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# MYCONCRETE

THE BULLETIN OF THE AMERICAN CONCRETE INSTITUTE - MALAYSIA CHAPTER
(E-bulletin)

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**MyConcrete:**
**The Bulletin of the American Concrete Institute – Malaysia Chapter**

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# ARTICLE

Reprint from CI Magazine, Volume 39, No 4 , Page 41-46

# Precast Concrete Pavement Implementation

## Projects in Connecticut and Texas demonstrate a great potential for repair and rehabilitation.

*by Shiraz Tayabji and Sam Tyson*

Precast concrete pavement (PCP) technology is gaining wider acceptance in the United States for the rapid repair and rehabilitation of heavily trafficked hot mixed asphalt (HMA) and portland cement concrete pavements. In the former cases, PCP is being used to replace HMA ramps, intersections, and bus pads that exhibit extensive rutting. In the latter cases, PCP is being used to make intermittent, full-depth repairs of concrete pavement joints and panels as well as to complete full-depth replacements of large contiguous areas of concrete pavements.

Over the last 16 years, many advances have been made in the design, fabrication, and installation aspects of PCP technology, leading to reduced costs, improved performance, and faster installation.[1] And, many projects have been constructed by contractors with no prior experience in the technology.

Departments of transportation (DOTs) in several states— including California, New Jersey, New York, Utah, and Illinois—now routinely use PCP for rapid pavement rehabilitation, including many projects that have required hundreds or thousands of panels.[2] Rehabilitation work using PCP is typically performed overnight over a single lane, and it includes the following steps:

- Laying out repair area, typically 600 to 800 ft (183 to 244 m) of a lane;
- Making saw-cuts in the repair area 1 to 2 days before the night of panel placement; and
- Taking the following actions during each night of panel placement:
  - Starting lane closures at about 8:00 p.m. A two-lane closure is a minimum requirement unless good shoulder access is available;
  - Removing existing pavement and the base per design or as needed;
  - Compacting and grading the existing base or new granular base as needed. In California, an existing cement-treated base or lean concrete base is typically removed and replaced with a mobile-mixer-produced, rapid-setting lean concrete base;
  - Starting just before midnight, placing about 40 to 50 panels within 4 to 5 hours of available time;
  - Completing finishing work during the remaining hours before opening the lanes to traffic. Finishing work includes installing a thin (1/4 to 1/2 in. [6 to 13 mm]) layer of high-strength bedding grout beneath the panels and placing rapid-setting, high-strength grout/patching material within slots or ducts for load-transfer devices; and
  - Opening the lanes to traffic by 5:00 to 6:00 a.m. or as required by the contract.

## Background

PCP technology can be used in discrete (intermittent) or continuous applications. In the former cases, workers replace isolated pavement panels or pavement on each side of a joint. Load transfer between the PCP and existing pavement is normally achieved using dowels. The installation of an intermittent repair at a joint is shown in Fig. 1.

**Fig. 1: A PCP repair is installed to restore an isolated pavement joint**

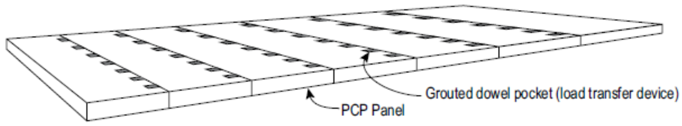


**Fig. 2: Multiple adjacent PCP panels can be used for full-scale reconstruction of jointed pavements**

In the latter cases, large-scale rehabilitation (resurfacing) or reconstruction is performed using multiple, contiguous precast concrete panels. Load transfer between the panels can be achieved using dowels or a combination of shear keys plus post-tensioning. These systems are typically termed jointed PCP or post-tensioned PCP, respectively. A schematic of a continuous application of a jointed PCP system is shown in Fig. 2.

To date, PCP has been most widely applied to restore main sections and access ramps in the interstate highway system, where traffic is heavy and alternative routes are unavailable. Other important applications of PCP include replacement of existing pavement at intersections of urban streets, bus pads, and bridge approach slabs. In all cases, repairs can be completed overnight. And because PCPs are not susceptible to shoving and rutting, they provide superior performance to HMA reconstructions.

Since 2001, several generic and several proprietary PCP systems and components have been developed. PCP systems are usually differentiated by the detailing of the load-transfer mechanism at transverse joints or by the system used to ensure full panel bearing with the base. Typically, the load-transfer mechanisms at joints comprise dowels installed in slots or ducts formed in the panels.[3] The mechanism is completed by filling the slots or ducts with grout. PCP applications also require the installation of an "interlayer" of material between the base and the bottom of the panels. In some systems, the interlayer is placed and leveled before panel installation.[4] In other systems, the interlayer is pumped into place after panel installation.

## FHWA/SHRP2 PCP Implementation Program

Between 2001 and 2007, New York and New Jersey area agencies started using PCP actively for pavement rehabilitation, and several DOTs and the Illinois Tollway constructed demonstration projects. However, the applications of PCP technology were not well documented.

In 2007, the Strategic Highway Research Program 2 (SHRP2) initiated Project R05 to develop the necessary technical information and guidelines that would encourage the rapid and successful adoption of the technology.[5] Conducted from 2008 to 2012, Project R05 demonstrated that sufficient advances had been made to reliably design and construct PCP systems. It also showed that many of the PCP systems available in the United States could help highway agencies renew their highway systems.

In 2013, the Federal Highway Administration (FHWA) and the American Association of State Highway and Transportation Officials (AASHTO) created an Implementation Assistance Program (IAP) to help state DOTs and other interested stakeholders deploy SHRP2-developed products. Seven rounds of the IAP were offered between February 2013 and April 2016. Financial support for implementation of PCP technology was provided in Rounds 3 and 6.

Funding went to agencies in Hawaii, Kansas, Texas, Wisconsin, Alabama, Connecticut, the District of Columbia, Florida, Indiana, Louisiana, Pennsylvania, and Virginia. The intent was to support the construction of a demonstration project that would provide a learning environment for operations such as fabricating panels, installing panels, and carrying out related activities needed to implement precast panel installations. Two programs for implementation of PCP technology are presented herein. Both comprise jointed PCP systems.

