overlay casting the slabs were kept moist for seven days using wet burlap followed by indoor storing at a temperature of about 20 C (68 F) and a relative humidity of approximately 30 percent. A proper curing of the overlay reduces the magnitude and the rate of the differential shrinkage, delaying the development of the maximum tensile stress until the developed available strength is higher.⁸

Pull-off tests

In all, 47 pull-off tests were conducted on the five composite slabs and the two monolithic slabs. Cores with a diameter of 100 mm (3.9 in.) were drilled from the top surface down through the concrete overlay, through the interface, and about 50 mm (2 in.) into the base layer of the composite slabs. Cores were drilled

to an equal depth in the monolithic slabs. Steel plates were glued to the top surfaces of the cores and a pure axial tensile load was applied to them (Fig. 3 and 4). The pull-off tests were carried out twelve months after casting of the overlays.

Shrinkage-induced strains and deflections were measured in the intervening time. Prior to the pull-off tests, the slabs were supported at the corners and loaded with a single central load. No bond failures occurred during the shrinkage or loading phases. (These tests are reported elsewhere.⁸) The cores were drilled in undamaged regions at the edges of the slabs.

Test results

The results of the pull-off tests are shown in Table 2 and summarized in Fig. 5 and 6.

Theoretically, a failure will occur at the weakest section of the core. If an interface failure is obtained, the bond strength can be defined as the failure stress. If the failure occurs elsewhere, the result is more difficult to interpret: the strength of the interface may be equal to the fail-

ure stress, but also it may be higher. Consequently, a comparison between the pull-off tests with interface failures is most relevant. The problem is that the number of interface failures may be fairly small, making the conclusions inherently uncertain.

Compared to mechanical chipping, water-jetted interfaces were twice as strong. Also, a lower percentage of interface failures was obtained for the water-jetted slabs.

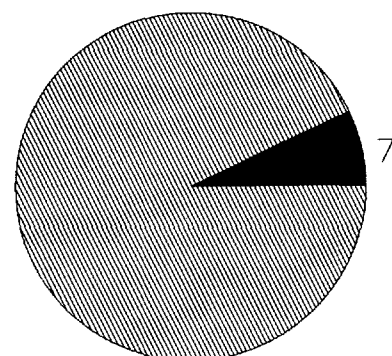
When comparing the slabs with water-jetted interfaces to the slab with a sandblasted interface, failure stresses above 2 MPa (290 psi) were frequently obtained for both cases (Table 2). Among the cores with interface failures, however, the water-jetted interface showed the highest average bond strength. The low percentage of interface failures also indicated the superiority of the water-jetted interface.

The differences in interface strengths between the three surface treatments were due to differences in surface roughness as well as differences in the amount of microcracks in the old concrete (Fig. 7

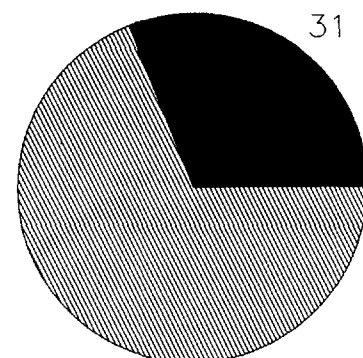
Table 2 — Laboratory pull-off tests

Slab No.	Interface treatment	Core No.	Failure stress (MPa)	Failure mode
1	Pneumatic hammer	11	1.18	Base ¹
		12	0.92	Base
		13	1.20	Base
		14	1.82	Base
		15	1.03	Base
		16	1.45	Base
		17	1.55	Base
		18	0.88	Interface ²
2	Pneumatic hammer	21	0.83	Interface
		22	1.03	Base
		23	0.98	Interface
		24	1.02	Base
		25	0.90	Interface
		26	0.71	Base
		27	0.97	Base
		28	1.13	Interface
3	Water-jet	31	1.88	Base
		32	1.53	Base
		33	2.28	Base
		34	1.84	Base
		35	1.64	Base
		37	1.82	Base
		38	1.78	Base
4	Water-jet	41	2.12	Base
		42	2.23	Interface
		43	2.04	Base
		44	2.47	Base
		45	2.11	Base
		46	2.20	Base
		47	2.01	Base
		48	1.77	Base
5	Sandblasting	51	2.93	Base
		52	2.35	Base
		53	2.69	Interface
		54	3.04	Base
		55	3.04	Base
		56	2.51	Base
		57	1.86	Interface
		58	0.64	Interface
6	Monolithic Cast of concrete used for the base layer	61	2.24	
		62	1.82	
		63	2.25	
		64	2.30	
7	Monolithic Cast of concrete used for the overlay	71	3.60	
		72	3.53	
		73	3.65	
		74	2.90	

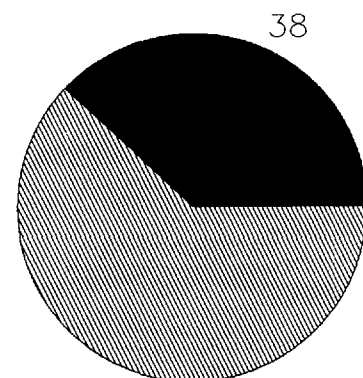
¹Failure mainly in the base layer
²Failure mainly an interface failure



Water-jet



Jack-hammer



Sandblasting

Fig. 5 — Percentage interface failures obtained through pull-off tests.

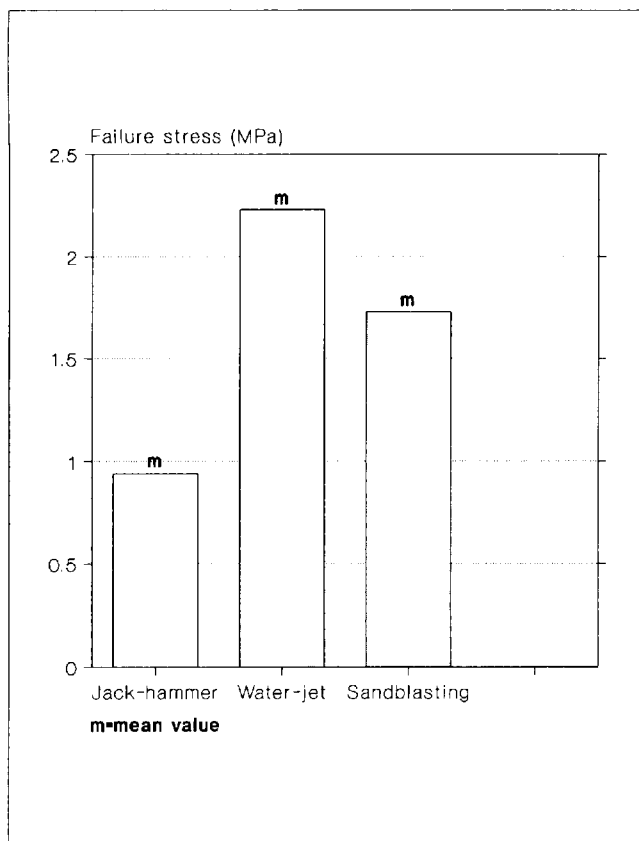


Fig. 6 — Comparison between pull-off tests resulting in interface failures on slabs 1-5.

