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Material Investigation of the Full-Depth, Precast Concrete Deck Panels of the Old Woodrow Wilson Bridge

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16. Abstract

The Woodrow Wilson Memorial Bridge crossing the Potomac River near Washington, D.C., was replaced after more than 45 years of service. Researchers examined the full-depth, precast lightweight concrete deck panels that were installed on this structure in 1983. This report covers the visual survey and concrete material tests from this investigation.

The concrete deck appeared to be in good condition overall, with no discernible cracks or signs of impending spalls on the top surface, except for a few signs of distress evidenced by asphalt patches. From below the deck, there were some indications of efflorescence and some panel joints exhibited rust staining, efflorescence, and small pop-out spalls. Closure pours for the expansion joints had more severe corrosion and efflorescence. Steel bearing plates and hold-down rods used for panel-to-deck connections were generally in good condition, although there were the occasional elements that rated poorly.

The concrete sampled from the lightweight precast deck panels had an average compressive strength of 7.01 ksi (48.3 MPa), which represented little increase over the average 28-day strength. The average elastic modulus was 2,960 ksi (20.4 GPa), which is on the low end for typical modern concrete mixtures. The average splitting tensile strength was within a typical strength range at 535 psi (3.67 MPa). The average equilibrium unit weight of the plain concrete was 116.5 lb/ft^3 (1866 kg/m^3). The concrete was sound with no evidence of cracking or other deleterious reactions. The results of absorption, permeability, and chloride tests indicated a material matrix with the capability of absorbing moisture and other contaminants. An epoxy concrete surface layer, an asphaltic concrete wearing surface, and cover depths greater than 2 in seemed to have limited harmful chloride exposure to the reinforcing steel, which appeared to be in good condition.

The full-depth, precast lightweight concrete panels appeared to have performed well, with few maintenance issues observed. Reports of similar, more recent, projects have noted additional direct costs associated with precast deck systems on the order of $26 to $30 per square foot. However, anecdotal information from those projects, as well as an analysis of the construction alternatives presented herein, demonstrates that use of precast deck systems for deck replacement of existing bridges can shorten construction time by several weeks or months and induce far less disruption to travel than the conventional cast-in-place alternative, resulting in a dramatic reduction in user costs. When total life-cycle costs, including those associated with road user costs, construction time, construction safety, and maintenance, are taken into account full-depth precast concrete deck panels are the more economical alternative.

The costs and benefits assessment demonstrated a clear advantage to using precast bridge deck technology for select deck rehabilitation projects. However, the nature of the estimates and the infrequency with which this sort of repair is implemented make it unreasonable to attribute a direct value in annual savings.

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ABSTRACT

The Woodrow Wilson Memorial Bridge crossing the Potomac River near Washington, D.C., was replaced after more than 45 years of service. Researchers examined the full-depth, precast lightweight concrete deck panels that were installed on this structure in 1983. This report covers the visual survey and concrete material tests from this investigation.

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INTRODUCTION

Since 2001, the Federal Highway Administration (FHWA) guidelines for roadway and bridge construction and maintenance have been encouraging the policy of "get in, get out, stay out" in terms of limiting disruptions to local traffic flow (Keever et al., 2001). This policy can be executed with a three-pronged strategy: (1) quickly starting the construction process once the design concepts have been approved; (2) using accelerated construction techniques such as precast concrete elements and self-propelled modular transports; and (3) selecting high-quality materials, such as high-performance steel, high-performance concrete, and fiber-reinforced polymers as well as using enhanced design specifications, to ensure that the structure will last for many years and require minimal maintenance (Gee, 2003).

Perhaps 20 years before its time, the Woodrow Wilson Memorial Bridge (Woodrow Wilson Bridge) spanning the Potomac River and connecting Maryland, Virginia, and Washington, D.C., was already a major example of the "get in, get out, stay out" concept when it was renovated in the early 1980s. As is true today, the Woodrow Wilson Bridge carried I-95/495, a major north-south corridor along the east coast of the United States. This portion of the corridor had become overburdened with traffic, thus becoming a choke point in the regional transportation system. Therefore, FHWA, the Maryland State Highway Administration, the Virginia Department of Transportation (VDOT), and the District of Columbia Department of Transportation elected to widen the bridge after 20 years of service. In doing so, the engineers opted to employ some innovative materials and construction techniques to avoid disrupting the traffic flow and to minimize future maintenance needs. One such combination of material and technique was the use of lightweight concrete in full-depth, precast deck panels (Lutz and Scalia, 1984).

However, traffic volume had once again exceeded the bridge's capacity by the end of the century. Therefore, transportation authorities approved plans to replace the existing structure. This replacement has afforded researchers the opportunity to retrieve portions of the lightweight precast deck system and examine them after 23 years of performance. In a collaborative effort among the Virginia Transportation Research Council (VTRC), Virginia Tech, and Pennsylvania State University, a team of scientists visited the bridge site to perform a visual inspection of the deck panels and retrieve samples of the deck. The samples were obtained for structural and material testing of the deck elements. This report focuses on the concrete material testing carried out by the VTRC in this collaborative effort.

PURPOSE AND SCOPE

The purpose of this project was to conduct a material analysis of the lightweight concrete in the full-depth, precast deck panels that had been in service for 23 years in the Woodrow Wilson Bridge. This study was carried out in conjunction with the work of Virginia Tech and Pennsylvania State University, where structural tests and additional material tests determined the long-term behavior of the post-tensioning systems in the decks.

The scope of the study entailed obtaining 16 concrete core samples at two locations on the western portion of the bridge: an area of deck where there were visible signs of distress, and an area where the concrete appeared to be in good condition. Various tests on these core samples determined the following properties of the concrete:

- elastic modulus
- compressive strength
- splitting tensile strength
- density
- absorption rate
- resistance to rapid chloride penetration
- chloride content.

In addition, a petrographic analysis helped to describe the air voids and microcracking in the concrete and the quality of the constituent materials.

LITERATURE REVIEW

Typically, lightweight concrete weighs 115 to 120 lb/ft^3 (1840 to 1,920 kg/m^3), which is less than the 145 to 150 lb/ft^3 (2325 to 2,405 kg/m^3) unit weight of normal weight concrete. The weight reduction comes from the coarse and/or fine aggregate, which is typically made from expanded slate, shale or clay. The lower unit weight results in a reduced dead load in the superstructure when compared to that with normal weight concrete, allowing for an increased live load or a smaller substructure and extending the functional life of an existing bridge. Beyond the benefit of a reduced dead load, lightweight concrete exhibits a lower modulus of elasticity, a more continuous contact zone between the aggregate and the paste, and additional moisture in the aggregate pores for internal curing (Holm, 1994). These characteristics result in fewer cracks in the concrete and inhibit water ingress that can be particularly detrimental to bridge decks (Neville, 1996).

Generally, prefabricated bridge elements allow techniques to minimize construction impacts on traffic flow and harm to the environment at the job site, improving work zone safety and providing more structurally efficient and constructible designs (Berger, 1982; FHWA, 2002). All of these factors may lead to a higher quality design with a greater probability that the structure will last longer than with traditional cast-in-place construction methods. Specifically, full-depth, precast concrete deck panels help achieve these goals by eliminating the need for erecting formwork and enabling rapid installation on top of beams through the use of grouted pockets around the shear connectors extending from the beams (FHWA, 2004).

The original Woodrow Wilson Bridge, shown in Figure 1, was completed in 1962 along I-95/495 at the point where the boundaries for Maryland, Virginia, and Washington, D.C., meet, as shown in Figure 2. The bridge was 5,900 ft (1798 m) long and had a 212-ft (64.6-m) double-leaf bascule span over the river channel. The noncomposite, reinforced concrete deck system for the approach spans was supported by steel stringer-floor beam construction on four-girder, continuous-span units, except for the three spans on either side of the bascule, which had stringers and floor beams supported by three-girder simple-span units (Lutz and Scalia, 1984). At 89 ft (27.1 m) wide, the deck supported three lanes going in each direction, separated by a median barrier, and a 3-ft (0.91-m) safety walk. The concrete deck had a 2-in (51-mm) asphaltic concrete wearing surface on top (Lutz and Scalia, 1984).

When the time came to increase the vehicle capacity of the bridge, regional authorities realized disruption in service would have a significant impact on transportation along the I-95/495 corridor. Therefore, construction had to take place in a manner that would not significantly hinder traffic flow and would minimize future maintenance needs. Hence, criteria for the bridge expansion included adding a lane in each direction while maintaining the original six lanes of traffic during peak travel periods and providing one lane in each direction during the night-time hours (Lutz and Scalia, 1984). Further, modifications to the existing steel superstructure and the number of roadway joints were to be kept to a minimum.