## Connecticut Project

In early 2016, the Connecticut Department of Transportation (CTDOT) was awarded $150,000 to help offset the cost of implementing PCP technology. CTDOT chose to apply the funds toward rehabilitating two rutted HMA pavement bus pads along a section of CTfastrak, a bus rapid transit system. CTDOT also received IAP leading to agency-wide adoption of PCP technology.

Shortly after CTfastrak became operational on March 28, 2015, about 110 ft (33.5 m) of the HMA pavement at the two bus pads at its East Main Street Station in New Britain, CT, exhibited rutting. The rutting was most severe along the bus wheel path adjacent to the platforms, and it was deep enough to prevent opening of the bus doors at the platforms. After several patches had been made, CTtransit consulted CTDOT for a permanent solution. CTDOT recommended using PCP to allow the rehabilitation work to be performed over a weekend and avoid disturbing weekday commuter traffic.[6]

CTDOT managed the rehabilitation work for CTtransit. The project specification required the use of a grout-supported PCP system. Grout-supported systems typically use a leveling system to set the panels at the desired elevation while maintaining a gap of about 1/4 to 1/2 in. between the panel bottom and the base surface. The gap is filled with a rapid-setting, high-strength grout. The contractor elected to use panels fabricated by the Fort Miller Company, Inc. (FMC). Panels were fitted with leveling lifts that were also used as panel lifting inserts. Figure 3 shows the precast panel layouts for one of the bus pads.

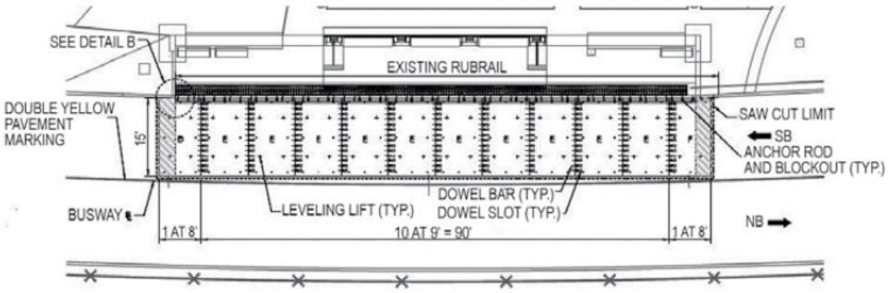


**Fig. 3: Panel layout for one of the bus pads at the East Main Street Station, New Britain, CT**

## Precast pavement details

Each station was repaired using 10 PCP panels (typical panels were 9 ft [2.7 m] long), and two 8 ft (2.4 m) long transition panels at the ends. The PCP panels were 10 in. (254 mm) thick and 15 ft (4.6 m) wide.

The existing pavement comprised 10.5 in. (267 mm) thick HMA pavement on a 6 in. (152 mm) thick processed aggregate base. The specification allowed the contractor to regrade and compact the existing base, supplementing it with new material passing the 3/4 in. (19 mm) sieve. Panels were set at the desired elevation using leveling lifts, and rapid-setting grout was pumped through ports in the panels to create a bedding layer between the base and the panels. The bedding grout was required to reach minimum compressive strengths of 500 psi (3.4 MPa) before the panels could be opened to traffic, and it was required to reach 4000 psi (27.6 MPa) at 28 days.

Joint load transfer was provided by 1.25 in. (32 mm) diameter, 14 in. (356 mm) long dowel bars uniformly spaced at 12 in. (305 mm). The PCP panels had bottom slots for the dowels, and the slots were filled with a rapid-setting grout. The grout was required to reach a strength of 2500 psi (17.2 MPa) before the panels could be opened to traffic. To ensure smooth transitions, the HMA pavement was milled and new HMA was placed to match the PCP elevations at the ends of the new pavement sections.

## Panel fabrication

The 24 panels were fabricated at the FMC precast plant in Schuylerville, NY, about 170 miles (274 km) from the project site. They were delivered to a staging area near the project site a few days before panel placement.

The panels comprise a proprietary system, referred to as the Super-Slab® system, developed by FMC in 2001. Several highway agencies have used Super-Slab panels for intermittent concrete pavement repairs and for rehabilitation of concrete and asphalt pavements. A typical panel used for the CTtransit project is shown in Fig. 4. The standard panels were provided with dowel bars on one end (Fig. 4(a)) and corresponding dowel bar slots at the other end (Fig. 4(b)).

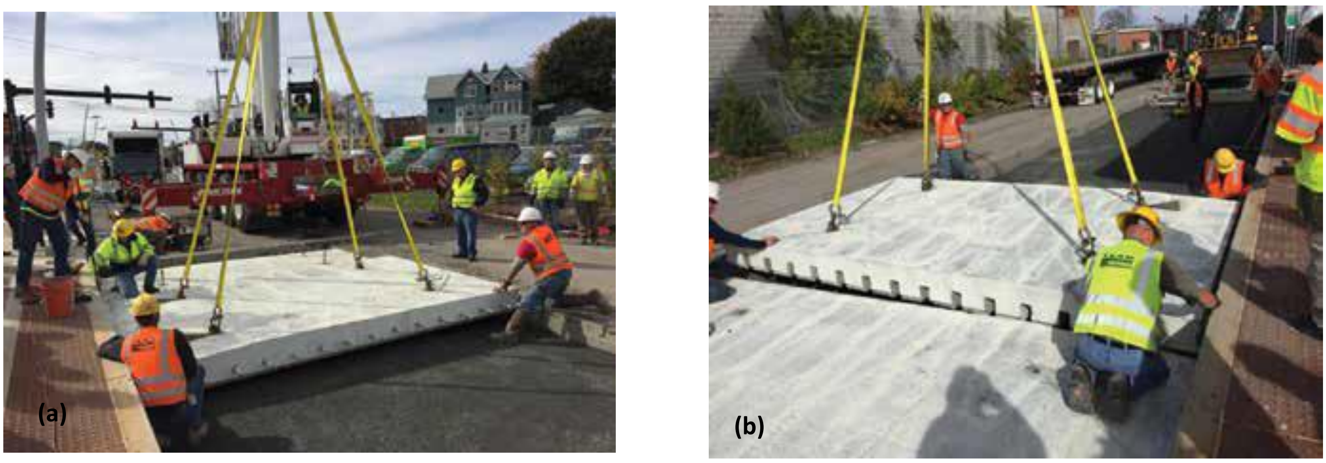


**Fig. 4: A typical precast panel used for rehabilitation of two Cttransit bus pads: (a) dowel bars embedded at one transverse side; and (b) dowel bar slots along the opposite side**