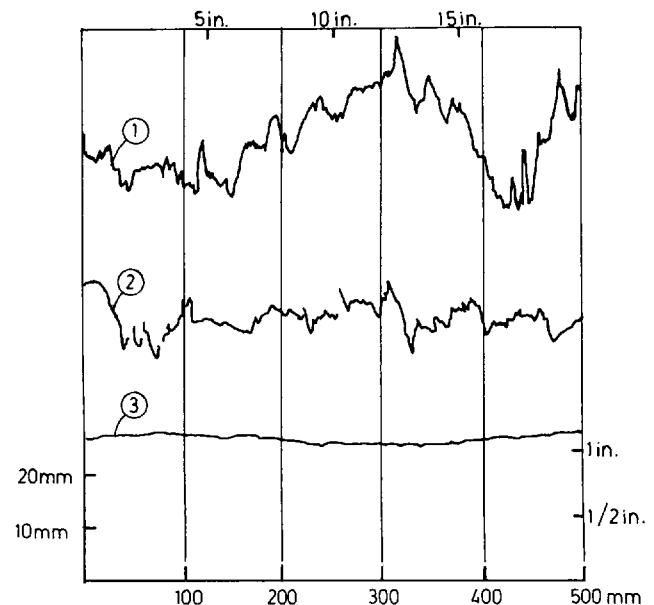


Fig. 7 — Comparison between contact surfaces treated by (1) water-jet, (2) pneumatic hammer, and (3) sandblasting. Note: the depth scale is exaggerated.

and 8). There was a pronounced difference in surface roughness due to water-jetting or sandblasting, but the difference in bond strength was less than expected. Neither of the methods introduced microcracks in the existing concrete. One may conclude that surface roughness does not have a major influence on the bond strength, but the relatively high percentage of interface failures for the slab with the comparatively smooth sandblasted interface indicates there may be a threshold value.

The main difference between water-jetting and mechanical chipping was the amount of microcracks introduced. The difference in bond strength showed that concrete removal must be done by a method minimizing microcracking.

The water-jet technique is superior to the mechanical chipping method because it results in higher failure stresses and lower percentages of interface failures.

Field test

Since the early 80s, concrete bridge

decks have been repaired on more than 300 bridges in Sweden. The water-jet technique has been used to remove deteriorated concrete on all of these bridge decks. Pull-off tests have been carried out on about 200 cores from 30 repaired bridge decks. The same equipment was used for both field tests and for the laboratory tests.

The repair procedures were similar to the laboratory tests, i.e., using conventional concrete (sometimes with a steel fiber content) without applying bonding agents prior to placing the new concrete overlay. Old concrete strength, removal depth, surface preparation, concrete overlay mixture, compaction, curing, and time at testing have, however, changed with time and from one bridge to another.

Test results

For pull-off tests carried out between 1984 and 1988, the average failure stress has increased since 1984 (Fig. 9). Some cores from 1984 and 1985 had very low values, de-

creasing the average. Dust and loose particles were found on the interface of some of those and air pockets because of poor compaction were found on others.

Using the water-jet technique for concrete removal leaves a clean surface if all loose concrete particles are removed. The Swedish National Road Administration⁷ recommends hosing the surface down with water at high pressure. The surface must be clean at the moment the overlay is placed. Furthermore, use of the water-jet technique leads to a very rough surface, and consequently compaction is particularly important in order to prevent the creation of air pockets in the surface depressions.

Since careful cleaning and proper compaction have become part of the construction practice, the average failure stress has exceeded 1.5 MPa (220 psi) and the percentage of interface failures has dropped below 20 percent. At present the failure usually occurs in the old concrete because all of the deteriorated concrete was not removed.

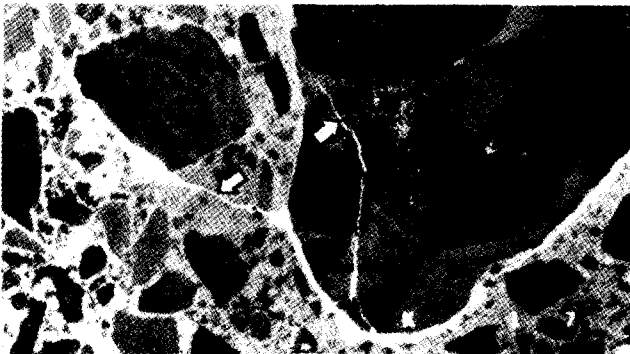
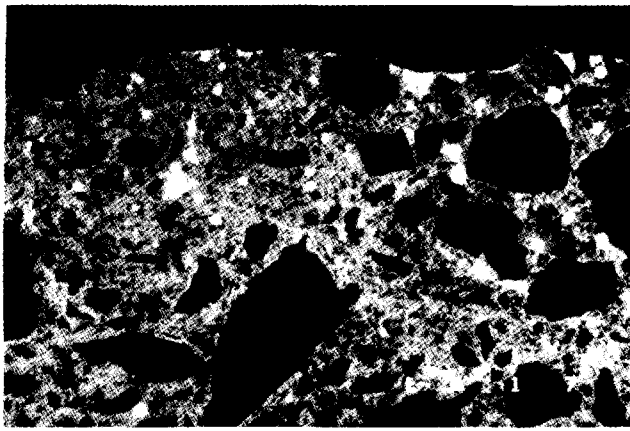


Fig. 8 — Differences in amount of microcracks in interfaces treated by (a) water-jet and (b) pneumatic hammers.

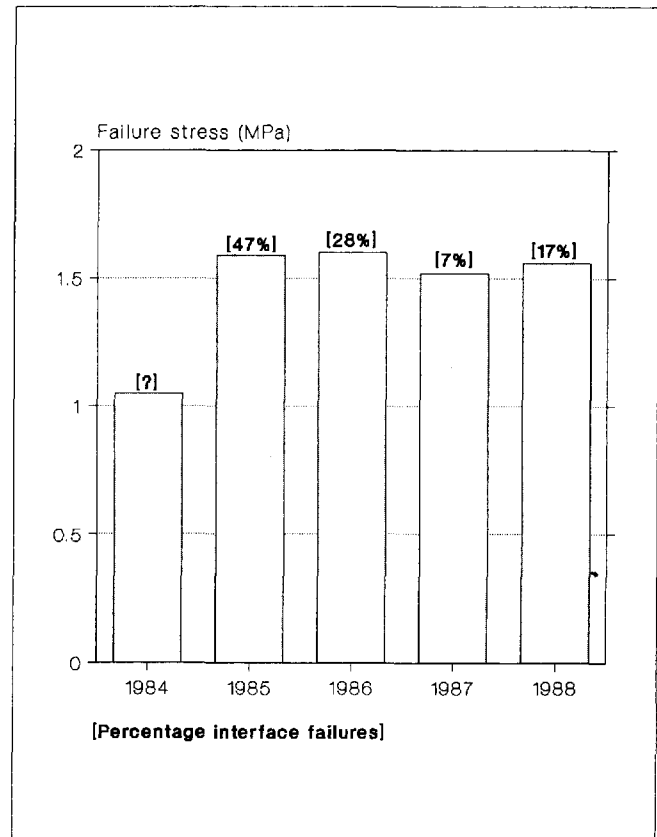


Fig. 9 — Annual average values of in-situ pull-off tests and corresponding annual average percentage interface failures.

Conclusions

Pull-off tests on composite concrete test slabs and on repaired concrete bridge decks show that a good bond between old and new concrete can be obtained without expensive connecting dowels if certain precautions are taken:

- Remove concrete without introducing microcracks into the remaining concrete. This can be done by proper water-jetting.
- Clean the contact surface from dust, grease and loose particles prior to placement of the overlay, making sure that the surface is still clean at the time of placing.
- Compact the new concrete overlay properly.
- Cure the overlay properly.

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Received and reviewed under Institute publication policies.