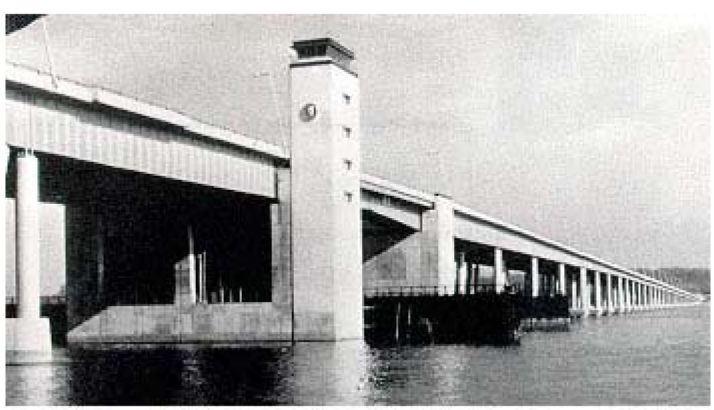

Figure 1. Old Woodrow Wilson Memorial Bridge in 1961. Source: Federal Highway Administration. FHWA By Day. http://www.fhwa.dot.gov/byday/fhbd1228.htm.

Figure 2. Location of Bridge

Precast Deck Panel Design

In keeping with these requirements, the design called for a system of full-depth precast concrete deck panels that were one-half the width of the entire bridge, or approximately 46.6 ft (14.2 m) wide, with about 8 ft (2.47 m) of each panel cantilevering beyond the exterior girder. This panel width essentially added an emergency shoulder lane for vehicular accidents and breakdowns that frequently impeded traffic flow. These panels were generally 10 to 12 ft (3.05 to 3.66 m) long and 8 in (203 mm) thick, with an additional 5 in (127 mm) for the thickness of the haunch at the exterior girder.

To retain the original superstructure, the panels were post-tensioned in both the transverse and longitudinal directions and were fabricated with lightweight concrete. While the lightweight concrete reduced the dead load on the superstructure, the post-tensioning provided extra stiffness for the cantilevered portion and crack control. The post-tensioning strands were sheathed in plastic with a PT coating (grease) around the strand, and the ducts for multiple strands in the longitudinal direction were grouted. A rapid-setting, high-strength methyl methacrylate polymer concrete was placed between the adjoining ends of the deck panels prior to the longitudinal post-tensioning. The same type of concrete was placed into formed bearing pad closures between the panels and the steel stringers (see Figure 3). There was no composite action between the deck and the stringers, as the polymer concrete bearing pad was cast onto a steel plate with short welded studs that was able to slide along the top flange of the stringer. A pair of hold-down rods passed through holes in the precast panel and were tightened against a retainer bar on the underside of the bottom flange of the girder, as shown in Figure 3. The holes for the hold-down rods were filled with the same aforementioned polymer concrete.

Precast Deck Materials

Believed to be one of the first applications of lightweight concrete in full-depth, precast concrete deck panels, the precast concrete mixture design for the Woodrow Wilson Bridge called for a dry unit weight of 115 lb/ft^3 (1840 kg/m^3), a 28-day compressive strength of 5,000 psi (34.5 MPa), and a maximum water-cement ratio of 0.44. The coarse aggregate was expanded slate provided by Carolina Stalite Company, which is known for its low absorption rate and high-

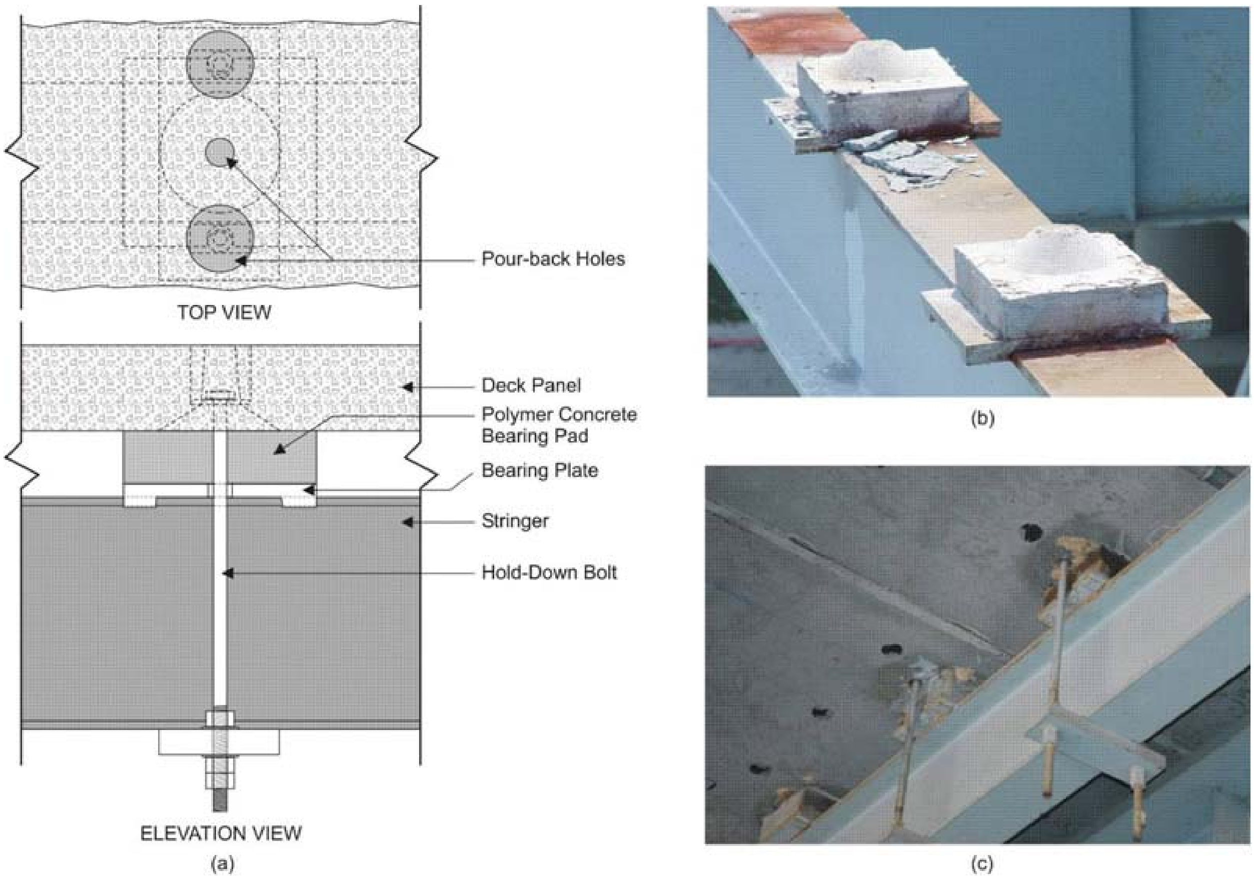

Figure 3. Section of Precast Deck to Steel Girder Connection, Showing (a) Schematic of Entire Connection, (b) Cast Polymer Concrete Bearing Pad, and (c) Hold-Down Rods and Retainer Bar

quality physical properties. The fine aggregate was manufactured limestone sand. The average results for unit weight and 28-day compressive strength were 115.7 lb/ft^3 (1854 kg/m^3) and 6,570 psi (45.3 MPa), respectively. After curing, the panels were coated with two layers of sand-epoxy overlay, which primarily helped to protect the panels from deicing salts and provided temporary skid resistance until the asphalt wearing surface had been applied (Lutz and Scalia, 1984).

Precast Deck Construction

A typical construction cycle involved removing slightly more old deck than would be replaced with the new deck panels. Once the deck was placed and leveled, construction crews placed the polymer concrete in the closure pours and then set the parapets after the polymer concrete reached a compressive strength of 4,000 psi (27.6 MPa). Prior to rush hour, workers placed steel grates to bridge any gaps between the existing and new deck for motorists to travel over. On the next night, the grates would be removed, and the cycle would begin anew. On average, each setting crew placed 5 panels per night, with the entire project requiring 129 nights to install 1,026 precast panels. Normally, a job of this magnitude that replaced the entire deck system on a six-lane bridge more than 1 mile (1.6 km) long would require closing half of the lanes at a time on the bridge for more than 1 year. However, the Woodrow Wilson Bridge redecking project was finished in 8 months, and traffic was maintained in all lanes during peak hours (“A Quick Switch in the Dark,” 1983).

