Performance of Unbonded Concrete Overlay Project in Canada

T. J. KAZMIEROWSKI AND HARRY STURM

The rehabilitation of concrete freeways has challenged highway authorities for many years. This challenge has been met with the use of various techniques such as full- and partial-depth repairs, diamond grinding, and unbonded overlays. In 1989 the Ontario Ministry of Transportation constructed a project to demonstrate the feasibility of using these techniques on the freeways in its jurisdiction and to monitor the long-term performance of the freeways rehabilitated by these various techniques. The project incorporated the use of full-depth and partial-depth repairs, which was followed by diamond grinding and joint sealant replacement on the northbound lanes. The southbound lanes received a 180-mm-thick undoweled plain-jointed unbonded portland concrete cement overlay, which is the focus of this study. Design of the overlay was undertaken by using PCA design guides, which used the familiar Corps of Engineers square root equation. During the design phase falling-weight deflectometer (FWD) testing was undertaken to assess the pavement condition. Following construction an extensive program was implemented to systematically monitor long-term performance. The program included FWD testing for corner-to-center deflection ratios and joint load transfer efficiencies, along with roughness and skid resistance testing and condition surveys. Roughness and skid resistance measurements have been completed on an annual basis, with FWD testing and condition surveys carried out as needed. The design and construction of the unbonded overlay are highlighted, and the results of the performance monitoring program carried out on this 4-year-old project are detailed.

The rehabilitation of concrete pavements has undergone significant development in recent history and has provided options that were previously not realized. One such technique is the use of unbonded concrete overlays. Construction of a concrete overlay is not significantly different from that of conventional concrete pavements; however, design of these structures has not been as thoroughly researched as conventional designs.

The project described here was initiated to assess the feasibility of constructing a concrete overlay and to monitor the long-term performance of such a design in the wet-freeze environment of Ontario, Canada.

BACKGROUND

Highway 126, now Highbury Avenue following transfer to the city of London, is approximately 200 km west of Toronto. The road is a 4-lane divided arterial, with volumes ranging from 22,000 to 30,000 average annual daily traffic in 1992, of which 3,000 are commercial vehicles.

T. J. Kazmierowski, Pavement Design, Evaluation and Management Section, Ministry of Transportation of Ontario, 1201 Wilson Avenue, Downsview, Ontario M3M 1J8, Canada. H. Sturm, Ontario Region, Canadian Portland Cement Association, 1500 Don Mills Road, Suite 501, Toronto, Ontario M3B 3K4, Canada.

The original pavement was constructed in 1963 with 225 mm of mesh-reinforced portland concrete cement (PCC) on 300 mm of granular base and subbase materials placed full width. The pavement was constructed with a 21.3-m joint spacing and load transfer dowel bars. The joints were sealed with preformed neoprene seals. The pavement on the southbound lanes (SBLs) showed significant distress, with full- and partial-width transverse cracks in each slab, severe D-cracking, spalling of all joints throughout, and moderate pavement edge distress. Deterioration had advanced to the stage in which routine maintenance activities could no longer preserve a safe riding service.

The northbound lanes (NBLs) were in much better condition and as such were rehabilitated by using full- and partial-depth concrete repairs and diamond grinding.

The variation in pavement performance was attributed to the two types of aggregate used in the concrete for the opposing lanes. The quarried aggregate used on the SBLs exhibited absorptive characteristics and was highly susceptible to D-cracking under freeze/thaw conditions. A pit run gravel source was used for the coarse aggregate on the NBLs.

Three rehabilitation strategies were considered for use on the SBLs and included crack and seat with an asphalt overlay, repair of the PCC pavement with hot mix and placement of a multiple-lift asphalt overlay, and overlay with concrete. Because of the extensive deterioration of the concrete on the SBLs it was believed that the unbonded concrete overlay would provide the highest level of long-term performance and would provide the opportunity to assess this type of rehabilitation technique.