## Preparation and installation

Because of the short timing for the work, CTDOT elected to have a contractor who was already under contract with CTtransit to rehabilitate the two bus pads. The work started on a Thursday evening and was substantially complete before rush hour on Monday morning. The work included:

- Removal of the existing HMA pavement at each pad;
- Grading and compaction of the base, including addition of new crushed stone material to correct for any over excavation of the existing base;
- Placement of 12 PCP panels at each bus pad;
- Adjusting a total of 48 leveling bolts to match PCP panel elevations with the adjacent HMA pavement;
- Grouting the dowel bar slots and filling joint gaps to the level of the panel surfaces;
- Removing leveling bolts as soon as the dowel bar slot grout had hardened sufficiently to carry the weight of the panels;
- Pumping bedding grout beneath the panels;
- Milling about 3 in. (76 mm) of the HMA surface for a distance of about 10 ft (3 m) from each end of each pad and placing HMA surfacing over of the milled surface and the PCP transition panels; and
- Sawing and sealing joints.

Removal of the distressed HMA pavement was initiated during the night of Thursday, October 27, 2016, when the section designated for removal was sawcut over its full depth into 7 x 5 ft (2.1 x 1.5 m) segments. The HMA pavement sections were removed starting Friday evening, and most of the panel installation activities were completed by early morning on Monday, October 31, 2016. Bus traffic was allowed to use the completed bus pads at 4:00 a.m., the start of Monday morning's CTfastrak operation. Figure 5(a) shows the placement of the first panel, a transition panel, at the north bus pad. Figure 5(b) shows a view of the north bus pad with all panels in place and set at the desired elevation. Figure 5(c) shows a view of the north bus pad in operation.




**Fig. 5: The north bus pad rehabilitation:**
**(a) placement of the first panel;**
**(b) placed panels with leveling lifts engaged; and**
**(c) in operation**

Although a few issues (related to base preparation and panel installation) had to be resolved at the work site, the use of the PCP technology on a production pavement rehabilitation project was an important step for CTDOT. CTDOT was able to complete its first PCP installation in a challenging setting, and it expects to use the findings from this demonstration project to refine specifications and plans for use of PCP technology in future pavement repair and rehabilitation projects along roadways with high traffic volumes.

## Texas Project

In early 2015, the Texas Department of Transportation (TxDOT) was awarded $300,000 to help offset the cost of implementing PCP technology. TxDOT chose to apply the funds toward rehabilitating the SH 97 and SH 72 intersection.[7] Located near Fowlerton, TX, the intersection's HMA pavement was exhibiting extensive rutting—a combination of high temperatures and heavy truck traffic associated with the local energy industry. If the project was successful, TxDOT anticipated using the technology to rehabilitate similarly distressed HMA pavement intersections and roadway sections in the region.

The SH 97 and SH 72 intersection is T-shaped, with SH 97 as the through highway and SH 72 meeting SH 97 at a right angle. TxDOT developed generic plans and specifications for use of PCP at the intersection. The plans called for the use of pretensioned PCP panels. Figure 6 shows the panel layout and placement stages.

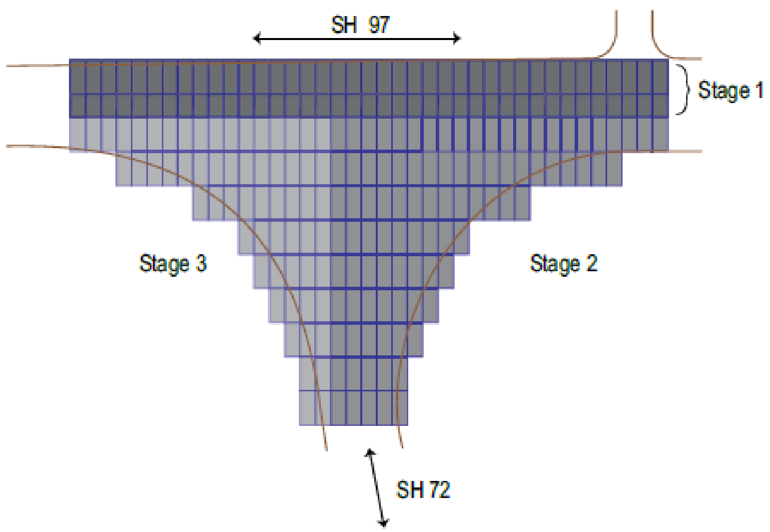


**Fig. 6: Precast panel layout and installation stages for the SH 97 and SH 72 HMA pavement intersection rehabilitation project, Fowlerton, TX**

## Precast pavement details

The project plans called for removal of the existing HMA pavement, cement-treating 6 in. of subgrade, and placing a new 4 in. (102 mm) thick HMA pavement base. The plans called for 12 in. thick PCP panels to be placed directly on the HMA base. Panel elevations were to be adjusted, if necessary, using built-in leveling lifts in the panels.

A total of 235 panels were installed. Load transfer across the joints was achieved using 14 in. long dowel bars positioned within dowel bar ducts using mini support chairs. The dowel bars were secured within the ducts using grout. The specified compressive strength for the grout was 3000 psi (20.7 MPa) in 8 hours.

## Panel fabrication

The panels were fabricated at the Bexar precast plant in San Antonio, TX. All panels were 8 ft wide. Two panel types were 18 ft (5.5 m) long, and a third panel type was 12 ft (3.7 m) long. All panels were prestressed along the long dimension. Figure 7 shows a view of the panel formwork, corrugated ducts for the dowel bars, two layers of reinforcement, prestressing strands, and lifting/leveling inserts. Figure 8 shows a view of a panel with dowel bars ready to be placed in the ducts.