METHODOLOGY

Visual Inspection

On June 15, 2006, and again on July 13, 2006, researchers from VTRC, Virginia Tech, and Pennsylvania State University convened at the Woodrow Wilson Bridge to conduct a visual inspection of the deck panels along the entire length of the western portion of the bridge. Teams paired off into groups of two, with each group tasked to inspect the different sections of the bridge deck from below. At the interior girders, inspectors walked along catwalks that hung about 10 to 12 ft (3.0 to 3.7 m) below the deck. For the overhangs past the exterior girders, however, there was no access adjacent to that portion of the deck. Therefore, researchers made their observations from the ground, which was about 40 to 80 ft (12 to 24 m) below the bottom of the deck. Although the original intent was to include a detailed survey of the top of the deck, this was not done because the entire concrete deck had been resurfaced with asphalt over time. There were a few spans where the asphalt had recently been milled from the surface. However, inspection was still hindered by the grooves in the concrete and the debris that remained after the milling.

The team took photographs and logged comments about representative samples of the deck, including such details as the following:

- bridge deck cracking
- panel-to-panel joints
- panel-to-stringer connections
- panel-to-girder connections
- pour-back locations
- evidence of leaking joints causing girder corrosion
- post-tensioning anchorage locations
- expansion joints
- drainage details.

Concrete Sampling

The visual inspection of the top of the bridge deck guided the location selection for gathering concrete core samples for testing material properties. One location, designated *Group A,* was an area where the asphalt wearing surface had already been removed and the concrete appeared to be in sound condition. These samples were located about 6 to 8 ft (1.8 to 2.4 m) east of the fifth joint west of the bascule span and were taken from the exterior westbound lane of the bridge. The second group, designated *Group B,* was taken from an area where there had been some asphalt patching, which might have been an indicator of distress occurring at or below the surface of the concrete deck panels. This area was approximately 30 ft (9 m) east of the west abutment and 9 to 12 ft (2.7 to 3.7 m) south from the median wall separating the two travel directions, i.e., the interior eastbound lane.

On July 20, 2006, researchers from VTRC enlisted the help of VDOT's Fredericksburg District to obtain eight samples in each test group. The VDOT crew used a concrete core-drilling rig that had a 4-in rotary diamond drill bit that drilled vertically down into the deck. The goal was to core each concrete sample to a depth of 8 in, and for each of those samples to be free of any steel reinforcement. However, the concrete typically fractured before the intended depth was reached. Further, the thickness of the asphalt overlay in the area of Group B samples, nearly 4 in in some locations, made it more difficult to obtain 8 in of concrete. With regard to the steel reinforcement, the cover of both the asphalt and the concrete was too thick for a pachometer to detect the location of the reinforcing steel. Therefore, once the VDOT crew drilled the first core in each group, the researchers directed them to move the rig accordingly to avoid the reinforcing steel. However, given the complexity of the orthogonal spacing of both the top and bottom mats of mild steel, along with the post-tensioning tendons going in both the longitudinal and transverse directions, the likelihood of obtaining drilled core samples without any steel reinforcing was low. Indeed, all but one core contained some portion of mild reinforcement, and six of the cores intersected segments of prestressing strand or post-tensioning duct. Figures 4 and 5 show typical concrete cores that had been removed from the original bridge deck. Note that the nominal maximum aggregate size in these cores is 0.75 in (20 mm).

Once removed from the deck, the individual concrete cores had moisture on the surface because of the water used during the drilling process. The samples were allowed to air dry just until the surface was no longer wet before being labeled. To maintain the concrete in its as-sampled moisture state, the cores were then wrapped in one layer of polyethylene, one layer of aluminum foil, a second layer of polyethylene, and then a final layer of duct tape. There was a total of 16 cores, 8 for each group, which was enough to provide two samples from each group for each of the intended tests. After drilling was complete, the VTRC researchers transported the samples back to their laboratory facilities.

Figure 4. Typical Sample from Group A

Figure 5. Typical Sample from Group B

Concrete Testing

The specimens remained wrapped and were stored indoors at about 72 ^{o}F (22 ^{o}C) with no specific humidity control until tested. Prior to testing, the researchers unwrapped some of the cores to determine which specimens would be used for which tests, based on the overall length of the specimens and the amount and location of reinforcing steel in them. The primary goal was to identify specimens that were long enough to allow tests of the compressive strength of the concrete. A secondary goal was to identify specimens that were either free of reinforcing steel or had steel located in the concrete such that the steel would not adversely affect the test results. By the process of elimination, cores were selected for each test, the final specimens being used for determining the unit weight of the concrete. These samples were not unwrapped until the time testing began; the other specimens remained uncovered until the time of testing.

The basic process for testing each cylinder was as follows. After unwrapping a core sample, one researcher made a sketch of the cylindrical specimen. The sketch showed the top and bottom of the core and the circumference of the cylinder projected onto a two-dimensional grid. The grid had 0.5-in by 0.5-in increments and showed the relative location of distinguishing features of the core, such as mild reinforcing steel, post-tensioning tendons or ducts, large air voids, discoloration around any aggregates, and the cylinder's profile along the fracture plane at the bottom. A second researcher then took high-resolution photographs of the top and bottom of the sample and the cylinder lying on its side and then the cylinder was rotated a quarter turn for each of four successive pictures. If there was any post-tensioning steel that could be retrieved from the concrete, the researcher sealed these items in polyethylene sheet, aluminum foil, and duct tape.

As mentioned, if a particular core was designated for an experiment other than the density test, the sample was set aside after sketching and photographing. On the other hand, those specimens to be used for determining the density also contained sufficient material for testing absorption, chloride permeability, and chloride content. Therefore, as soon as the sketching and photographing was complete, two 2-in sections were cut from these samples using a dry-cut saw and set aside. The first cut was made 2.125 in below the concrete surface to account for the material lost during the cutting process and was used for determining the chloride content. The second cut was made 2.125 in below the first; this section was used for the rate of absorption and chloride permeability studies. The remaining mass of concrete was weighed and placed into a drying oven in accordance with American Society for Testing and Materials (ASTM) Standard C 642-97 (ASTM, 1997) to determine density and absorption capacity. Once the research team was ready, they tested the remaining samples using the appropriate ASTM standards, as indicated in Table 1.

For ASTM C 1152-04, technicians pulverized the material into a fine dust with a milling machine fitted with ganged diamond-impregnated tuckpointer blades. These blades were capable of cutting an 0.5-in-wide path in the concrete with each pass. Profiles, generated at ¼-in increments sampled from each core, were obtained from the concrete surface to the depth of 2 in, or to the reinforcement, where available. The results from ASTM C 1152-04 for each depth increment were then combined to create a profile of the chloride content down into the concrete.

Table 2 shows which cores were used for which tests. Since the concrete samples had very uneven top and bottom surfaces, both ends of the compressive strength cores were saw-cut and then capped with sulfur mortar in accordance with ASTM C 617-98 to comply with the criteria in ASTM C 39-05. The drilled cores were 4 in in diameter, and the ratios of the core length to the diameter of samples used in the compressive strength tests were greater than 1.75. Hence, the results of those compressive tests did not require correction factors, as stated in ASTM C 42-04.

After samples were cut for the chloride content and rate of absorption/chloride permeability studies, the remainder of those cores was used for determining concrete density and absorption capacity. In the cases of cores A-7 and A-8, there was only about 17.9 in^3 (293 cm^3) and 20.9 in^3 (343 cm^3), respectively, of concrete. Although this volume was nearly 17% and 2%, respectively, less than the 21.4 in^3 (350 cm^3) required under ASTM C 642-97, it was considered adequate for the purposes of this study. The two cores from Group B used for the density test had enough volume to satisfy the stipulations in ASTM C 642-97.