DESIGN

The thickness design of the overlay was completed by using PCA guidelines, which used the Corps of Engineers square root equation. This formula requires the user to determine the thickness of a single slab, from which the existing pavement thickness is deducted after correcting for the condition and deterioration. In this case it was determined that a slab thickness of between 220 and 240 mm would be required in an initial design. By using an existing pavement condition coefficient of 0.35 for badly cracked pavements, the existing slab was deducted to arrive at an overlay thickness of 180 to 200 mm. An overlay thickness of 180 mm was selected. The selection of a condition coefficient has a significant influence on the overlay thickness. By using a condition coefficient for a pavement with corner cracking as the major distress, an overlay thickness of 115 mm is required; if an intermediate value of 0.55 is input an overlay thickness of 160 mm is estimated. This indicates that selection of this coefficient is critical in determining the overlay thickness. Correlation of the coefficient with nondestructive test [falling-weight deflectometer (FWD)] information as well as visual data would be of value for future designs.

The design included a bond breaker of compacted asphaltic sand hot mix with a maximum aggregate size of 4.75 mm. This was placed to prevent bonding between the new and old PCC pavements and minimize reflection cracking in the overlay. Other design features were the use of undoweled skewed randomized joints, short slab lengths, a burlap drag/tined surface texture, and improved joint sealing procedures. Typical lane widths were 3.65 m for a total width of 7.3 m in the two lane sections. Tie bars were specified in the longitudinal joints. The 7.3-m-wide overlay typically matched the width of the underlying concrete pavement.

The addition of partial-width concrete shoulders was evaluated during design; however, the decision was made not to incorporate them. One reason for this decision was the underlying rigid nonerodible base, which would provide significant edge support and minimize the risk of long-term faulting. It was concluded that performance would be adequate without concrete shoulders, and the additional expense could not be justified. Typical sections and other design details have been provided previously (*1*).

Drainage of the existing and proposed pavement structure was evaluated, and investigations indicated that the existing full-width granular materials were providing adequate drainage. This was further substantiated by the absence of any joint or crack faulting or staining. On the basis of this assessment no retrofitted drainage measures were incorporated into the rehabilitation design.

CONSTRUCTION

Before placement of the concrete overlay, loosely spalled materials were removed by hand from the existing badly cracked pavement. The spalled areas (427 m^2) were then patched with conventional hot mix, and a compacted 20-mm asphaltic sand mix bond breaker (884 metric tons) was placed over the existing pavement surface. Following placement of the bond breaker, the 180-mm-thick unreinforced plain-jointed concrete pavement (38,200 m^2) was placed. This was the first concrete highway pavement to be placed in Ontario since 1982. Slipforming of the unbonded overlay was carried out by using Ontario's revised specifications and drawings for concrete pavement. The revisions were based on experience gained on the experimental concrete pavement sections on Highway 3N (*2,3*).

Construction of the unbonded concrete overlay went smoothly following a few initial breakdowns with the slip-form paver. The two lanes were slipformed simultaneously, with longitudinal tie bars being inserted every 600 mm on the center line. The plastic concrete surface was dragged longitudinally with a burlap mat and tined transversely to achieve the specified surface texture. Speed change lanes were then slipformed and tied onto the outside of the core lanes when required.

Skewed joints were sawcut into the overlay at random intervals that varied from 3.7 to 5.8 m. No uncontrolled transverse cracking of the pavement occurred with revision of the specification to require sawcutting within 12 hr after placement rather than the 24 hr specified previously. No effort was made to match or avoid matching the original pavement joints.

The pavement ride was somewhat better than that in the previous slipforming project, which was in part because of a requirement for stringline control on both sides of the slip-form paver. Pavement ride is discussed in further detail in the following section.

PAVEMENT PERFORMANCE

Roughness Measurements

The motoring public is generally most concerned with the ride of a pavement; therefore, monitoring of ride condition or roughness was initiated. Roughness was measured just after construction and before opening to traffic and has been measured yearly since construction. The California profilograph was used immediately after construction, and the portable universal roughness device (PURD) was used for ongoing monitoring.

The profilograph measurements were taken on the new concrete overlay, with readings taken in both wheelpaths of the driving and passing lanes. The average profilograph reading, the profile index, for the length of the project was 11.1 in./mi in the driving lane and 10.7 in./mi in the passing lane. These readings were taken before diamond grinding at 21 locations where the roadway did not meet the surface tolerance specifications of 3 mm in 3 m. These values compare reasonably well with the numbers obtained by other authorities constructing considerably larger quantities of concrete pavement. Postgrinding profilograph measurements were not completed because a ride specification was not being used at the time.