Acknowledgements

The tests described in this paper were part of a research project carried out at the Department of Structural Mechanics and Engineering at the Royal Institute of Technology in Stockholm in cooperation with the Swedish National Road Administration and the Stockholm City Streets and Traffic Administration. The Swedish construction company ABV carried out the water-jet removal using Atlas Copco's Conjet equipment. The author wishes to express his gratitude to his former adviser Professor Sven Sahlin, FACI, Royal Institute of Technology in Stockholm, for his valuable comments on this article.

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Rehabilitation of a Chicago span

Reinforced Concrete Deck Used for Bascule Bridge

By Byron T. Danley

In a first for the Chicago, Illinois, area, a bascule bridge recently underwent rehabilitation with the erection of a conventional reinforced concrete deck. Engineering design for the North Ashland Avenue span was provided by Teng & Associates, Inc. and will provide a lower cost, lower maintenance floor system than the concrete-filled grating or open steel grating found on most "Chicago-style" or trunion bascules.

The initiation of the new design began when the Chicago Department of Transportation (CDOT) performed a detailed bridge inspection of the North Ashland bridge over the North Branch of the Chicago River; highway agency officials immediately realized that closing the bridge to traffic was the only option. Although the double-leaf bascules' main trusses were in good condition, inspection of the bridge from a river barge revealed a severely deteriorated floor system.

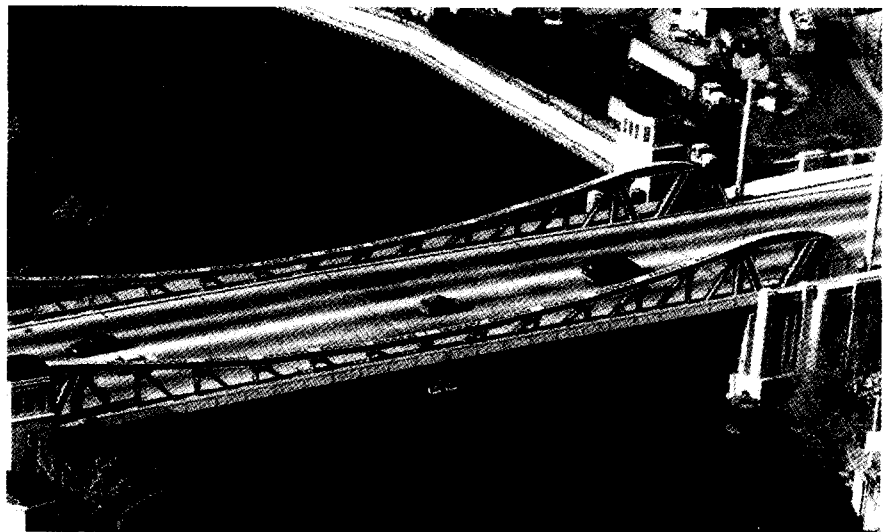
The bridge's original driving surface of wood decking with asphalt planks (c. 1935) had been replaced in 1963 with an open steel grating supported on transverse steel channels (jack beams) that spanned between the original steel stringers. The open steel grating was the driving surface of choice for CDOT for many area bascule bridges in that time period, due to the economy of weight and the elimination of snow removal efforts on the movable leaves. In the case of the North Ashland Avenue bridge, however, it left the steel floor system open to corrosion from de-icing salts, weather, and roadway debris.

Design features

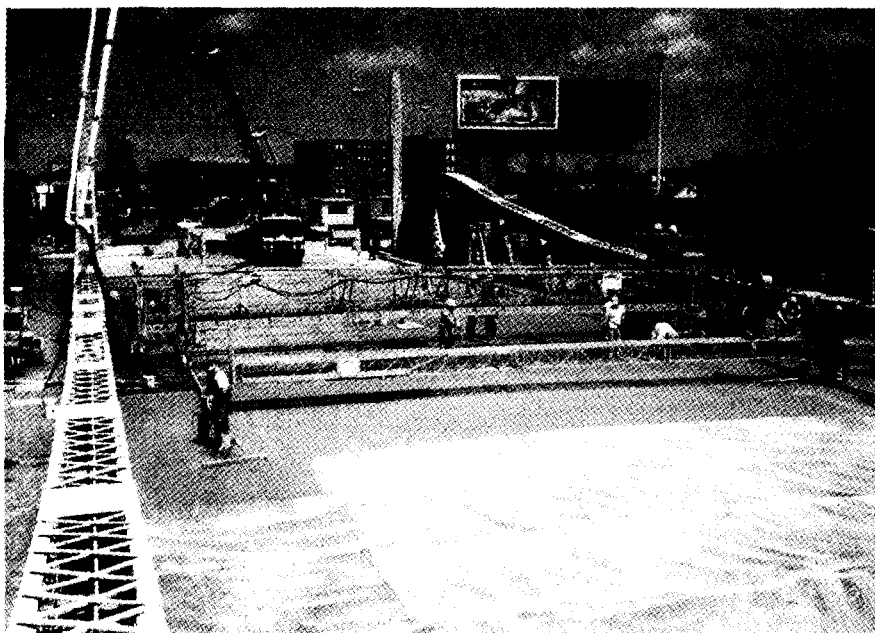
When CDOT closed the bridge to traffic, agency officials, Teng & Associates, Inc., and Cook County Highway Department (CCHD), which would eventually fund the reconstruction, quickly met to resolve two critical issues. The assembled team was faced with the challenge of developing a closed deck floor system that would alleviate the performance problems of the open steel grating and bring the design to bid and construction rapidly. A 6½ in. concrete slab on composite steel stringers and floor beams was determined to be the most favorable system based on weight, serviceability, and future maintenance.

Teng & Associates was then faced with the task of finalizing the structural design and minimizing the weight of the movable leaves. Teng was able to achieve a lighter leaf weight than the original construction while complying with CDOT's "Design Standards for Bascule Bridges," which specifies a structure weighing 93,500 lb with two 35,000 lb axles spaced at 50 in.

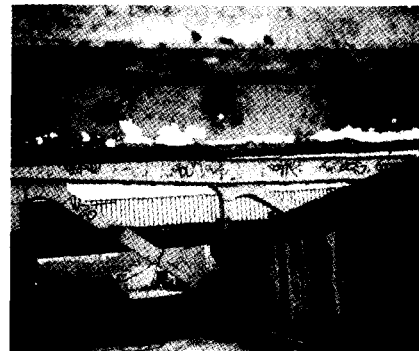
While Teng was completing the design and the preparation of contract documents for the movable leaves and the approach spans, CCHD was arranging all aspects of the bidding and award process to expedite the project. CCHD and CDOT established a deadline for the completed documents so that a June



Completed concrete bridge deck on movable spans.



Concrete placement on the movable leaves.



This view of an existing stringer prior to its removal shows the extent of deterioration of the steel system.

1, 1993 completion date coinciding with the start of an adjacent Interstate 90 reconstruction could be specified in the construction contract. Teng completed the design and contract documents in eight weeks and coordinated with CDOT in walking the project through required approvals by the Illinois Department of Transportation, the U.S. Coast Guard, and other permitting agencies. CCHD received the bid sets, advertised the job, awarded the contract, and gave the contractor notice to proceed, all within one month.

Construction

When CCHD awarded the construction contract to general contractor Walsh Construction Co., Illinois, a winter construction schedule requiring heated concrete placements was not the only obstacle preventing opening the bridge to traffic. Bascule machinery constraints for the replacement of the floor system dictated that work on the movable leaves be performed with the leaves in the "down" position. Due to this constraint, the contractor was required to submit a scheme for shoring the 1000 ton counterweight boxes that would maintain the final structure in balance until the new concrete slab was in place. Additionally, steel removal and replacement on the movable spans was limited to removal of no more than two 17 ft bays of steel before subsequent re-

placement to maintain stability of the 115 ft cantilever trusses.