Table 1. Concrete Material Tests and Respective ASTM Standard

Test	ASTM Standard
Compressive Strength	C 39-05
Elastic Modulus	C 469-02
Splitting Tensile Strength	C 496-04
Density and Absorption of Hardened Concrete	C 642-97
Rate of Absorption of Water by Concrete	C 1585-04
Rapid Chloride Penetration	C 1202-05
Acid-Soluble Chloride in Concrete	C 1152-04
Petrography	C 856-04
Obtaining and Testing Drilled Cores	C 42-04
Capping Cylindrical Concrete Specimens	C 617-98

Table 2. Test Samples and Designated Tests

Group	Core No.	Sampled Length (in)	Tested Length (in)	Test
A	1	7.00	6.69	Splitting Tensile
	2	5.50	—	Petrography
	3	7.50	7.38	Compression
	4	7.25	7.44	Compression
	5	7.00	—	Petrography
	6	8.00	8.00	Splitting Tensile
	7	6.50	—	Density
	8	5.50	—	Density
B	1	7.25	—	Density
	2	2.75	—	Petrography
	3	7.13	7.00	Splitting Tensile
	4	7.13	—	Density
	5	6.81	6.81	Splitting Tensile
	6	7.06	7.19	Compression
	7	5.38	—	Petrography
	8	7.06	7.19	Compression

Note: The sampled length of each specimen does not include any asphalt or sand-epoxy overlay that might have been present. The tested length is the length of a given concrete sample after it had been cut and capped in accordance with ASTM C 42 and ASTM C 671. Modulus of elasticity tests were performed on the same cylinders used for testing compressive strength. The cores used for determining density also contained sufficient material for the absorption, chloride permeability, and chloride tests.

RESULTS AND DISCUSSION

Visual Inspection

As mentioned previously, the surface of the full-depth, precast concrete deck panels for the Woodrow Wilson Bridge had been topped with an asphalt wearing surface after the panels had been set in place and longitudinally post-tensioned, as shown in Figure 6. This wearing surface was in addition to the 1/4-in (6 mm) layer of sand-epoxy overlay that the fabricator had applied at the precasting yard. At the time of the survey, a few spans adjacent to the bascule span had been milled to remove the asphalt and epoxy mortar prior to demolition of that portion of the structure. Given the rough surface texture after the milling operation, the concrete deck appeared to be in good condition. There were no discernible cracks or signs of impending spalls. Even the expansion joints appeared to be in fairly good condition. As for the rest of the approach spans, there were only a few signs of distress near the west abutment, as evidenced by asphalt patches.

Deck drainage was facilitated by slotted scupper drains beneath the exterior parapet walls and drain holes in the shoulder lanes. Generally, drainage scuppers were clogged with debris, as is common with deck drainage system designs of this type. In some cases, the scuppers had been partially covered up with asphalt over time, as shown in Figure 7. Although the deterioration of the median barrier is exemplified in Figure 8, the inspection teams found that the barrier

Figure 6. Looking West at Eastbound Approach to Old Woodrow Wilson Memorial Bridge

Figure 7. Drainage Scupper Partially Obstructed by Asphalt Overlay

connections to the concrete deck from below were in good condition. Figure 9 shows these connections with minor rust staining and efflorescence occurring around the connections and the longitudinal joint.

Unfortunately, the researchers were unable to view the post-tensioning anchorages. However, the team did have access underneath the deck in between the two exterior girders. Generally, the concrete appeared to be in excellent condition. There were some indications of efflorescence at various locations throughout the deck but no signs of severe rust staining. What

Figure 8. Concrete Spalls Attributable to Rusted Rebar in Median Barrier

Figure 9. Median Barrier Connection to Underside of Full-Depth, Precast Concrete Deck Panels. The black circles are plastic reinforcing bar chairs used during the fabrication process.

initially appeared to be pattern cracks were cobwebs, as seen in Figure 9. Figure 10 shows the concrete deck and panel joint in good condition. Some panel joints exhibited rust staining; efflorescence; and, on occasion, small pop-out spalls, as shown in Figure 11.

More corrosion and efflorescence were evident at the joints where construction crews placed closure pours after longitudinally post-tensioning a group of panel segments together.

Figure 10. Underside of Adjacent Deck Panels and Accompanying Joint in Good Condition

Figure 11. Panel Joint with Rust Staining and Spall

Typically, the armor plating at these joints was moderately rusted and moisture from a prior rainfall was evident at many closure pour locations. Further, the stringers supporting the deck panels at these locations had rust and scaling, as seen in Figures 12 and 13. In addition to the joints, some steel bearing plates and hold-down rods were corroded where water had apparently seeped down between the interface of the precast concrete and the methyl methacrylate polymer concrete used for the pour backs that served as bearing pads, as seen in Figures 14 and 15. Although most of the hold-down rods affected by the water had only moderate corrosion, some had significant section loss.

Figure 12. More Efflorescence and Rust Staining at Closure Pours

Figure 13. Corroded Steel at Expansion Joint

Figure 14. Bearing Seat Subject to Corrosion

Figure 15. Corroded Hold-Down Bar Connecting Deck to Stringer

Concrete Testing

Compressive Strength Tests

Table 3 gives the results of the compressive strength, elastic modulus, and splitting tensile strength tests, showing the average results within each group of cores. The average compressive strength for the four cylinders tested was 7,010 psi (48.3 MPa). The average for Group A cylinders was about 8% greater than that for Group B cylinders.

Table 3. Results of Compressive Strength, Elastic Modulus. and Splitting Tensile Strength Tests

Test	Group	Core No.	Result psi (MPa)	Average Result psi (MPa)
Compressive Strength	A	3	7,270 (50.1)	7,280 (50.2)
		4	7,290 (50.3)	
	B	6	6,670 (46.0)	6,730 (46.4)
		8	6,790 (46.8)	
Elastic Modulus	A	3	2.80×10^6 (19300)	2.79×10^6 (19300)
		4	2.79×10^6 (19200)	
	B	6	3.44×10^6 (23700)	3.17×10^6 (21860)
		8	2.91×10^6 (20100)	
Splitting Tensile	A	1	495 (3.4)	520 (3.6)
		6	545 (3.8)	
	B	3	540 (3.7)	555 (3.8)
		5	565 (3.9)	

The four cylinders had either Type 3 or Type 4 fracture patterns described in ASTM C 39 and seen in Figure 16. None of the cores had fracture planes near any reinforcing bar that might have been present. Reinforcing steel, if present, was removed and visually inspected after compressive strength testing. As with all cores removed from the bridge, the mild-steel reinforcement was observed to be coated with a brown epoxy and appeared to be in excellent condition, except for minor rust on exposed ends of the reinforcing bar at the surface of the cylinder, which had occurred in storage. The post-tensioning strands also appeared to be in good condition. The steel retrieved from the cylinders was sent to Pennsylvania State University for further analysis and is discussed later in this report.

The average of compressive strength results indicate a less than 7% gain in strength compared to the average 28-day compressive strength of 6,570 psi (45.3 MPa) for the Woodrow Wilson Bridge full-depth, precast concrete deck panels, as reported by Lutz and Scalia (1984). This gain in compressive strength is relatively small compared to strength increases from 18% to 260% found in other long-term studies of normal weight concrete (Al-Khaiat and Fattuhi, 2001; Neville, 1996; Washa and Wendt, 1975). The concrete mixture was noted to contain Type II portland cement, and no use of supplementary cementitious materials was reported. In addition,

Figure 16. Typical Fracture Patterns from Compressive Strength Tests

the panels were reported to have been steam cured for the first 16 hours after casting, followed by 6 days of moist curing (Lutz and Scalia, 1984). The choice of cementitious materials and curing regime might explain the absence of significant additional hydration and further strength development after initial curing. However, Washa and Wendt also reported that some concretes, and, in particular, those with finely ground cement, reached their peak compressive strength between 10 and 25 years of age, after which there was some decline in strength. After 23 years of service, the compressive strength in the Woodrow Wilson Bridge may have diminished somewhat from its peak value.

Modulus of Elasticity

The average modulus of elasticity among the four cylinders tested by VTRC was 2,960 ksi (20.4 GPa). No modulus test results were available from the original production for comparison (J. Getaz, personal communication, May 17, 2007). However, Table 4 shows summary information from studies on other lightweight concrete at VTRC since 2001. In comparison, the concrete placed in 1983 for the deck panels in the Woodrow Wilson Bridge fell at the low end of these compressive strength and modulus values. However, it is important to note that these modern lightweight concretes are high-strength, high-performance mixtures that incorporate fly ash or ground-granulated blast furnace slag to improve strength and reduce permeability.