Ongoing postconstruction roughness measurements have been completed by using the PURD, a trailer-mounted, accelerometer-based, response-type device. It is operated at constant highway speed and uses the root mean square vertical acceleration, in milli*g*'s, of the trailer axle to measure roughness. PURD measurements are converted into a ride comfort rating (RCR) by using a transfer function for application in Ontario's Pavement Management system. RCR is based on a scale from 0 to 10, with 10 being a smooth and pleasant ride.

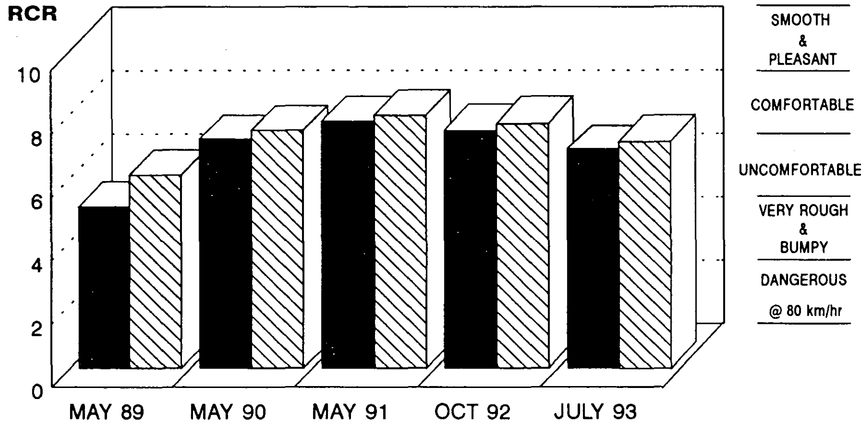
An RCR level of 7.7 to 7.8 for the driving and passing lanes, respectively, was achieved following construction, which is only slightly rougher than generally attained on a new asphalt surface. Since construction the ride values have shown little change and are now at 7.5 and 7.7 for the driving and passing lanes, respectively (see Figure 1 for details). This is a relatively small change over 4 years and would appear to point to acceptable short-term performance.

Load Response Characteristics

A nondestructive load testing program was set up to evaluate the condition of the pavement before rehabilitation and to assess long-term performance of the project. Before rehabilitation slabs were randomly selected for testing of corner-to-center slab deflection ratio and load transfer efficiency across joints. This information was used in the design to determine the presence of voids beneath the slabs and to establish the need for stabilization of the joints.

Deflection testing was completed by using the Dynatest 8000 FWD. Most testing was performed early in the day to minimize the influence of slab curl resulting from differential slab temperatures.

After rehabilitation deflection testing was undertaken at the same locations or at the nearest point to the preconstruction test-



	MAY 89	MAY 90	MAY 91	OCT 92	JULY 93
SDL	5.1	7.7	7.9	7.8	7.5
SPL	6.1	7.8	8	7.9	7.7

FIGURE 1 Summary of PURD roughness survey.

ing. Testing was again undertaken in the summers of 1992 and 1993 to evaluate performance of the overlay. Postconstruction deflection testing yielded some significant improvements in corner-to-center deflection ratios and a slight improvement in load transfer efficiencies over the original pavement as noted previously (*4*). Data from the 4-year monitoring program have complemented information from the more limited program completed in year 3 and substantiates the visual performance of the pavement.

The load transfer results showed very high values, with mean values in the 95 to 97 percent range (Figure 2). The range of values has remained constant after exposure to traffic. The old concrete pavement, which is functioning as a base for the overlay, appears to be providing necessary support for the undoweled joints that have not lost aggregate interlock. In similar pavements constructed on lean concrete bases on the Highway 3N test project (*5*), load transfer efficiencies had generally fallen to below 60 percent after 3 years of service. In a thicker (300-mm) section built on subgrade, the load transfer had fallen to 80 percent.