CCHD retained Teng & Associates to perform the on-site resident engineering and the project was completed on time and within budget on a fast track schedule at the end of May, 1993. A dedicated team effort by CDOT, CCHD, Teng, and Walsh Construction were responsible for the completion of the \$2.7 million reconstruction only nine months after the bridge was closed to traffic. The benefits of a conventional reinforced concrete deck will also be used on a new bascule bridge at Eighth Street over the Sheboygan River in Sheboygan, Wisconsin. Teng and Associates recently completed the Sheboygan design for the Wisconsin Department of Transportation utilizing lightweight concrete; construction is scheduled for the spring of 1994.

Selected for reader interest by the editors.

ACI member **Byron**

T. Danley is a project manager with Teng & Associates, Inc., an engineering/architectural firm based in Chicago, Ill. He is a member of ACI Committee 345, Concrete Bridge Construction and Maintenance.



Rehabilitation of an Elevated Roadway Bridge

by Alexander M. Vaysburd

Rehabilitation of concrete in bridges prematurely damaged by deterioration continues to be a major problem throughout North America. Approximately 564,000 bridges are standing in the United States. At least 105,500 of them need repair or replacement. Millions of square feet of bridge decks show heavy deterioration—cracking, spalling, corrosion of reinforcement, etc.

One such bridge, the elevated roadway at Baltimore/Washington International Airport, was repaired between September 1988 and July 1989. Built in 1977, the bridge superstructure is supported by 152 concrete columns and two abutments on spread footings. The 1720 ft (524 m) long concrete structure (Fig. 1) consists of 73 spans in continuous units three or four spans long, with cantilevers at the end of each unit and a suspended slab from 12 ft (3.7 m) to 28.8 ft (8.8 m) long between cantilevers (Fig. 2). An expansion joint is provided at each end of the suspended spans and at each abutment.

The width of the bridge varies from 24 to 44 ft (7.3 to 13.4 m) of clear roadway with a parapet and a safety walk on one side of the bridge and a sidewalk on the other. The bridge deck is a 20 in. (51 cm) thick cast-in-place reinforced concrete slab. The positive moment regions in the deck slab in each 24 ft (7.3 m) clear roadway span contain 20 equally spaced 10 in. (25 cm) diameter voids.

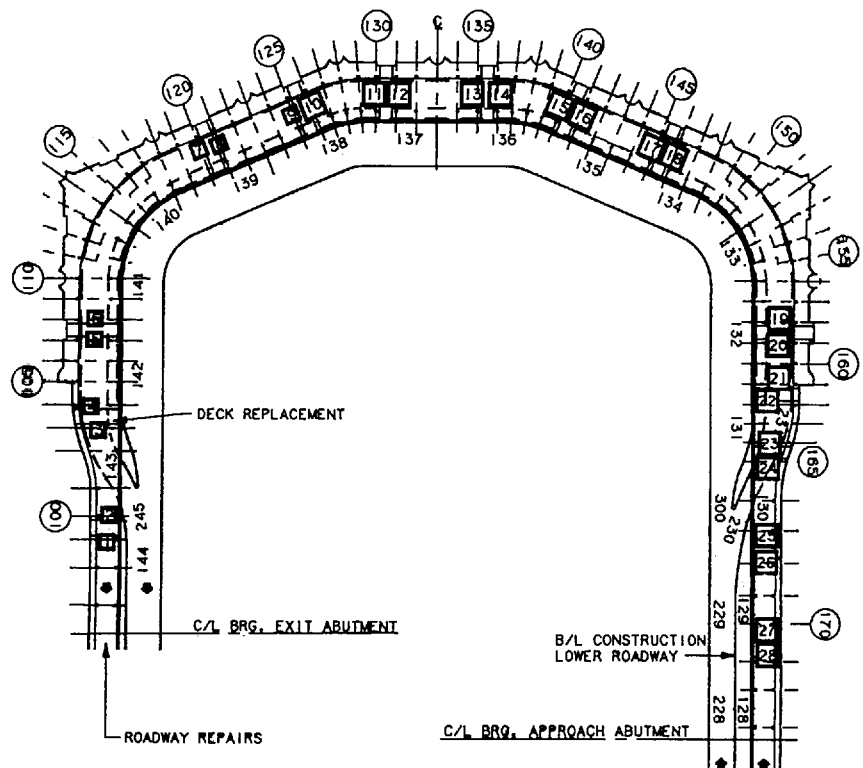


Fig. 1 — Bridge plan view.

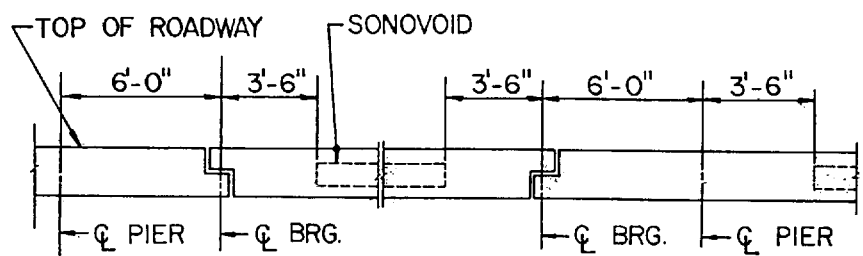


Fig. 2 — Typical suspended span longitudinal section.

The specified concrete strength for the columns and deck slab was 4500 psi (31 MPa).

Field inspection

The first phase of the project included an in-depth inspection to determine the physical condition of the structure. The bridge was visually inspected for spalls, cracks, rust stains, discoloration, corrosion of reinforcement, and other indications of concrete deterioration. The surface areas were also inspected by sounding (hammer tapping) to locate delaminated areas. The inspection yielded the following:

- The top of the slab was free of deterioration with the exception of some minor spalls along the expansion joint angles.
- Cracking, light at most locations and moderate at others, existed throughout the deck surface. Transverse cracking was predominant, although some longitudinal and random cracking was observed.
- Severe deterioration was found on the underside of the deck slab at the expansion joints. Leakage through the joints had caused extensive corrosion of the reinforcement, spalling, and delamination of the concrete cover (Fig. 3). Concrete spalls at the cantilever supports were about 4 in. (10 cm) deep at several locations and the main reinforcing bars were heavily corroded.
- The specified cover to reinforcement at the bottom of the deck slab was 1 in. (25 mm); measurements indicated that at some locations the cover did not exceed ¼ in. (6 mm).
- The concrete sidewalk at several locations had severe random cracking with crack widths varying from fine to ¼ in. (6 mm).
- Of the 30 expansion joints, 26 had preformed neoprene elastomeric compression seals. Four joints had been repaired in 1986 by replacing the existing compression seals. All of the remaining joints were showing evidence of leaks (some very severe.)

Testing program

A concrete sampling and testing program was carried out to supplement visual observations and to determine the potential durability of the concrete and the expected service life of the structure. This program included obtaining and testing cores and samples for compressive strength, chloride ion content, depth of carbonation, pH value, and for performing petrographic examinations. The sampling plan was designed to obtain samples from concrete areas judged to have light, moderate, and severe deterioration. The most severe areas of concrete deterioration were adjacent to expansion joints on the underside of the deck slab.

Cores taken near the expansion joints (Fig. 4) were visually examined and tested for compressive strength. A total of 15 cores were taken from various locations throughout the bridge.