## Panel installation

The panel installation was performed in three stages (Fig. 6) during the spring of 2016. Stage 1 included installation of the outer two panel lanes on SH 97 (78 panels) for a total width of 30 ft (9 m). During Stage 2 and Stage 3, panels were installed along SH 97 and SH 72 at the northern half of the rest of the intersection (68 panels) and at the remaining southern half of the intersection (89 panels), respectively.

The panels were lifted from the staging area at the intersection to the point of placement. Figure 9 shows a panel placement on the HMA base. Dowel bars with chairs were placed in the ducts along the interior side of the panels. Once a pair of adjacent panels had been placed and the elevations adjusted, the dowel bars were manually shifted in the ducts until they were centered at the joint. After all dowel bars were positioned in adjoining ducts, the ducts were filled with rapid-setting grout. Finally, after all panels for each stage were placed and grouting of all dowel-bar ducts and open slots was completed, panel undersealing was performed using grout ports extending the full thickness of the panels. Figure 10 shows the completed intersection in use.

**Fig. 7: Panel formwork within a prestressing bed**

**Fig. 8: Typical panel with dowel bars ready to be placed in the ducts**

**Fig. 9: A panel being placed on the HMA base**

**Fig. 10: Completed intersection in use (view from SH 97)**

The use of the PCP technology on an actual pavement rehabilitation project was an important step for TxDOT. Although the project did not meet the original goal of placing all panels over a weekend (to demonstrate the rapid rehabilitation potential of PCP), TxDOT considers the demonstration project at the SH 97 and SH 72 intersection to be successful.

## Summary

PCP technology is gaining wider acceptance in the United States. Implementation projects in multiple states have demonstrated that PCP systems can achieve four key attributes of successful pavements:

- Constructibility—Techniques and equipment are available for ensuring acceptable production rates for the installation of PCP systems;
- Quality—Plant fabrication of precast panels has been shown to result in excellent concrete strength and durability;
- Load transfer at joints—Reliable and economical techniques have been demonstrated to provide effective load transfer at joints in PCP systems; and
- Panel support—Techniques have been developed to provide adequate and uniform base support conditions, and improvement continues to be made.

## Acknowledgments

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Selected for reader interest by the editors.

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# TECHNICAL REPORT

Reprint from CI Magazine, Volume 39, No 5, Page 31-36

# Bonded Concrete Overlays

## Over 30 years of Swedish research and experiences

*by Johan Silfwerbrand*

Concrete overlays constitute one of the most frequently used measures to repair and strengthen concrete bridge decks, concrete pavements, and industrial concrete floors. Although unbonded overlays are used in certain applications, bonded concrete overlays are usually more widely applicable, not only for repairs, but also in new concrete structures to provide wear resistance or smoothness. Examples of such use are concrete pavements cast in two lifts or concrete toppings on elevated precast concrete slabs.

A composite concrete slab consisting of a base and a bonded overlay is significantly stronger and stiffer than a composite slab with an unbonded overlay. The simple case with two equally thick layers with equal moduli of elasticity shows that the bonded system has twice the load-carrying capacity and four times the stiffness of the unbonded system (Fig. 1). A sufficiently good bond between base layer concrete and concrete overlay is mandatory for bonded overlays—not only to obtain monolithic action between the two concrete layers but also to promote crack control in the overlay and prevent the transport of water and detrimental substances in the interfacial zone between the two layers.

In tests comparing bonded and unbonded concrete overlays subjected to differential shrinkage (for example, shrinkage difference between the new-cast concrete overlay and the older concrete base), Carlswärd has shown that the crack patterns are quite different.[1] Bonded overlays exhibit several cracks with harmless crack width (less than 0.1 mm [0.004 in.]), while unbonded overlays exhibit few cracks with significant width of 1 mm (0.04 in.). The interface in the unbonded concrete overlay may be regarded as a horizontal crack plane. If or when the overlay cracks, water and salt will have access to the large areas of the concrete base or substrate already at the first crack.

Within the field of bonded concrete overlays, there is a specific term for losing bond and it is called debonding.[2] The use of the present participle (verb ending with–*ing*) indicates that it is a process—the initial bond vanishes to zero during a certain time. Debonding is considered as following the zip principle; it starts at the edges (where the shear stresses often have their maximum) and propagates toward the center parts. Shear stress concentrations at the edges in an overlaid concrete beam subjected to differential shrinkage is one example (Fig. 2, from Reference 3). Several factors may contribute to debonding, for example, poor workmanship or heavy vibrations during overlay production, and a severe environmental event.[4] Granju identified two fundamental causes of debonding: (1) those of mechanical origin, due to wheel loading; and (2) those of length-change origin, due to differential shrinkage or temperature changes.[5] And so, an important question must be asked: *Is debonding unavoidable—will bond disappear over time?*

In theory, this question can be answered neither conclusively nor generally; however, this article provides evidence that debonding can be postponed significantly if all work during concrete removal, substrate preparation, casting, and curing is carried out meticulously.

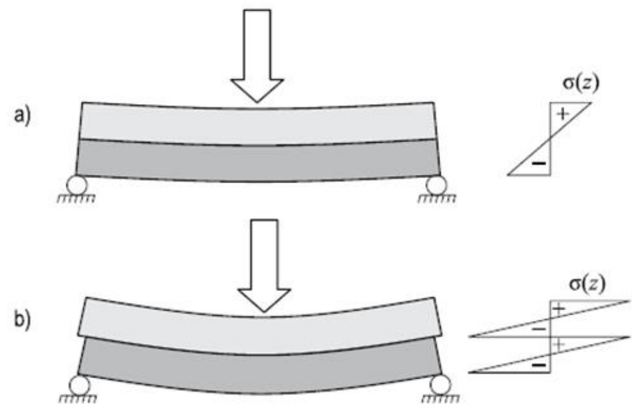


**Fig. 1: Deformation and normal stresses in composite beam: (a) with complete bond; and (b) without bond**

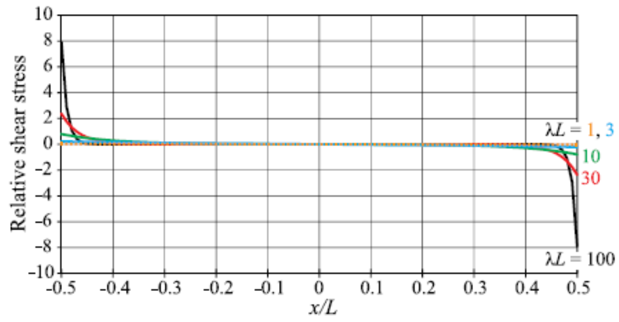


**Fig. 2: Relative shear stresses at the interface for a simply supported composite concrete beam as a function of relative length $x/L$ ($L$ = span length, $x$ = 0 at midspan) and the dimensionless parameter $\lambda L$ describing the bond between the layers ($\lambda L$ = 0 means no bond and $\lambda L \to \infty$ means complete bond)[3]**