The resulting increase in pavement strength after overlay placement was also affirmed by the corner-to-center deflection ratios, with mean values of 1.8 and 1.3 in the driving and passing lanes, respectively (Figure 3). These are low values and would indicate that the overlay has a high level of stiffness and is well seated on the underlying concrete base. The high load transfer efficiencies also contribute to the stiffness of the slab and ensure low deflection ratios. Performance numbers achieved on this project are similar to the center edge deflection ratios measured on 200- to 325-mm continuously reinforced concrete pavements constructed

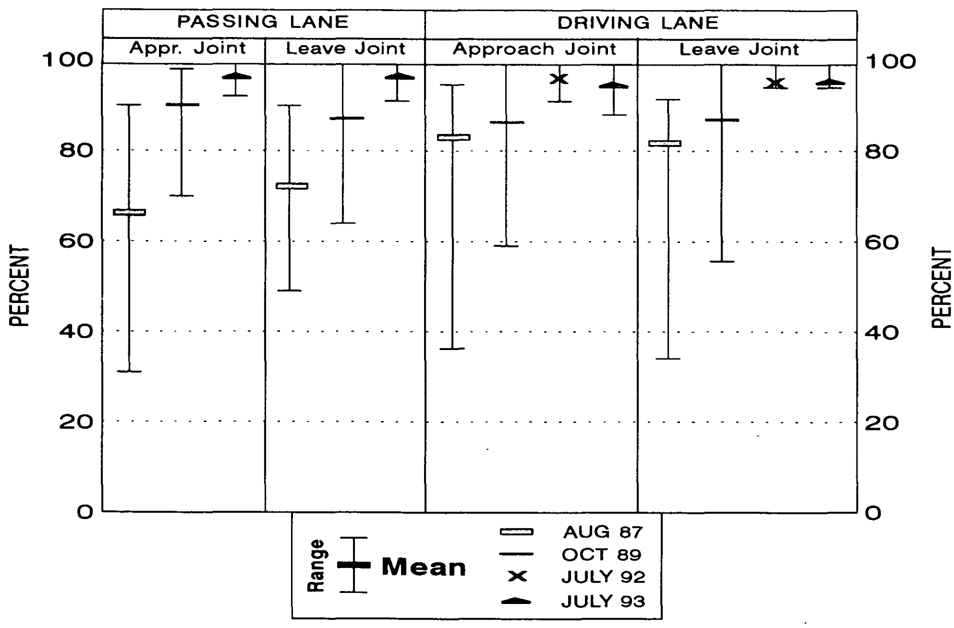


FIGURE 2 Load transfer results.

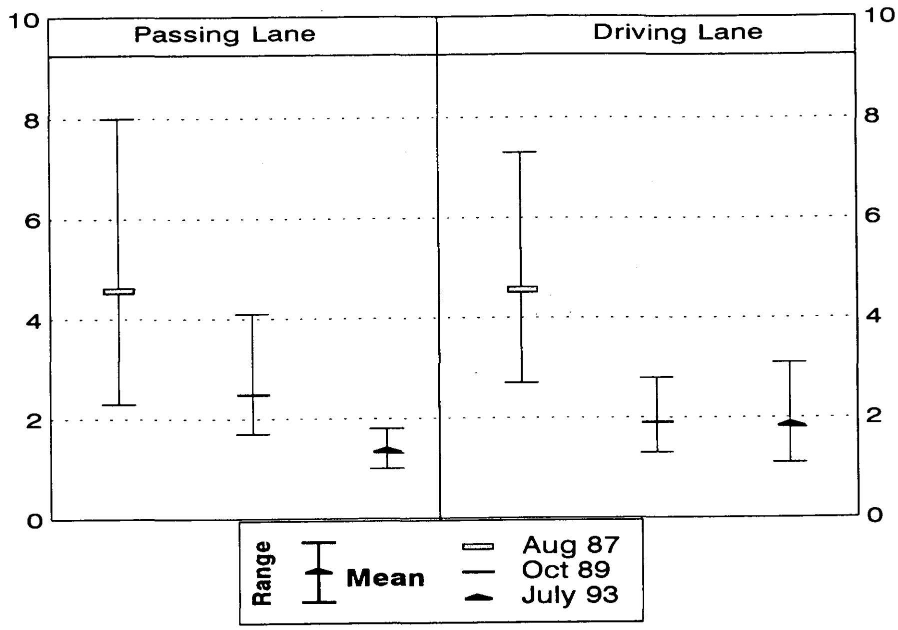


FIGURE 3 **Corner-to-center deflection ratios.**

in Texas (*6*). Center-to-edge deflection ratios ranged from 1.2 to 1.8 for asphalt shoulder sections in that project.