Using the dry pulverizing method, 12 samples for chloride ion content were taken from six locations on the underside of the deck slab adjacent to expansion joints and the longitudinal construction joint. Six of these were shallow samples taken at a depth of 0 to 3 in. (0 to 7.5 cm). The remaining six were deep samples taken at a depth of approximately 5 in. (12.5 cm).

For depth of carbonation tests, cores 3 in. (7.7 cm) in diameter were obtained at three locations. Two samples of concrete dust (left over from the chloride ion testing) were taken to determine the pH value of concrete.

Compressive strength

Compression tests performed on cores taken at the top of the deck in suspended slab areas did not indicate concrete deterioration. (Compressive strength averaged 7424 psi [51 MPa].) Cores taken from the underside of the deck at the cantilever supports and adjacent areas of suspended slabs were visually examined. The top portions of the cores (inside the joint) did not show any concrete deterioration, but the

bottom portions contained deteriorated concrete from ½ in. (12.5 mm) to 3 in. (76 mm) thick. Some of the cores contained delaminations and cracks, and the reinforcement in two cores had corroded to the extent that the bond between concrete and steel was severely affected. Average compressive strength of the two cores tested was 7590 psi (52 MPa); eight cores could not be tested due to either included reinforcing bars or delamination and cracking.

Carbonation

Laboratory measurements of depth of carbonation using a phenolphthalein indicator showed that color changes occurred at an average depth of ⅜ in. (8 mm).

Tests for pH value indicated an alkalinity of the concrete of about 11.

Chloride content

Sampling and testing for total chloride content was carried out in accordance with AASHTO T-260. The shallow samples showed levels of chlorides that ranged from 0.3 to 7.4 lb/yd³ (0.18 to 4.4 kg/m³) of concrete, and the deep samples ranged from 0 to 3.2 lb/yd³ (0 to 1.9 kg/m³).

Petrographic examination

The results of a petrographic examination indicated the concrete was properly made. The entrained air-void system was not entirely adequate, but there were no signs of freeze-thaw distress. The average entrained air volume was about 5 percent. The paste was of low water-cement ratio and was hard and dense. The bond of the coarse and fine aggregates to the cement matrix was good. The paste was well cured and hydrated.

The concrete had been affected by an alkali-silica reaction that generated fine cracks in the concrete. The orientation of the cracks was both parallel and normal to the exterior concrete surface. A gross manifestation of the reaction was the map, or pattern, cracking evident on the concrete surface. The cracks were shallow at some loca-



Fig. 3 — Spalling of concrete along the expansion joint.

Table 1 — Concrete mix for pumped concrete

Material	Mix proportions	
	lb/yd ³	kg/m ³
Portland cement Type I	750	445
Fine aggregate	1,400	830
Coarse aggregate (AASHTO M43 Size 7)	1,650	980
Water	31 gals.	155
Water reducing admixture	112 ozs.	3 L
Air entraining admixture	9.5 ozs.	0.6 L
Air content	6.5 ± 1.5%	6.5 ± 1.5%
Totals	4,066	2,414
Slump	8 ± 1 in.	20 ± 2.5 cm
Minimum 28-day compressive strength	5,000 psi	35 MPa

tions, and penetrated to a depth of about 1 in. (2.5 cm) at other locations. The paste immediately adjacent to the cracks was carbonated. In the interior of the concrete, the affected chert particles had become soft and internally cracked without the cracks extending into the mortar matrix.

Test results

Cracking

Transverse cracking was the predominant type of cracking observed. Shrinkage, induced by the differential rate of drying through the depth of the relatively thick slab, is believed to be involved in transverse cracking. Sizable tensile stresses tend to be developed in the first inch or so from the drying face, whereas restraining compressive stresses tend to develop in the interior of the slab.¹

The longitudinal cracking is believed to have resulted primarily from resistance to settlement and from shrinkage of the plastic concrete around void tubes and reinforcing bars. Inadequate cover at the bottom of the deck also increased the shrinkage cracking in the thin section of the concrete directly over or slightly offset from the reinforcing steel.

Initially, cracking may have resulted from several causes such as shrinkage, thermal stresses, alkali-silica reaction, or a combination of these causes. In any case, once formed, the cracks are repeatedly opened and closed by traffic action.

Precipitation ponding on top of the slab due to inadequate drainage can infiltrate the concrete through the cracks and, in combination with subsequent cycles of freezing and thawing, will result in progressive corrosion of reinforcement and deterioration of the concrete.

Carbonation

Carbonation results in various physical changes, but the main effect is a reduction in the alkalinity of the concrete from approximately

pH 12.8 to pH 9. This change causes the reinforcing steel to be unprotected from corrosion by the alkaline conditions.

The carbonation front moves into the concrete from the exposed surface at a rate that is dependant upon the moisture content of the concrete. The front progresses rapidly when relative humidity is between 50 and 70 percent and is negligibly slow at both low and high relative humidities.²

Using a simple relationship, $d =$

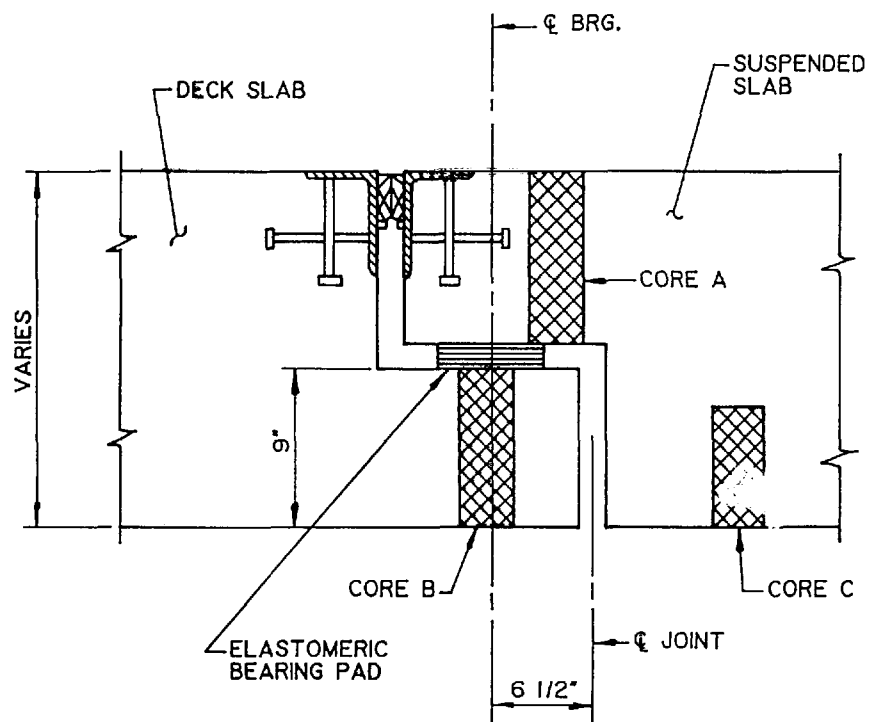


Fig. 4 — Core location sections.

The most important factor in selecting methods and materials for repair is the compatibility of the existing concrete and the new materials.

$K_c\sqrt{t}$, where d (mm) is the depth of the carbonation reaction front after time t (years). The carbonation coefficient K_c is related to the permeability of the concrete, the amount of reactible lime in the concrete, the relative humidity, and the carbon dioxide content of the particular environment. $K_c = 2.6$ for the concrete deck bottom surface,³ and the calculated depth of 0.33 in. (8.5 mm) approximated the actual measured values.