## Factors Influencing Bond

The research on factors influencing bond between the base layer concrete (substrate) and a concrete overlay is very comprehensive, as described by Silfwerbrand.[6] To rank the various factors, Silfwerbrand and Beushausen defined three classes of importance: major, intermediate, and minor influencers (Fig. 3).[7]

Five factors were identified as major influencers:

- A substrate free of microcracking;
- A substrate free of laitance;
- A clean substrate surface during overlay placement;
- Sufficient compaction of the overlay; and
- Good curing during a sufficiently long period after placement.

Three factors were identified as intermediate influencers:

- Prewetting of the substrate;
- Overlay properties; and
- Time after overlay placement (age of overlay).

Factors identified as having minor importance included substrate properties and roughness, bonding agents, overlay placement method, early traffic, and fatigue and environment.

# Fatigue Testing

## Concrete beams with bonded concrete overlays

In the 1980s, the Swedish National Road Administration recognized that several bridges needed repair. Edge beams and concrete bridge decks were found to be more damaged than other parts of the bridges. The bridge deck was in turn found to be more damaged in the top half than in the bottom half. In many cases, the bottom part was in a rather good shape whereas the top contained cracks, spalling, corroded reinforcement, and chlorides from deicing salt. This called for a repair consisting of removal of damaged concrete, replacing corroded reinforcing bars, and placing a bonded concrete overlay.

Concrete bridge decks are subjected to high traffic loads, which means that the bond will be subjected to fatigue loading. At KTH Royal Institute of Technology, Stockholm, Sweden, a series of tests was carried out in the 1980s to investigate bond and fatigue. In these tests, 18 simply supported concrete beams were evaluated in three-point bending (Fig. 4) either by static or cyclic loading.[8] The tests were intended to reflect real conditions and, consequently, the base layer concrete was cut from the concrete bridge deck of Skurubron, in the Stockholm archipelago, originally opened for traffic in 1915. This was before waterjet technology (hydrodemolition) was introduced in Sweden for concrete removal as the first step in repairing concrete bridge decks.

One aim was to study the influence of the treatment of the substrate prior to overlay casting. Twelve beams cut from Skurubron were treated by pneumatic hammers. Additional measures such as dowel bars through the interface and epoxy bonding agent were applied to some of the test beams (Table 1). To investigate the effect of debonding, four beams were cast, each comprising two new layers cast at different times. In each beam, the interface between the concrete layers was smoothed out by steel trowelling. These beams were in addition to the original ones and did not have a base from Skurubron. Finally, the tests also contained two homogeneous reference beams constructed using the overlay concrete mixture over the full depth.

The tests were carried out on twin beams, one beam was statically loaded to failure and the other was subjected to up to 1 million cycles of load. After initial wet curing, the specimens were stored at room temperature and humidity in the laboratory until testing. At the time of testing, concrete compressive strength was measured using 150 mm (6 in.) test cubes. Compressive strength ranged between 56 and 64 MPa (8120 and 9280 psi) for the various concrete overlays. The strength of the 68-year-old concrete was 85 MPa (12,330 psi) based on compressive tests of 99 mm (4 in.) diameter drilled cores.

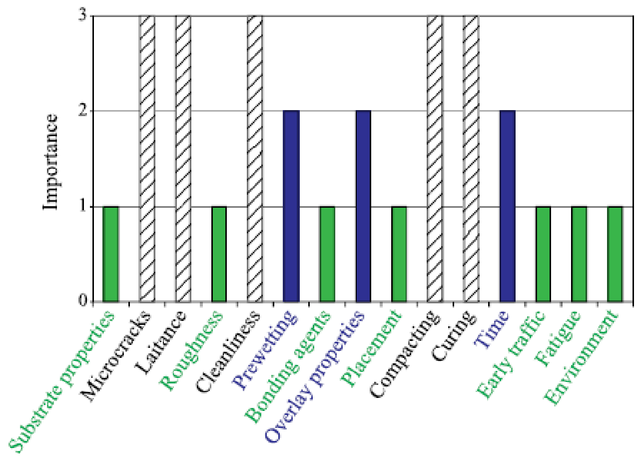


**Fig. 3: Ratings for factors influencing bond, listed in chronological order from left to right. A rating of 3, 2, or 1 indicates that the factor has a major, intermediate, or minor influence on bond, respectively (based on Reference 7)**