Another interesting comparison is the relation of elastic modulus to density and compressive strength. In design, the modulus is calculated as a function of unit weight and the *specified* compressive strength, f_c' (lb/in^2). The American Concrete Institute's (ACI) equation in section 8.5.1 of ACI 318-05 (ACI, 2005) estimates the modulus as:

$$E = w_c^{1.5} 33\sqrt{f_c'} \qquad \text{[Eq. 1]}$$

where w_c is the unit weight of the concrete (lb/ft^3). If actual compressive strengths from limited testing are used instead of specified compressive strength, f_c', the relational constant in Eq. 1 will be different from that used in the design. In the case of the cores taken from the deck panels of the Woodrow Wilson Bridge, the average factor was 28.3, which is within the range of 23 and 39 seen in results from VTRC, as indicated in Table 4. Since the compressive strength of production specimens must be higher on average than the minimum specified compressive strength, a lower relational constant for production material versus the constant for design strength is to be expected.

Table 4. Strength and Modulus Results from Recent VTRC Studies of Lightweight Concrete

Statistic	Compressive Strength psi (MPa)	Elastic Modulus ksi (GPa)	Splitting Tensile Strength ksi (GPa)
Average	8,270 (57.0)	3,350 (23.1)	585 (4.0)
Maximum	9,470 (65.3)	4,330 (29.9)	710 (4.9)
Minimum	5,960 (41.1)	2,820 (19.4)	400 (2.8)
Median	8,400 (57.9)	3,210 (22.1)	570 (3.9)

Source: C. Ozyildirim, unpublished data, May 2007.

Splitting Tensile Strength

The average strength from the splitting tensile test was 535 psi (3.67 MPa). Again, splitting tensile strength data from the time of casting were not available. However, VTRC has data from recent studies of lightweight concrete, and Table 4 lists these results. The splitting tensile strengths of the deck panels of the Woodrow Wilson Bridge were comparable to those of other recent projects. Although Group B had a lower compressive strength than Group A, it had slightly higher splitting tensile strengths.

Though generally not taken into account for strength design, engineers need to consider the tensile strength of concrete in order to reduce concrete cracking and thus mitigate reinforcement corrosion. Section R11.2.1.1 in ACI 318R (ACI, 2005) states that the relationship between the splitting tensile strength, f_{ct}, and the specified compressive strength, f_c', for normal weight concrete is:

$$f_{ct} = 6.7\sqrt{f_c'} \qquad \text{[Eq. 2]}$$

However, for sand-lightweight concrete (i.e., lightweight concrete that has lightweight coarse aggregate and natural sand for fine aggregate), ACI recommends reducing the design splitting tensile strength by 15%, because research shows that the tensile strength of lightweight concrete is a fraction of the tensile strength of normal concrete. This reduction effectively makes the constant in Eq. 2 equal to 5.7.

Similar to the explanation given in the discussion on elastic modulus, the constant in Eq. 2 will be different with experimental data. Based on the results shown in Table 3, the constant factor correlating splitting tensile strength to compressive strength in the deck of the Woodrow Wilson Bridge averaged about 6.4, which matches the average value of 6.4 calculated from 28-day data gathered by Ivey and Buth (1967) and that reported in recent VTRC studies of lightweight concrete. This result is somewhat surprising, since Komloš (1970) stated that the rate of increase in the tensile strength of concrete after 90 days slowed considerably and the ratio between tensile strength and compressive strength, f_t/f_c, decreases as the age of the concrete increases. Assuming a similar 28-day f_t/f_c ratio in the Woodrow Wilson Bridge full-depth deck panels 23 years ago to that reported by Ivey and Buth or in recent VTRC results, the relationship between splitting tensile and compressive strengths of the lightweight concrete in the deck of the Woodrow Wilson Bridge has changed little during its service life. This suggests the compressive strength of the concrete in the deck panels did not increase much after 28 days of age.

Generally, the plane of fracture in these splitting cylinder tests passed through the lightweight aggregate, indicating a good bond between the mortar and the aggregate, as seen in Figure 17. In core A-1, however, the researchers observed that a secondary crack formed around a piece of reinforcing bar embedded in the core. The result for core A-1 was about 9% below both the splitting tensile strength of sample A-6 and the average splitting tensile strength of all four cores. The reinforcing bar, shown in Figure 18, may have influenced this lower result.

Figure 17. Fracture Plane Passing Through Aggregate After Splitting Tensile Test

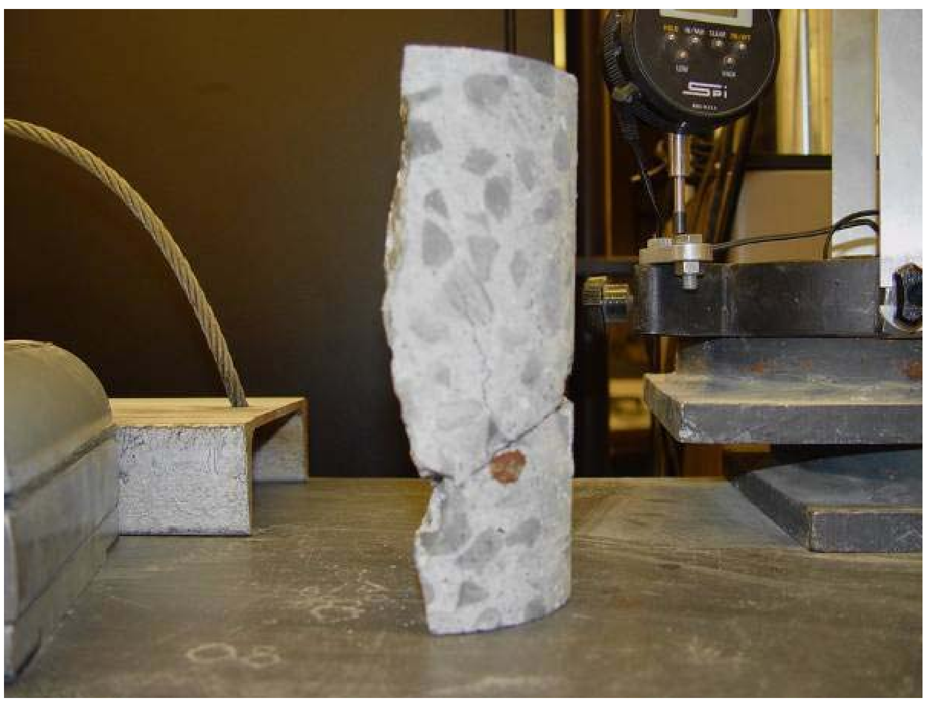

Figure 18. Crack Forming Near Rebar After Splitting Tensile Test

Density

Table 5 gives the unit weights of four concrete core samples, determined using ASTM C 642-97. The equilibrium density is considered to be the same as the in-situ conditions preserved by the four layers of wrapping around the core samples. The average unit weight for the four

Table 5. Unit Weight of Four Core Samples Using ASTM C 642-97, in lb/ft^3 (g/cm^3)

Density Measurement	Sample Number				Average
	A-7	A-8	B-1	B-4	
Equilibrium	117.3 (1.88)	118.6 (1.90)	114.9 (1.84)	115.2 (1.85)	116.5 (1.87)
Dry	108.6 (1.74)	109.6 (1.76)	110.5 (1.77)	110.9 (1.78)	109.9 (1.76)
Immersed	118.1 (1.89)	119.3 (1.91)	118.2 (1.89)	118.5 (1.90)	118.5 (1.90)
Boiled	118.6 (1.90)	119.8 (1.92)	118.9 (1.90)	119.0 (1.91)	119.0 (1.91)
Apparent	129.2 (2.07)	131.0 (2.10)	127.6 (2.04)	127.5 (2.04)	128.8 (2.06)

concrete cores was 116.5 lb/ft^3 (1866 kg/m^3), which is less than 1 lb/ft^3 (16 kg/m^3) greater than the average unit weight reported for samples taken at the time of the deck was cast. Although the tests revealed that the Group A cores were about 3 lb/ft^3 (48 kg/m^3), or 2.5%, heavier than the Group B cores, this variance is fairly typical for concrete production. Both groups had results well within acceptable ranges of current standards for lightweight concrete. Individual samples from Group A had less volume (17.9 in^3 [293 cm^3] for core A-7) than ASTM C 642-97 requires. However, additional smaller samples retrieved from the same core (5.6 and 4.1 in^3 [91 and 68 cm^3]) indicated density less than 1% below the unit weight of the larger sample, showing consistent results.