The pH of the concrete was about 11, which correlated with the depth of carbonation tests and indicated that the process of carbonation had not significantly reduced the normal alkalinity of the tested concrete. Both the position of the carbonation front and the measured pH clearly indicated that the reinforcing steel with adequate cover in otherwise non-distressed concrete was protected from corrosion.

Corrosion

The total values of chlorides found at the level of reinforcement in spalled areas were well in excess of critical amounts of chlorides required to create a corrosion problem. High salt concentrations resulted from a progressive build-up of chloride levels due to saline water passing through the expansion and construction joints and capillary movement of saline solutions through the concrete.

Deicing salts are carried with snow and ice on the underside of vehicles. The salty snow and ice that melts was ponding on the bridge deck due to inadequate drainage and seeping through damaged seals at expansion joints. Reduced concrete cover and some micro-cracking in the concrete surface permitted moisture, chlorides, and oxygen to penetrate to the reinforcing bars.

Chloride ions penetrate the concrete to the level of the reinforcing steel slowly in non-cracked areas

and more quickly through cracks. Because of this, chloride ions are not uniformly distributed within the concrete. Chloride differentials along the reinforcing bars or between closely located bars, not merely the total chloride values, play a critical role in the corrosion initiation process. Portions of the reinforcement remain passive while the chloride ions degrade the passive film at other areas of the steel.

The amount of chlorides necessary to destroy the passive film is not a single value for all situations. It depends on the pH, water and oxygen contents, condition of the interface between concrete and reinforcement, and possibly other factors. The pressure developed by the increased volume of corrosive products of only 0.004 to 0.008 in. (0.1 to 0.2 mm) may be sufficient to damage the concrete cover by causing cracks to develop radially from the reinforcing bar in a variety of directions.⁴

At slab corners, along the expansion joints, the spalling was more severe because the cracking took place more rapidly due to faster chloride, moisture, and oxygen penetration and build-up from the two surfaces. Also at this location, the resistance to cracking provided by adjacent concrete for an infinite surface was lost at the surface boundary.

At some locations where concrete cover did not exceed ¼ in. (6.5 mm), the corrosion process seemed to begin with no chlorides or with a low value of chlorides. This may occur because the front of carbonation reached the reinforcement and the concrete alkalinity did not protect the steel from corrosion. Found by visual inspection, these areas were the most severely deteriorated.

The zone along the expansion joints at the underside of the deck had suffered the most damage from corrosion of reinforcing steel. It had

also been exposed to repeated cycles of wetting and drying and freezing and thawing, often in a saturated condition.

Alkali-silica reaction

The alkali-silica reaction also contributed to concrete deterioration. It was very difficult, if not impossible, to determine the existing state of the reaction and to predict the remaining service life of the structure. No correlation was observed between the degree of alkali-silica reactivity, the external deterioration, the age of the concrete, and the results of the compressive strength tests. Considering that the concrete is still structurally in good condition and remains confined within the reinforcing steel, structural failure is most unlikely.⁵ Also, by minimizing moisture and salt penetration into the concrete further expansion may be slowed down.

Repairs

The repairs can be broadly classified as: repair of spalls along the expansion joints, repairs of cracks, and repair of expansion joints.

Compatibility of materials

The most important factor encountered in selecting methods and materials for repair was the compatibility of existing concrete and new materials. Many materials change volume as they initially set, and practically all of them change volume with temperature and moisture changes. Tensile stresses are induced in one material and compressive stresses in the other, causing a substantial shear at the interface. Identical patterns of stress will result from the differential shrinkage and different moduli of elasticity.⁶

Considering all of these factors, pumped concrete was selected as a repair material for spalled areas along the expansion joints.

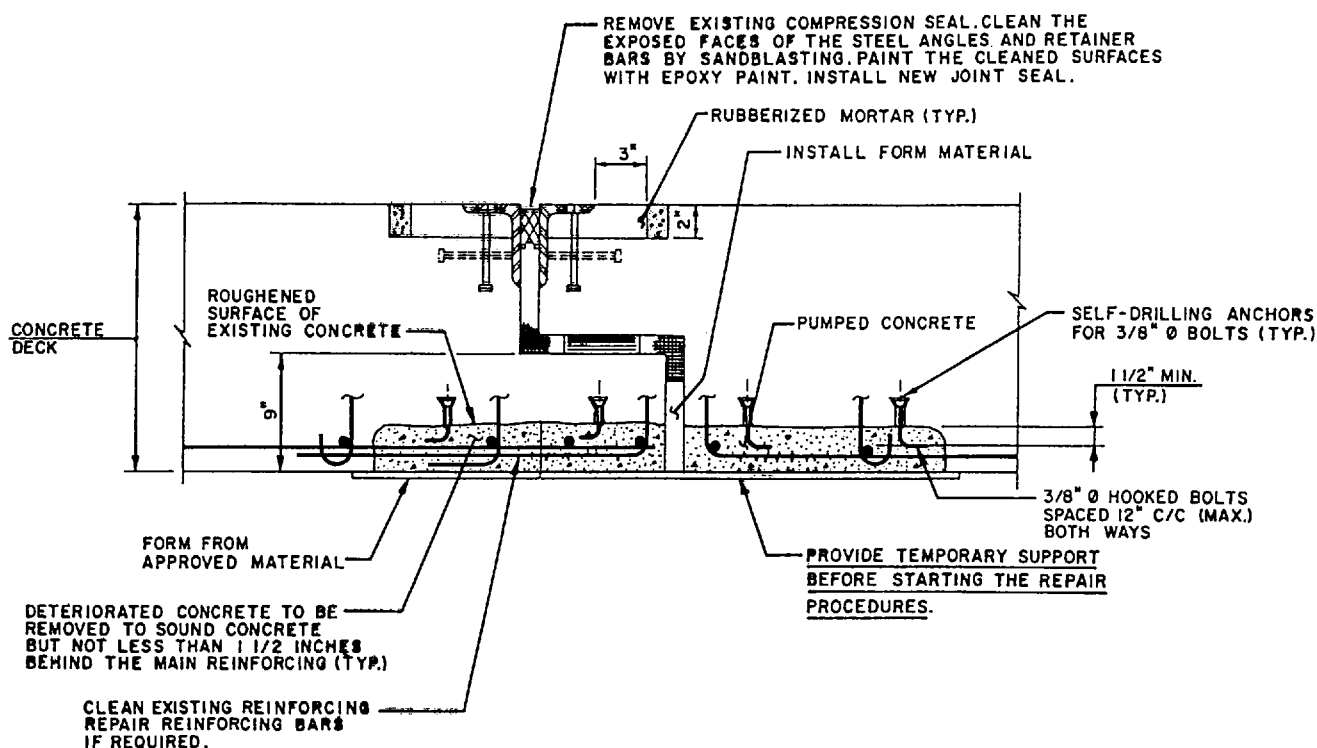


Fig. 5 — Expansion joint repair detail.

Preparation for repair

Before the repair operation started, temporary supports for the suspended slabs were installed.

Proper preparation of the surface for repair is critical to the success of the repair. In some cases, the care exercised during the removal of deteriorated concrete and preparation of a repair area will determine the longevity of the repair, regardless of the material and technique used.

The concrete surface was prepared by removing the deteriorated concrete to sound concrete, but not less than to a depth of 1½ in. (3.8 cm) behind the main reinforcement. At several locations, the depth of deteriorated concrete was about 4½ in. (11.5 cm) into the cantilever support of a suspended slab and the cantilever support was completely replaced with new concrete.