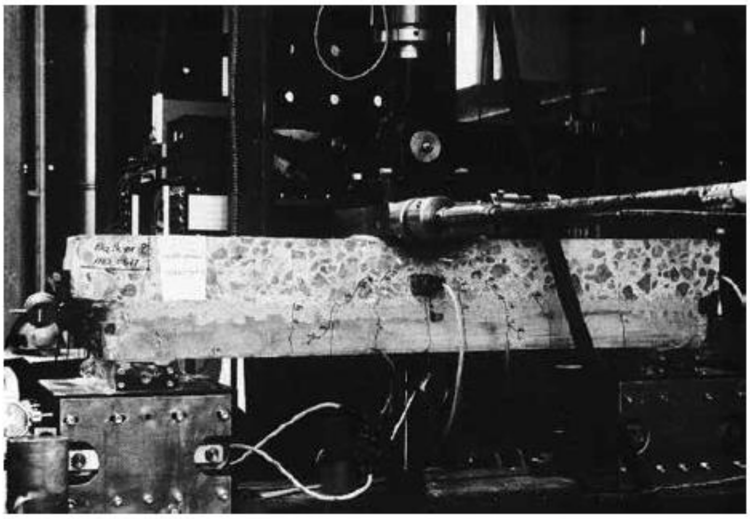

**Fig. 4: Test set-up for the two-layer concrete beam tests[8]**

## Table 1:
### Summary of fatigue tests on two-layer beams[8]

| No. | Old | New | Interface type | Dowels (number × diameter, mm) | Ultimate load of statically loaded twin beam, kN | Minimum/maximum load, kN | No. of cycles at failure | Failure mode |
|---|---|---|---|---|---|---|---|---|
| 8 | A | B2 | 1 | | 130 | 10/90 | 85,266 | Shear |
| 9 | A | B2 | 2 | | 150 | 10/90 | >10⁶ | |
| 10 | A | B2 | 1 | 8 × 7 | 200 | 10/90 | >10⁶ | |
| 11 | A | B2 | 1 | 8 × 10 | 235 | 10/90 | 812,907 | Shear |
| 12 | A | B2 | 1 | 8 × 12 | 235 | 10/90 | >10⁶ | |
| 13 | A | C2 | 1 | | 130 | 10/90 | 6681 | Shear |
| 14 | B2 | B2 | N/A | | 130 | 10/90 | 109,691 | Shear |
| 16 | E | F | 3 | | 70 | 10/40 | >10⁶ | |
| 18 | E | G | 3* | | 69 | 10/40 | >10⁶ | |

A is concrete cut from old bridge; B2 is concrete C50/60 cast in January 1983; C2 is vacuum treated concrete of type B2; E is concrete C50/60 cast in December 1983; F = G = concrete C50/60 cast in February 1984; 1 is interface treated with pneumatic hammer; 2 is interface same as 1 but with an additional epoxy bonding agent; 3 is steel-trowelled interface; 3* is interface same as 3 but with an additional grouting; No. 14 is homogeneous beam (Note: 1 mm = 0.04 in.; 1 kN = 225 lbf)

The fatigue tests were carried out in the laboratory using a sinusoidal load $F = 50 \pm 40$ kN (11,240 ± 8990 lbf) at frequencies varying between 1 and 4 Hz. The beams were tested with the overlay facing down and subjected to flexural tension (Fig. 4).

The most important result was that no interface failure occurred in any of the fatigue tests. In fact, shear failure occurred in four cases (including the homogeneous beam, No. 14). The remaining five beams withstood the entire fatigue program of 1 million load cycles (Table 1). The observed resistance of the interface was rather unprecedented because the maximum load applied during fatigue tests was high in relation to the failure load of the undowelled beams (90/130 ≈ 70% and 40/70 ≈ 60%). In design, the fatigue load will hardly be higher than 30% of the failure load. Assuming elastic conditions, the shear at the interface could be estimated as $(3/2)\times(F/2)/(bh)$ =1.7 MPa (250 psi) at $F = F_{max} = 90$ kN (20,230 lbf) and 0.75 MPa (110 psi) at $F_{max} = 40$ kN (8990 lbf) for $b = h = 200$ mm (8 in.).

In those cases where no failure occurred during the fatigue loading, the beams were subsequently loaded statically to failure. Beams with bonded interfaces exhibited full-depth shear failure (diagonal cracking), as shown in Fig. 5(a). Beams with smooth interfaces exhibited interface failure before shear failure in the lower layer, as shown in Fig. 5(b). This fact indicates that the surface roughness of the smooth interface resulting from steel trowelling might be lower than a threshold value for sufficient bond strength.

## Concrete beams with bonded shotcrete overlays

Traditional cast-in-place concrete overlays are suitable for repairing concrete bridge decks, concrete pavements, industrial concrete floors, and other top surfaces on horizontal structures. However, to use them on vertical and overhead surfaces, rather complicated formwork structures are necessary. Obtaining a good bond may also be difficult in these cases since we cannot benefit from gravity. Shotcrete becomes then a very good alternative.

Aiming at investigating the possibilities to repair concrete bridge decks with shotcrete, a second test series on beams (with shotcrete overlays) were carried out at KTH.[9,10] The test program consisted of the following steps:

1. Casting of four reinforced concrete slabs (Types A, B, C, and R);
2. Waterjetting of slab Types A, B, and C to different depths;
3. Shotcreting (dry mix) of Types A, B, and C to original slab depth;
4. Cutting three beams (denoted A1, A2, A3, etc.) from each slab;
5. Static loading (three-point bending) of one beam from each slab; and
6. Fatigue loading of the remaining eight beams (Fig. 6).

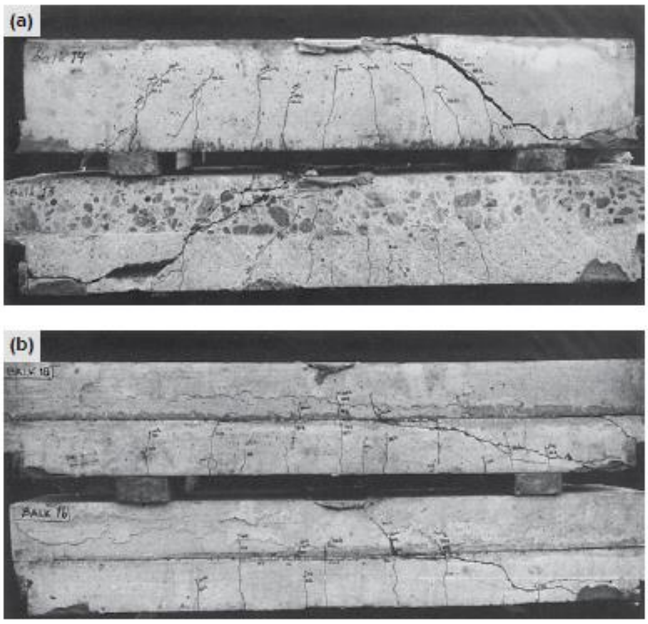


Fig. 5: Photographs from Reference 8, showing four of the test beams listed in Table 1, after fatigue loading and subsequent static loading to failure: (a) Beam No. 14 (top) and Beam No. 13 (bottom) developed shear cracks through the full depth; and (b) Beam No. 18 (top) and Beam No. 16 (bottom) developed shear cracks only through the lower (reinforced) half after horizontal cracks developed at the interface between layers