In 1993 center deflection measurements yielded mean values in the range of 0.03 and 0.05 mm for the driving and passing lanes, respectively. These values again indicate a very stiff and stable slab and compare favorably with numbers on the order of 0.04 to 0.05 for other concrete pavements of a similar age that are generally equal in effective thickness. These center deflection values have not changed significantly from the postconstruction mean values of 0.04 for both lanes in 1989.

Pavement Distress Survey

The visible distresses in the pavement surface were surveyed to provide an overall indication of performance. Generally a high level of performance has been maintained with the unbonded overlay. The only distress observed is very slight longitudinal cracking in 11 driving lane slabs. This cracking can probably be attributed to inadequate longitudinal joint construction because the cracks were visible soon after construction. One of the cracked slabs is now beginning to exhibit more severe distress in the form of spalling along the crack face where it crosses the right wheelpath.

No transverse or reflection cracking was observed, which correlates with the experiences of other agencies (*7*). Some transverse cracking is anticipated in the future because the longest slabs in the randomized pattern are 5.5 and 5.8 m, which are somewhat longer than the recommended maximum of 4.6 m. No faulting of joints has been observed to date.

Skid Testing

Skid resistance was measured by using the skid trailer conforming to ASTM Standard E274. The measured friction force is described as a skid number (SN) at a specific speed; for example SN 80 is the skid number at 80 km/hr.

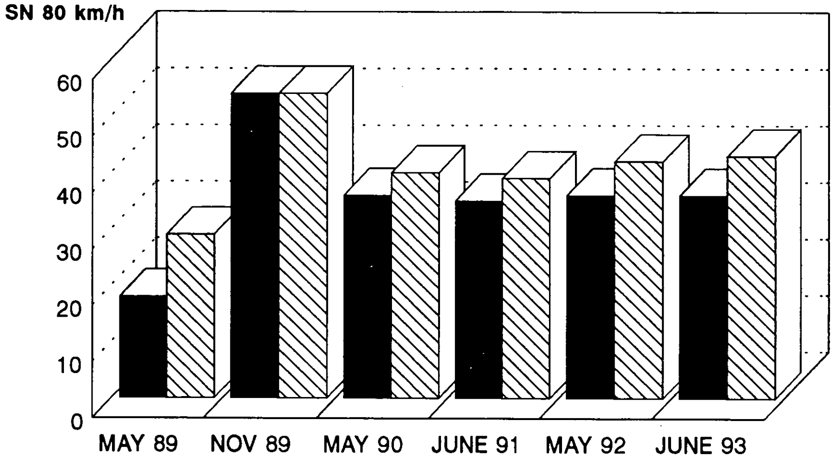
Results of skid resistance testing are detailed in Figure 4. The test results indicate a substantial increase in skid resistance immediately after rehabilitation, as one would expect. The skid numbers in the SBLs increased to an average SN 80 of 54 for both lanes from SN 80 values of 18 and 21 in the driving and passing lanes before rehabilitation, respectively. Skid numbers of this magnitude indicate a good friction factor and are similar to numbers obtained on premium hot-mix surface courses (*8*).

As anticipated, after the first year the initial sharp projections of the drag/tined surface have been worn, polished, or sheared off by traffic and snow removal operations. This elimination of sharp projections reduced SN 80 values from values in the mid-50s to the values in the mid-30s for the driving lane and low 40s for the passing lane. On the basis of previous experience this trend was anticipated. However, it would appear that skid resistance levels have stabilized at these acceptable levels for the present time.

CONCLUSIONS

The 4-year performance review of this unbonded concrete overlay project warrants the following general conclusions.

1. Initial pavement roughness was somewhat better than that experienced on previously constructed concrete pavements in Ontario. Subsequent PURD readings indicate that an acceptable level of ride is being maintained.
2. Load deflection testing verifies the substantial increase in pavement section strength with low deflections and high load transfer efficiencies across all joints.
3. The distress surveys have indicated no cracking other than the few slight longitudinal cracks observed immediately after construction. No transverse cracking has been observed in the longer 5.5- and 5.8-m slabs.



SDL ■	18	54	36	35	36	36
SPL ▧	29	54	40	39	42	43

FIGURE 4 Summary of skid survey.

4. Skid resistance has stabilized at an acceptable level and is displaying performance similar to other exposed concrete pavement surfaces in Canada.

5. Overall the 4-year results indicate that a high level of performance has been maintained. Mid- to long-term performance results will continue to be monitored. The short-term results support the adequacy of the design procedure used.

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