The chipping was done using hammers with a maximum weight of 30 lb (14 kg). Realizing that it would not be easy to accurately determine when all unsound material is removed, specifications recommended removing concrete until the coarse aggregate particles were being broken rather than simply removed from the cement matrix. The concrete surface and exposed rein-

forcing steel were then sandblasted and finally cleaned of loose material with compressed air. An anchoring system was installed using self-drilling anchors for ¾ in. diameter hooked bolts spaced in a 12 x 12 in. (30.5 x 30.5 cm) grid (Fig. 5). Just prior to pumping the repair concrete, the existing surface was dampened with water.

The pumped concrete was an air-entrained portland cement concrete mixture with a minimum compressive strength of 5000 psi (31 MPa). Mix proportions for the pumped concrete are given in Table 1.

Epoxy crack sealing

Cracks wider than ¼ in. (3 mm) and cracks with spalled edges in the horizontal surfaces of the sidewalks were repaired using a low viscosity epoxy that has a workable life of approximately 45 minutes at 75 F (24 C) and feeds into the crack by gravity until it hardens. The exposed face of the crack was V-grooved ¾ in. (19 mm) deep and ½ in. (12 mm) wide, cleaned, and sealed with epoxy.

Epoxy pressure injection

All remaining cracks in the deck slab, parapets, abutments, and retaining walls were repaired by injection of a two-part epoxy composed

of a resin and hardener. As part of quality control, random cores were drilled through the repair after curing to visually evaluate crack filling. Control of the epoxy properties such as viscosity, cure temperature, pot life, and moisture insensitivity are very important for successful repairs.

Expansion joints

Bridge expansion joints must accommodate all superstructure movements and carry impact loads while being exposed to the prevailing environmental conditions. Infiltration of incompressible materials and water is a severe problem which a joint should be designed to prevent. Joints should safely accommodate all traffic passing the bridge and allow for snow plow operations without sustaining damage. Joints should be watertight and easy to maintain. The design of expansion joints for such severe conditions has become a challenging problem.

The repair of expansion joints included installation of elastomeric concrete nosing, epoxy coating of the steel angles, and installation of new seals.

The elastomeric concrete used was a pre-packaged material consisting of three components: a resin,

A more detailed version of this article is available in SP-122, "Paul Klieger Symposium on Performance of Concrete," an ACI special publication containing 24 papers presented at The Paul Klieger International Symposium on Performance of Concrete, sponsored by ACI Committees 201, Durability, and 222, Corrosion of Metals.

a hardener, and a mineral filler material. The existing concrete was removed along the expansion joints, the surfaces receiving the new material were cleaned, and a self-leveling elastomeric concrete mix was placed.

The old seal was removed from the joint and the top and inside portions of the steel angles were blast cleaned to a white metal finish immediately before the application of the bonding agent and the installation of the new seal material. After installation of the new seal, epoxy paint was applied to the exposed surfaces of the expansion joint angles.

Surface protective treatment

The final step in the rehabilitation project was the application of a penetrating sealer to the bridge deck, sidewalks, parapets, and abutments to minimize moisture and chloride penetration into the concrete.

Conclusions

Concrete strength is not a criterion for durability of concrete bridge structures. Quality factors such as design, inspection, concrete composition, and good workmanship are essential ingredients.

To minimize the difficulties and associated costs of repairing expansion joints, and the related concrete deterioration, joints should be properly designed to allow for easy maintenance, inspection, cleaning, and repair. In the author's opinion, good design practice should reduce the number of expansion joints to an acceptable minimum. Reduction of the amount of the joints may be accomplished by designing for continuity and taking into account the flexibility characteristics of the structure.

Thickness and quality of the concrete in the protective layer are important for protection of steel reinforcement against corrosion and durability of a concrete structure. Assuming that the permeability of the concrete is constant, reduced cover decreases the protective physical barrier between reinforcement

and corrosive agents and also increases the probability of shrinkage cracking in concrete directly over the reinforcing bar. It is very important in specifying the thickness of the concrete cover to consider the potential displacement of the reinforcing steel during field operations.

It is very important in assessing the physical condition of a concrete structure to determine the depth of the carbonation front and levels of chlorides present in concrete. A combination of carbonation and chlorides increases the corrosion rate and reduces the critical values of chlorides necessary for initiation of corrosion.

A variety of products and methods exists to repair or control corrosion destruction in bridges. To assure that a proper solution is selected for a particular condition, the engineer must understand not only the structural mechanism of the problem, but also compatibility of the materials.

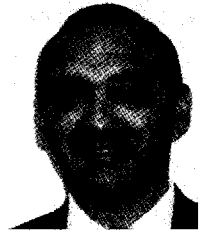
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Received and reviewed under Institute publication policies.

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Polyester Concrete for Bridge Deck Overlays

by D. N. O'Connor and M. "Saïid" Saïidi

A major problem facing federal and state Departments of Transportation is the deterioration of highway bridge decks. The problem is accelerated by the widespread use of deicing salts to keep roads free of snow and ice in the winter. Cracking and spalling is caused by oxygen, chloride, and water that infiltrates the deck, resulting in corrosion of the reinforcing steel.

The Nevada Department of Transportation currently is using 0.75 in. (20 mm) thick polyester-styrene polymer concrete overlays for bridge deck protection. Previous investigation¹ has shown that polyester concrete can provide effective protection for portland cement concrete bridge decks.

Additionally, research was conducted to determine the basic engineering properties of the polymer concrete currently in use. Laboratory testing was conducted to determine the effects of elevated temperature and temperature cycling on the compressive strength of polyester concrete, the strength of bond between overlay concrete and portland cement concrete, and the modulus of elasticity of polyester concrete.

High temperature sensitivity

Compressive strength is the measured maximum resistance of a material to axial loading, and is an important property of polyester concrete because other properties, such as modulus of elasticity and modulus of rupture, have been found to be proportional to the square root of the compressive strength.

Temperature effects on the compressive strength are of concern because research has shown that the compressive strength of polyester concrete decreases as temperature increases. The polyester concrete used in protective bridge deck overlays may be subjected to solar radiation, which heats the overlay. To better understand the behavior of the polyester concrete currently in use, compressive strength was measured over a range of temperatures.

Compressive strength of the polymer concrete was determined in accordance with ASTM C 39,⁷ but with a loading rate of 83 psi/sec (572 kPa/sec), as specified in California Test 551.⁸ The higher loading rate was used to reach maximum load in no more than 80 sec from the start of loading.⁹

Temperature cycling sensitivity

Although polyester concrete compressive strength decreases as temperature increases, no data were found to indicate

whether residual strength loss exists after the concrete has returned to normal temperatures.

Likewise, data concerning the effects of numerous heat-up and cool-down cycles on compressive strength were not available. This test was designed to quantify the effects of temperature cycling.

Compressive strength was measured after a given number of temperature cycles. Each temperature cycle consisted of 6 hr at 150 F (66 C), followed by 18 hr at room temperature. The compressive strength of the concrete subjected to cycling was then compared to the compressive strength of a control group to determine the effect of temperature cycling.

Bond strength

Polyester concrete overlays are applied to bridge decks to protect against galvanic corrosion of the reinforcing steel and subsequent cracking and spalling of the concrete. Because the overlay seals the top surface of the slab against water and salt intrusion, the overlay must remain firmly bonded to the deck. The overlay bond strength to deck concrete must be greater than the tensile strength of the portland cement concrete.

California Test 551⁸ was used to determine the bond strength. The test measures the modulus of rupture of a beam specimen consisting of a half-span portland cement concrete beam, to which is bonded polymer concrete to form the other half-span. The beam is then loaded in flexure at the center of the span, directly onto the bond line.