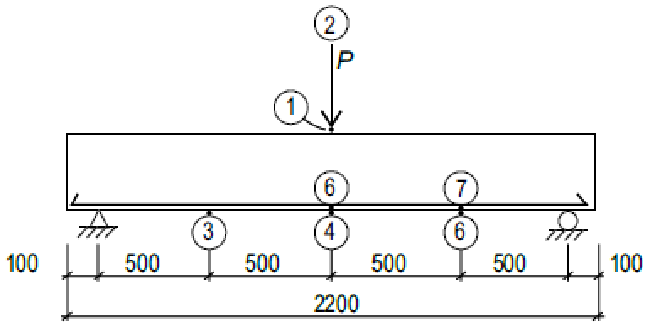


Fig. 6: Test set-up for fatigue loading of concrete beams with shotcrete overlay.[10] The encircled numbers 1 through 7 indicate measuring points (Note: 1 mm = 0.04 in.)

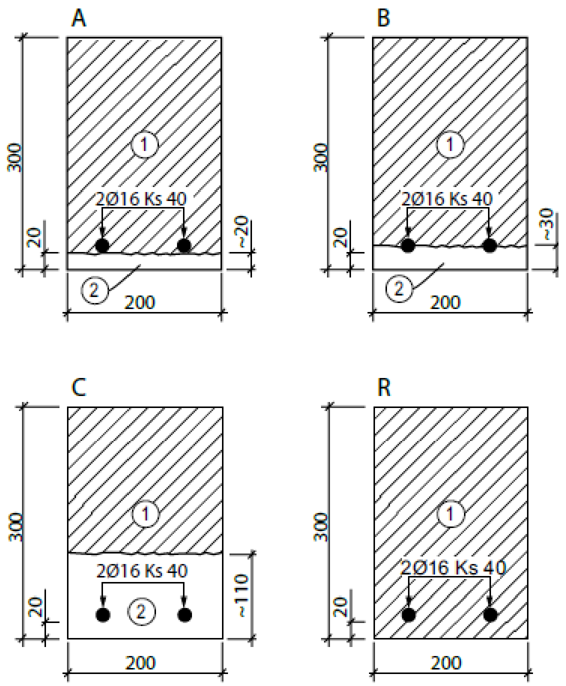


Fig. 7: Cross sections of test beams with shotcrete overlay.[10] The encircled numbers 1 and 2 indicate cast concrete and shotcrete overlay, respectively (Note: 1 mm = 0.04 in.)

As indicated in Steps 2 and 3, the primary test parameter was the location of the interface. The interface could be outside the flexural reinforcement (Type A), coinciding with the reinforcement plane (Type B), or markedly inside the reinforcement (Type C). These types were compared with a homogeneous reference slab (Type R), as shown in Fig. 7. At the time of testing, the base layer concrete had a compressive strength of 31.5 MPa (4570 psi) and the shotcrete layer concrete had a strength of 77.7 Mpa (11,270 psi).

The results of the tests were decisive; all Type A and C beams showed the same behavior as corresponding homogeneous reference beams (Type R). That is, the static failure loads and number of load cycles were of the same magnitude. The beams with a coinciding reinforcement and interface plane (Type B) had a brittle failure (combined shear and anchorage) instead of a ductile flexural failure. These beams also withstood only 1/10th of the number of load cycles at failure (Table 2). The recommendation drawn from these tests was that, if the reinforcement is uncovered during the concrete removal, the removal should continue past the reinforcement to enable the new concrete to surround the reinforcing bars. The free distance should exceed $d_{max}$ + 5 mm (0.2 in.), where $d_{max}$ is the maximum aggregate size of the new concrete or shotcrete to be applied.

**Table 2:**
**Summary of fatigue tests on beams with shotcrete overlay[10]**

| No. | Type | Average concrete removal, mm | Average uncovered/ total circumference of reinforcing bar, % | Ultimate load of statically loaded twin beam, kN | Minimum/ maximum load, kN | No. of cycles at failure | Failure mode |
|---|---|---|---|---|---|---|---|
| A1 | A | 22 | 2 | (A2:) 98 | 10/70 | 313,000 | Flexure |
| B1 | B | 30 | 23 | (B2:) 95 | 10/70 | 23,000 | Shear + anchorage |
| C1 | C | 110 | 100 | (C2:) 100 | 10/70 | 227,000 | Flexure |
| R3 | R | N/A | N/A | (R2:) 101 | 10/70 | 220,000 | Flexure |
| A3 | A | 22 | 2 | (A2:) 98 | 15/75 | 205,000 | Flexure |
| B3 | B | 30 | 23 | (B2:) 95 | 15/75 | 20,000 | Shear + anchorage |
| C3 | C | 110 | 100 | (C2:) 100 | 15/75 | 166,000 | Flexure |
| R1 | R | N/A | N/A | (R2:) 101 | 15/75 | 268,000 | Flexure |

Note: 1 mm = 0.04 in.; 1 kN = 225 lbf

## Long-Term Field Tests on Bond

In the mid-1980s, many Swedish concrete bridge decks were repaired using waterjetting and placement of overlays. KTH measured bond strength on some of these bridges both shortly after repair and 10 years later using pulloff tests.[11] The later tests were carried out in the immediate vicinity of the initial ones. The number of cores varied from bridge to bridge. Despite differences in structural system, climate zone, traffic, and use of deicing salt, the average failure stress $\sigma_f$ was found to have increased slightly in all seven cases (Table 3). The probability that higher values would occur in seven out of seven areas just as a coincidence is less than 1%.