Absorption

The results for the absorption testing appear in Table 6, along with the accompanying graph of the ASTM C 1585-04 data points in Figure 19. The initial absorption rate, C_i, in Table 6 is an indicator of the rate of capillary suction, whereas the secondary absorption rate, C_s, measures longer term absorption and the fluid transport that can affect concrete durability. The initial rate is taken from time t = 1 min to t = 6 hr, and the secondary rate is taken from time t = 1 days to t = 7 days. In either case, the absorption rate is defined as the slope of the best fit line charting absorption, I, versus the square root of t (sec). I (mm), charted in Figure 19, is calculated as

$$I = \frac{\mathrm{m}_t}{\mathrm{A} \cdot \gamma} \qquad \text{Eq. 3}$$

where m_t is the change in mass (g) at time t (sec); A is the area of the specimen exposed to the water (mm^2); and γ is the density of water (g/mm^3).

Table 6. Absorption Rates and Total Absorption Determined Using ASTM C 1585-04 and ASTM C 642-97

Group	Core No.	Initial Absorption (C_i)		Secondary Absorption (C_s)		Total Absorption	
		C_i x 10^{-4} (mm/s^{½})	$C_{i\,avg}$ x 10^{-4} (mm/s^{½})	C_s x 10^{-4} (mm/s^{½})	$C_{s\,avg}$ x 10^{-4} (mm/s^{½})	Result (%)	Average (%)
A	7	55.0	59.0	20.2	21.9	9.1	9.2
	8	63.0		23.6		9.3	
B	1	53.9	50.5	24.5	25.8	7.6	7.4
	4	47.1		27.1		7.3	

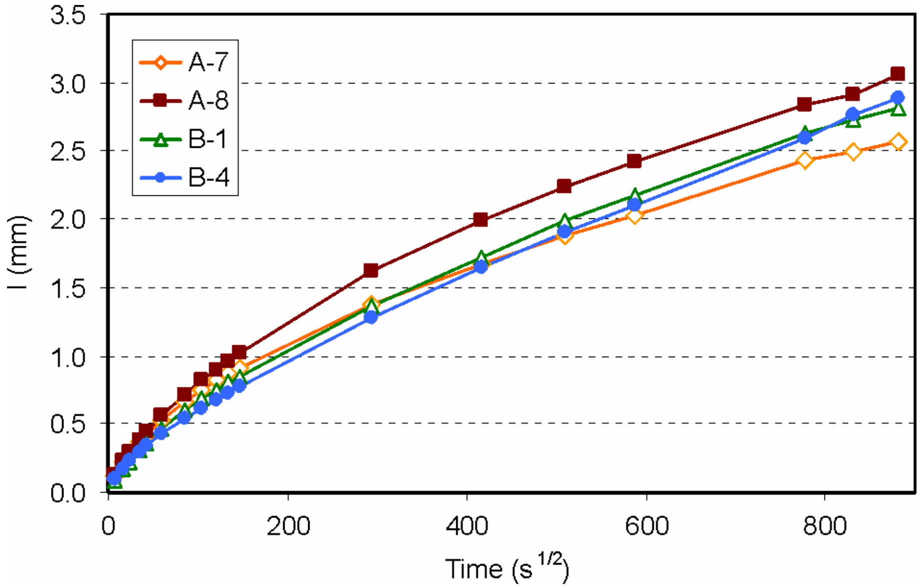

Figure 19. Absorption Rates from ASTM C 1585-04

The average C_i for the four samples tested is 54.7 x 10^{-4} mm/s^{½}, and the average C_s is 23.8 x 10^{-4} mm/s^{½}. The correlation coefficient for both initial and secondary rates for the cores was 0.98 or greater, indicating linear relationships in the absorption rates. One note of interest is that Group B had a slightly lower absorption rate than Group A. The lower absorption rate might indicate lower susceptibility to chloride ingress, but as mentioned earlier, Group B came from a location where there was asphalt patching. The researchers thought that this patching was a result of concrete distress, but the exact cause remains unconfirmed.

Compared to normal weight concrete samples taken from a broad range of in-service bridge decks across Virginia, the results in Table 6 are extremely high, especially when considering that the comfortable limits for C_i and C_s are 20 x 10^{-4} mm/s^{½} and 10 x 10^{-4} mm/s^{½}, respectively (Lane, 2006b). The Group B average for initial absorption is higher than that for all but three of the samples studied by Lane, whereas the Group A average is higher than all of Lane's data. Results were similar for the secondary absorption rate. In both groups, the secondary absorption rate was about one-half to one-third of the initial absorption rate and was fairly stable after the first 24-hr period, similar to what Lane found in his study.

The fact that the absorption rates in the deck of the Woodrow Wilson Bridge were high is not entirely surprising, because one bridge deck in the Lane study was constructed with cast-in-place lightweight concrete. This particular deck had both initial and secondary absorption rates (33.9 x 10^{-4} mm/s^{½} and 22.1 x 10^{-4} mm/s^{½}, respectively) that were in the top one-third of that for the samples in the study. Further, the total absorption was high (about 7.8%). However, the initial absorption rates for this Woodrow Wilson Bridge study were much higher than for that bridge, and the secondary absorption rate and total absorption rate were about the same or higher than the rates for the aforementioned bridge deck, which was constructed with an expanded shale lightweight aggregate that has relatively high absorption values. Although the absorption potential of expanded slate may be lower than that of shale, the material's inherent porosity will still affect the absorption rate, as indicated by the results in Table 6.

The concern with high absorption rates is that they indicate a potential for rapid chloride ingress into concrete, where chloride levels are a major factor limiting the durability of concrete bridge decks attributable to corrosion (Lane, 2006b; Ozyildirim and Halstead, 1991). However, Lane indicated that ASTM C 1585-04 may not be an effective test for lightweight concrete, because this standard was designed to measure the quality of the cementitious matrix in the concrete (Lane, 2006b). The connectivity between both pores in lightweight aggregate and pores on the aggregate surface to capillaries in the cement paste matrix, as well as the absorptivity of the aggregate, will affect the results in comparison to normal weight low absorption aggregate.

Chloride Permeability

There is some debate regarding the reliability of the rapid chloride penetration test (ASTM C 1202-05), since it does not accurately mimic the actual mechanisms of transport of chlorides through concrete, i.e., absorption and diffusion (Lane, 2006a). Nevertheless, the results are listed in Table 7. Figure 20 graphs the data leading up to the information in Table 7. The total charge passed through each specimen during the test is presented in Table 7, where each value is simply a discrete integration of the respective curves in Figure 20 and converted into coulombs. Since the core diameters were not equal to 3.75 in, the nominal value for the test, the results had to be adjusted in accordance with ASTM C 1202-05.

Table 7. Results from ASTM C 1202-05, Testing Chloride Ion Penetration

Group	Core Number	Total Coulombs	Adjusted Coulombs	Qualitative Rating
A	7	3229	2878	Moderate
	8	4762	4244	High
B	1	2338	2084	Moderate
	4	2301	2051	Moderate

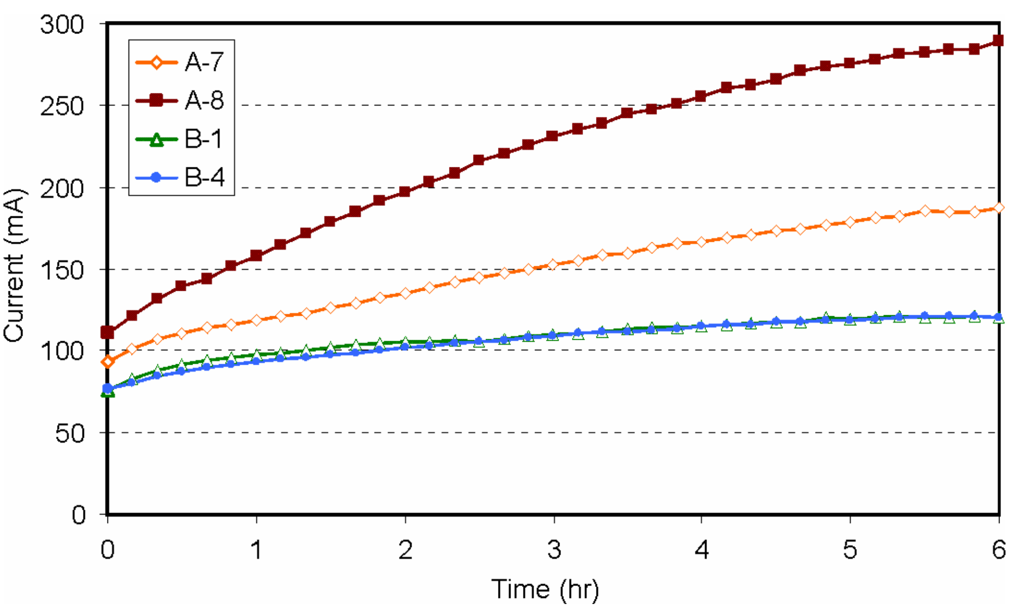

Figure 20. Raw Data from Chloride Ion Penetration Test, ASTM C 1202-05

The results for two samples from Group B appeared to match quite well. On the other hand, the results for the two samples from Group A differed, by about 47%. Re-testing confirmed these results. The disparity may be the result of such factors as the following:

- different ion (i.e., Cl^-) concentrations in the concrete that affect current flow
- localized variations in the aggregate-to-paste ratio and paste structure
- differences in initial placement and vibration of the concrete
- uncertainty as to whether or not the samples actually came from the same batch
- possible presence of undetected microcracking in a sample.