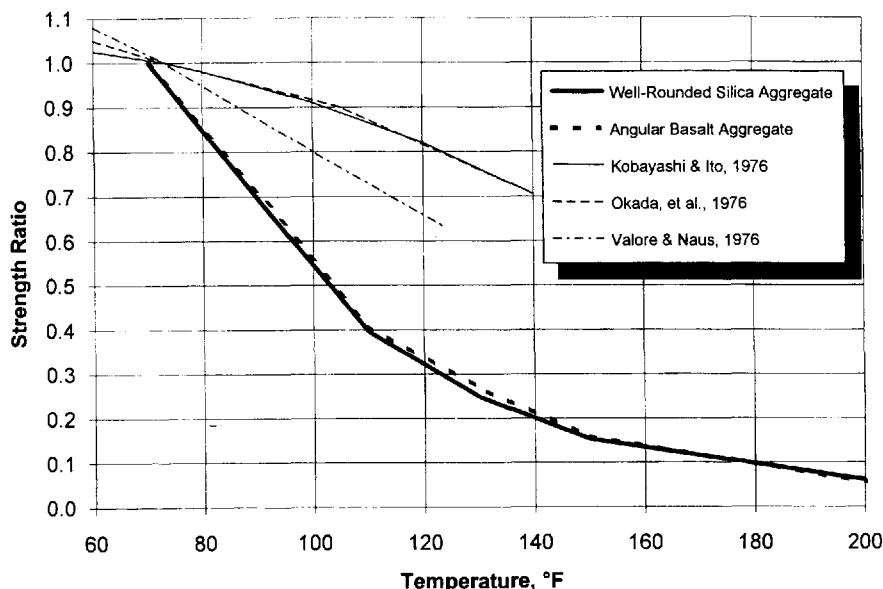


Fig. 1 — Sensitivity of polyester concrete compressive strength to temperature

Modulus of elasticity

Modulus of elasticity is a principal property of concrete, and is a measure of the deformation that will occur under a given load. An empirical relationship exists to calculate the modulus of elasticity, given the compressive strength.³ However, this relationship was determined in the mid-1970s and may not be applicable because more flexible resins are currently being used.¹⁰ As part of the tests discussed in this article, the modulus of elasticity of the polyester concrete was determined according to ASTM C 469.⁷

Materials

The polyester resin used in the testing program was Transpo T-35 resin, manufactured by Alpha Resins Corporation, and is typical of resins used by several western states. It is an unsaturated isophthalic polyester resin dissolved in styrene monomer and pre-promoted with cobalt naphthenate for room-temperature curing. Methyl ethyl ketone peroxide was used as the initiator. Transpo T-70 high molecular weight methacrylate (HMWM) primer was used in the bond strength testing; this acrylic resin uses cumene hydroperoxide and cobalt octoate as the initiator/promoter system.

Two polyester concrete aggregate types were used: a No. 4 (4.75 mm) maximum size well-rounded silica aggregate from Marina, California and a .37 in. (9.5 mm) maximum size angular basalt aggregate from Renton, Washington. Aggregates and cement for fabricating portland cement concrete bond strength blanks were obtained from a local source.

Polyester-styrene polymer concrete

Optimum resin contents determined in previous testing² were exclusively used in this program. The optimum initiated resin content for polyester concrete made with the silica aggregate is 12 percent by weight. For concrete made with the basalt aggregate the optimum resin content is 13 percent by weight. Both mixtures use an initiator fraction of 1.5 percent by weight of resin.

Specimens for evaluating temperature effects were 2 x 4 in. (50 x 100 mm) cylinders; modulus of elasticity specimens measured 4 x 8 in. (100 x 200 mm).

Effects of high temperature

Compressive strength of polyester concrete was determined at 70, 90, 110, 130, 150, and 200 F (21, 32, 43, 54, 66, and

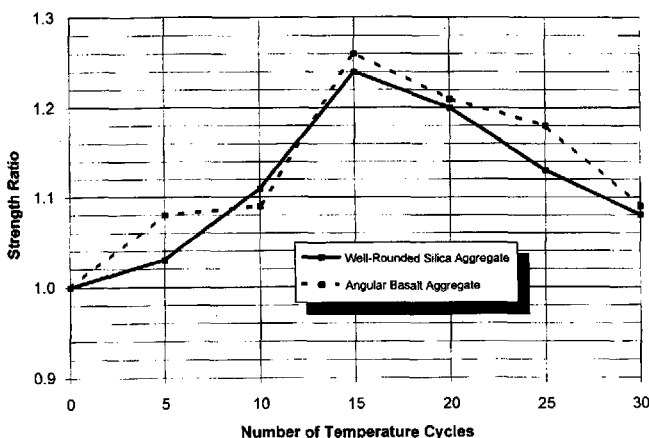


Fig. 2 — Temperature cycling on polyester concrete compressive strength

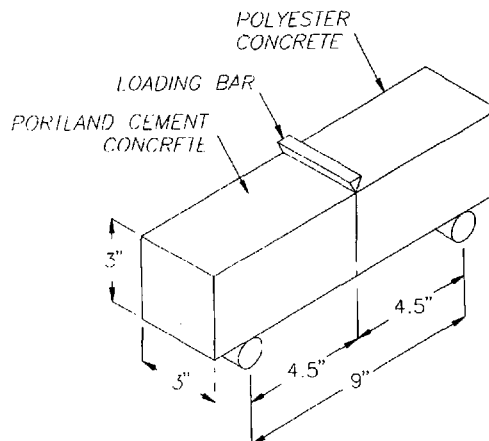


Fig. 3 — Bond strength specimen

3 C) and compared to the compressive strength of control specimens kept at room temperature. Elevated temperature specimens were allowed to soak for 24 hr and were tested hot, immediately after removal from the oven. Fig. 1 illustrates the relationships between strength ratio and temperature.

The relationship between compressive strength and temperature did not match the relationships from other researchers,^{3,4,5} also shown in Fig. 1. To ensure that the polyester resin was not defective, a small amount of the same type resin was obtained from another source. Concrete made with this alternate resin displayed the same behavior, indicating that the original resin was not faulty.

Effects of temperature cycling

The sensitivity of polyester concrete to temperature cycling was determined by comparing the compressive strength of the concrete after cycling to the compressive strength of control specimens. Each cycle consisted of a 6 hr soak at 150 F (66 C) followed by 18 hr at room temperature. Specimens were tested after 5, 10, 15, 20, 25, and 30 cycles.

Fig. 2 presents the results of temperature cycling for polyester concretes made with the two aggregate types. Initially, the compressive strength increased under heat-up and cool-down cycling, possibly as a result of additional curing at high temperature. Beyond 15 cycles the compressive strength began to decrease. However, over the 30 cycles of testing, compressive strength remained greater than the initial strength. Due to time constraints, longer cycling programs were not conducted.

Bond strength

Earlier research concerning the thermal compatibility between polyester concrete and portland cement concrete found that the bond between the overlay and the deck can be critical to the success of the protective system.¹ Ideally, the bond should have a shear and tensile strength greater than those of the bridge deck concrete; this will ensure that failure does not occur at the interface between the deck and the overlay. This method determines the modulus of rupture of a beam consisting of polyester concrete bonded to portland cement concrete. This test provides baseline bond strength information at room temperature. A typical specimen is illustrated in Fig. 3.

Bonding blanks for the bond strength test were fabricated with portland cement concrete designed to model the prop-