**Table 3:**
**Long-term tests on bond strength $\sigma_f$ in repaired concrete bridge decks[11]**

| Bridge location | Structural system | Climate zone | Average daily traffic | Use of deicing salt | Repair year | $\sigma_f$ at time of repair, MPa | $\sigma_f$ in 1995, MPa |
|---|---|---|---|---|---|---|---|
| Bjurholm | Steel girders | 5 | 1440 | Occasional | 1985 | 1.96 | 1.99 |
| Mälsund | Concrete arch | 2 | 210 | Occasional | 1986 | 1.71 | 2.17 |
| Skellefteå | Steel girders | 4 | 25,000 | Moderate | 1987 | 1.43 | 1.82 |
| Södertälje | Flat slab | 2 | ~10,000 | Intensive | 1989 | >1.5 | 2.05 |
| Umeå | Steel girders | 4 | 23,000 | Moderate | 1987 | 1.56 | 1.61 |
| Vrena | Steel girders | 2 | 170 | Occasional | 1987 | 1.49 | 1.56 |
| Överboda | Concrete arch | 2 | 7600 | Intensive | 1986 | 2.09 | 2.18 |

Climate zone 2 has an annual frost amount of 300 to 600 days × degree of frost, zone 4 has 900 to 1200 days, and zone 5 has 1200 to 1500 days. 300 days × degree = 30 days of −10°C (14°F) or 60 days of −5°C (23°F) (Note: 1 MPa = 145 psi)

## Concluding Remarks

Bond strength is crucial for obtaining monolithic action between base layer concrete and concrete overlay. Differential shrinkage, fatigue from intensive traffic, and environmental actions such as frost and deicing salts may eventually cause horizontal cracking along the interface (debonding). Over 30 years of Swedish research and experience show, however, that debonding is not inevitable. Field tests demonstrate that the bond strength in repaired concrete bridge decks does not diminish during 10 years of service, and laboratory tests show that a proper interface withstands fatigue loading.

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Selected for reader interest by the editors.

# CASE STUDY

Reprint from CI Magazine, Volume 40, No 2 , Page 44-45

# Three-Sided High-Rise in Mexico City

## Torre Reforma provides a model for concrete construction in earthquake zones

With its distinctive triangular volume and towering concrete walls, Torre Reforma is an aesthetically pleasing addition to the skyline of Mexico City, Mexico (see Fig. 1). Arup worked with L. Benjamin Romano Arquitects (LBRA) to ensure that the 57-story mixed-use building was not only striking in appearance but also safe in its performance—a concern in the seismically active location.

"Arup has been indispensable in helping to transform my architectural vision into an efficient and buildable structure," said Benjamin Romano, Principal of LBRA."They have provided innovative solutions to the complex seismic issues in Mexico City and have been instrumental in helping the bidding contractors understand that Torre Reforma is not more complex than standard vertical construction; it just applies traditional construction methods, that contractors are already familiar with, in a new and different way."

Tabitha Tavolaro, Associate Principal at Arup and Project Manager for Torre Reforma, said, "Building tall structures in Mexico City often means working in constrained conditions. Challenges can include small or irregular sites, coordinating diverse teams, and, of course, seismic hazards. In this project, we partnered with LBRA to create robust solutions that bring value to the client as well as the community."

### An Aesthetic Structure

Romano's design departs from the norm not only in form but also in its merging of materials with structure.Arup devised pre-tensioned double-V hangers that brace the glazed façade and simultaneously create a signature visual identity for the building. Because the architecture is so

**Fig. 1: Torre Reforma, Mexico City, Mexico: (a) front elevation; and (b) the north façade**

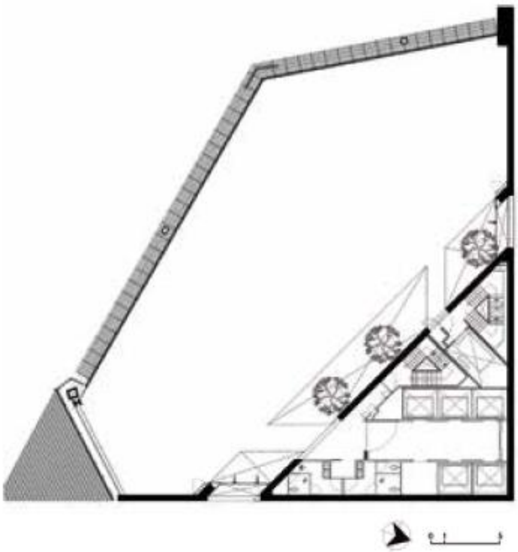

Fig. 2: A plan view of Level 41 at Torre Reforma. The north and east façades comprise perforated reinforced concrete shear walls


Fig. 3: Structural components of Torre Reforma: (a) shear wall formwork; and (b) atrium view of truss and perforated concrete shear wall

closely aligned with its material expression, the finish of the concrete is critical. Several design mixtures were evaluated; the final choice flowed well and created a surface that's free of honeycombing or flaws. The concrete was placed in increments of 700 mm (28 in.), highlighting the subtle variations in color that occur naturally across multiple concrete placements.

## The Core in the Corner

In conventional skyscrapers, vertical circulation is typically located in the central core of the building. At Torre Reforma, the elevators and egress stairways are contained in the apex of the triangle (see Fig. 2). This, paired with the long-span pyramidal floor trusses that allow plumbing, electrical, and mechanical systems to be concealed within the structure, results in maximum ceiling heights and a column-free interior, facilitating unobstructed, dramatic views over adjacent Chapultepec Park and the city from every level.

## Designed for Stability

Set in an area with a long history of significant seismic activity, skyscraper construction in Mexico City poses complicated engineering challenges. Because Torre Reforma is triangular in plan, the building has an inherent tendency to twist when subjected to wind or earthquake. Arup applied a comprehensive time-history analysis to establish the performance of the structure under extreme seismic conditions and engineered a solution that is both locally appropriate and consistent with international best-practice designs for tall buildings (Fig. 3). Torre Reforma will be able to withstand the full range of earthquake activity projected for a period of 2500 years. According to Tabitha Tavolaro: "The building resisted the September 7th and September 19th seismic events with zero observable structural damage."

## Green Goals

In addition to its structural innovations, the building offers extensive sustainability features. Pre-certified as a LEED Platinum Core and Shell project, Torre Reforma has multiple water conservation systems, including rainwater collection and gray- and black-water recycling plants. Interior temperatures are moderated using a combination of automated and passive ventilation. The tower's two reinforced concrete façade walls also contribute to the energy efficiency of the building—reducing the cooling load by protecting the interior from direct sun and providing thermal mass to modulate diurnal temperatures. —Arup, **www.arup.com**.

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