In looking at the Group A specimens used in ASTM C 1202-05, both contained voids with openings that were 0.2 to 0.3 in^2 (130 to 190 mm^2) at the cut surface or along the side of the specimens, although the depth of these voids was difficult to determine. Regardless, the results from ASTM C 1202-05 do match the data from the absorption test, where the Group B cores are fairly consistent whereas the Group A cores have higher rates than Group B and there is about a 15% difference between the two results.

The qualitative ratings listed in Table 7 are also prescribed in ASTM C 1202-05. Three of the samples had moderate permeability values, whereas sample A-8 had a high permeability value at 4244 coulombs. Such moderate to high permeability results would normally be an indicator of susceptibility to corrosion of the reinforcing bar. However, the surface layer of epoxy concrete placed on the panels at the manufacturing plant combined with the added protection of the asphalt wearing surface appears to have helped mitigate such damage. ASTM C 1202-05 states that the age of the sample may also affect the results, depending on concrete type and curing procedures, where the permeability will decrease with time if the concrete cures properly. Therefore, the permeability of the samples from the Woodrow Wilson Bridge would theoretically be lower now than of a sample tested at 28 days or a year. Compressive strength results discussed previously suggest little continued hydration after the first 28 days. If this were the case, permeability would also not be expected to change significantly over time. Theoretically, accelerated curing results in a less refined cement-paste matrix and a more open pore system, which causes lower long-term strength and higher permeability than conventionally cured concrete (Neville, 1996). Concrete that is steam cured too soon after initial set or heated too quickly may develop microcracking that would increase permeability and decrease strength (Neville, 1996). However, no evidence of such microcracking was observed in the samples.

Unfortunately, no records as to what types of admixtures were used in the concrete mixture are available, but certain admixtures could have influenced the results in this study. For example, mixtures designed with calcium nitrite, a common corrosion inhibitor for reinforced concrete and used sometimes in precasting, will have higher coulomb values because of the increased ion content in the concrete pore solution (Ann, 2006; Berke, 1989). In accordance with ASTM C 1543, concurrent ponding tests show that mixtures containing calcium nitrite have a resistivity to chloride penetration similar to that of control mixtures with no calcium nitrite, despite higher coulomb values as determined by tests conducted in accordance with ASTM C 1202 (ASTM 2005). It is possible, though unlikely, that the concrete mixture used for the full-depth, precast lightweight concrete deck panels in the Woodrow Wilson Bridge contained calcium nitrite. If calcium nitrite was added, it was not mentioned by Lutz and Scalia (1984), and since this would have been one of the early applications of this material in precast concrete superstructure elements, it would have been cause for note. Other factors affecting chloride ion penetration include the water-to-cementitious material ratio (w/cm), the air-void system, aggregate type, degree of consolidation, and type of curing (ASTM, 2005). None of the samples used for the ASTM C 1202 tests contained reinforcing steel; therefore, results were not affected by the presence of steel as cautioned in ASTM C 1202-05.

Acid-Soluble Chloride Concentrations

Milled concrete samples from ¼-in (6-mm) depth increments were prepared and tested in accordance with ASTM C 1152 and ASTM C 114 (Method 19) to determine the acid-soluble chloride concentration at each depth.

Figure 21 shows the resulting concentration profiles from four cores, two each from Groups A and B. The chloride concentrations in the Group A cores were indicative of significant chloride ingress, i.e., high concentrations and a classic diffusion pattern. By contrast, Group B chloride concentrations were very low and uniform, consistent with levels of inherent chloride found in fresh concrete mixtures in Virginia bridges (Cady 1983), typically on the order of 0.25 to 0.5 lb Cl^-/yd^3. It is possible that the concrete represented by Group A cores may have contained admixed calcium chloride used as an accelerator, a practice that was out of favor by the time this deck was constructed. Judging by the decrease in concentration at greater depths, it is more likely that the concrete represented by Group A cores had absorbed significant chloride during the 23 years the deck was in service.

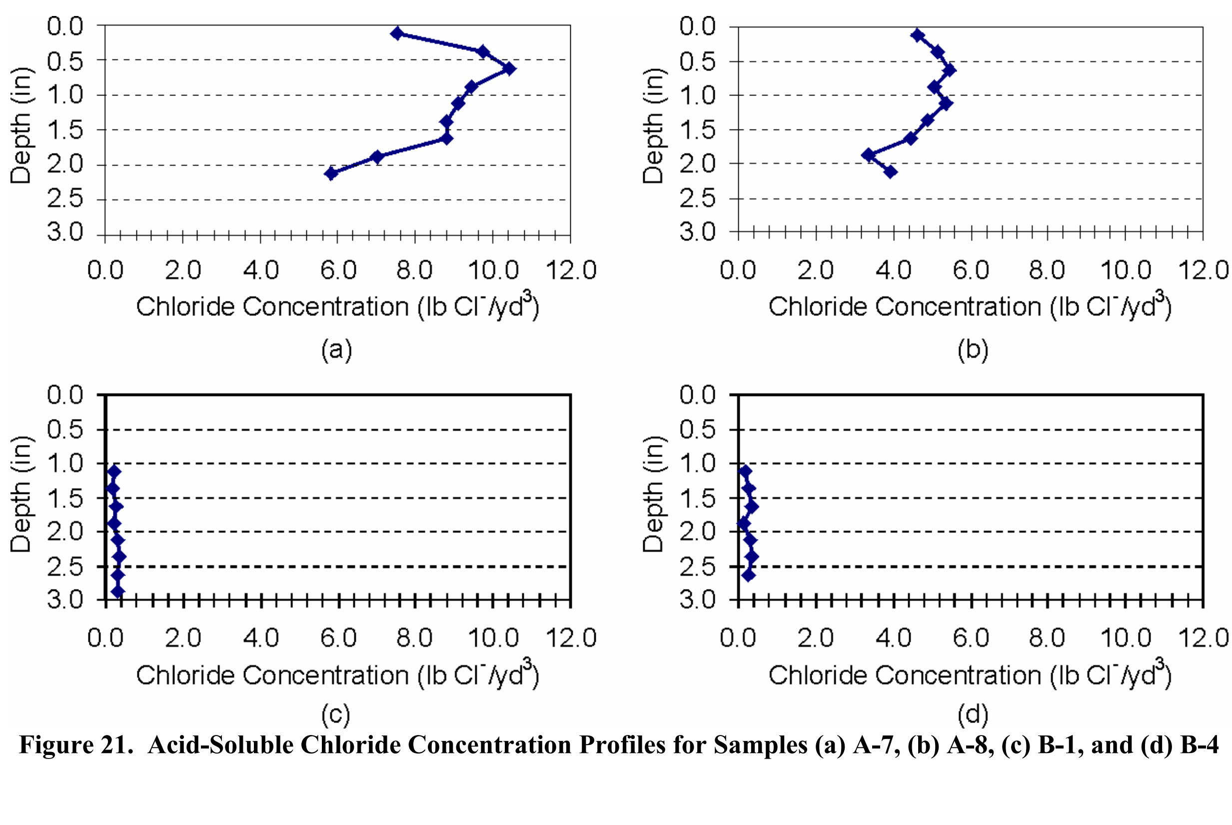

Figure 21. Acid-Soluble Chloride Concentration Profiles for Samples (a) A-7, (b) A-8, (c) B-1, and (d) B-4

It was expected that the polymer concrete wearing surface on the top of the precast deck panels would act as a barrier to chloride ingress. However, concentrations at a depth of 2 in (51 mm)or more in the concrete in Group A cores were well above those necessary to induce corrosion in mild reinforcing steel. Interestingly, aside from location along the length of the structure, the primary difference between Groups A and B was the presence of a much thicker layer, in some locations more than 4 in (102 mm), of asphalt wear surface in the region of the Group B cores. It appears that thicker asphalt surface layers may have been used to address unevenness of the deck panels and to enhance drainage. Despite the fact that asphaltic concrete pavement is known to be somewhat pervious to water, a possible unintended result of an additional thickness of the asphalt wearing coarse was to provide a greater filter to protect against chloride ingress in these areas. Alternatively, the asphalt layer may have had little influence, but the epoxy concrete surface layer in the area of Group A cores may not have provided as effective a barrier as did the application in the area of Group B cores.

Petrographic Analysis

Researchers cut four cores for petrographic examinations. The concrete in the cores was similar, containing expanded shale lightweight coarse aggregate and manufactured fine aggregate composed of limestone particles. The maximum size of the coarse aggregate was 0.75 in (20 mm). The aggregates appeared well graded and evenly distributed throughout the mass. Generally, the concrete appeared sound and of good quality, showing no signs of distress.

The concrete in all cores was air entrained, and the air-void systems appeared adequate for freeze-thaw protection. The amount of entrained air appeared to be higher in cores B-2 and B-7, with estimated air contents of 8% to 10%, compared to that in sample A-2 (4% to 6%). The estimated air content of A-5 was slightly higher (6% to 8%) than in core A-2 because of a prevalence of large voids. The voids were generally lined with a light coating of calcium hydroxide.

The paste in the cores was generally gray with a fairly even color distribution, except for specimen B-2, which appeared somewhat mottled with light areas in the paste and around fine aggregate particles. In addition, researchers observed rims of dark paste around coarse aggregate particles in specimen B-2. This dark color signified a dense paste with a low w/cm that may have caused a slightly higher w/cm in the bulk paste, which would result in more calcium hydroxide crystallization in the paste and around fine aggregate particles. The paste in all cores appeared to bond tightly with both coarse and fine aggregate particles, as well as the reinforcing bars and post-tensioning ducts. Overall, the paste was hard and dense. A pH indicator gave values in excess of 13 for the paste in all four cores below the surface skin.

Reinforcement Analysis

Many of the cores samples from the precast panels contained portions of reinforcement or tendons. Although a full autopsy of a panel would be needed to gain better insight in the condition of the tendons and anchor areas, the small samples obtained in this portion of the investigation are catalogued here.

Mild Reinforcement

The sections of mild reinforcement taken from the bridge were in excellent condition, with no corrosion apparent (other than surface corrosion at the cut ends initiated after removal from the bridge). The seven portions of reinforcement extracted from the cores are shown in Figure 22.

Figure 22. Mild Reinforcement Bars from Precast Deck Slab Core Samples. The brown pigment color of the reinforcement coating should not be mistaken for corrosion product.

Prestressing Tendons

Three greased/sheathed tendon samples were obtained from the cores. Figures 23 through 25 show the three samples prior to coating removal. Tendon A2 sustained heavy coating damage during the coring process. Otherwise, the tendon sheathing was in excellent condition.

Figure 23. Tendon A2 Prior to Sheathing Removal

Figure 24. Tendon A7 Prior to Sheathing Removal

Figure 25. Tendon B1 Prior to Sheathing Removal

The sheathing was then removed to inspect the condition of the PT coating (post-tensioning grease) on the strand surface. A longitudinal slice was made along the length of the tendon to expose the strand. Figures 26 through 28 show the greased strand. (The reinforcing

Figure 26. Mild Reinforcement and Tendon A2 Strand and Casing After Sheathing Removal

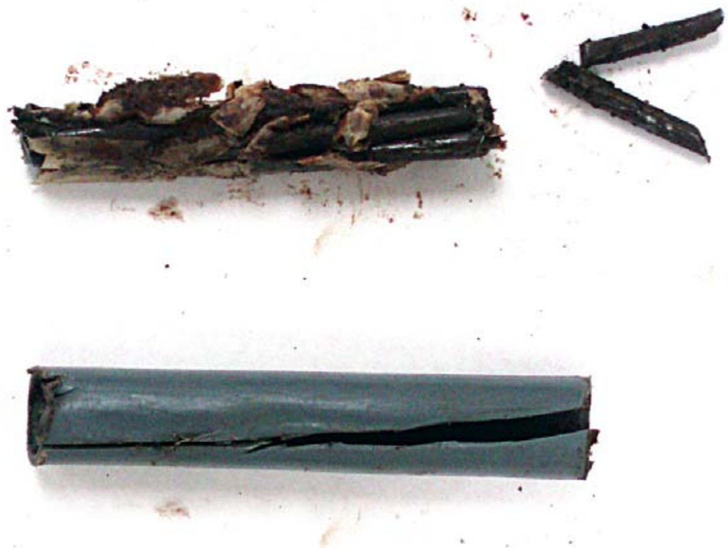

Figure 27. Tendon A7 After Sheathing Removal

Figure 28. Tendon B1 After Sheathing Removal

bar shown in Figure 26 was a bar from the same core.) The strand wires were cut or ruptured in several cases, likely from the coring process. The grease coating was still covering the tendons and providing a protective barrier for the wires in all cases. The strand appeared also to have the remains of a paper-like wrap between the grease and sheathing. This can be seen most easily in Figure 27, where portions of the "paper" are adhered to the grease on the strand.

The individual wires were then cleaned to remove the grease coating for a visual inspection of the strand condition. Figures 29 through 31 show the cleaned wires for each specimen. In all cases, the condition of the wires was excellent with no corrosion evident.

Figure 29. Tendon A2 Wires

Figure 30. Tendon A7 Wires

Figure 31. Tendon B1 Wires

Post-tensioning Grout and Duct

A small portion of grouted post-tensioning duct was obtained from a core sample. As shown in Figure 32, the condition of this sample was excellent. Even with the disruption of coring, the grout was intact and held without the duct section. This portion of grout showed consistent color, indicating that no settlement or segregation occurred during placement in this area. No voids or air bubbles were visible in the grout sample.

Figure 32. Grout Sample in Duct

CONCLUSIONS

- *The full-depth precast lightweight concrete deck panels placed in service in the Woodrow Wilson Bridge in the 1980s performed admirably under the extreme traffic conditions prevalent on the I-95/495 corridor around Washington, D.C.* Although the bridge was replaced to accommodate the ever increasing traffic volumes along the corridor, the bridge deck was still serviceable and might have continued to perform for many more years.

- *Some of the details associated with attachment of the deck to the superstructure and with the closure pours between panels resulted in minor leakage.* Corrosion was observed on underlying superstructure components, and efflorescence was evident in these areas. The leakage was likely a result of shrinkage of the closure grout, shrinkage of the concrete over time, and some differential movement attributable to thermal or structural loading. Nonetheless, the precast panels, which were placed under an accelerated construction schedule during the rehabilitation in the 1980s, appear to have performed well, with few maintenance issues.

- *Post-tensioning of the precast deck appears to have mitigated leaking at the precast panel joints*

- *The lightweight concrete used in the precast deck panels appeared to have adequate compressive and tensile strength, although there was only a small (7%) increase in compressive strength compared to the 28-day measurements.* Elastic modulus results were at the low end compared to current lightweight concrete mix designs. The concrete was sound and showed no evidence of cracking or other deleterious reactions. The regular reinforcing steel and the post-tensioning steel appeared to be in very good condition, indicating that the mild reinforcement coating and the strand and tendon casings were effective in protecting the steel from exposure to corrosive chloride ions in area A, where chloride concentrations were high.

- *Porosity and absorption were significantly greater than that found in modern normal weight concrete and even high relative to that with current lightweight concrete mixtures.* Density, absorption, permeability and chloride tests all indicated a material matrix with the capability of absorbing moisture and other contaminants, particularly chloride, if directly exposed. On the other hand, petrographic analysis showed that the paste matrix was dense and bonded well with the aggregate particles. The presence of an epoxy concrete surface layer, applied during precast fabrication, supplemented by an asphaltic concrete wearing surface, appeared to have limited the exposure to harmful chlorides in many areas of the deck. Further, cover depths significantly greater than the 2 in (51 mm) seen in common practice likely extended the service life of the precast deck panels.

- *VDOT should have confidence in the long-term durability and strength of the lightweight concrete in its bridge decks.*

RECOMMENDATIONS

1. *VDOT's Structure & Bridge Division should continue to use full-depth precast concrete deck panels in bridge rehabilitation projects as a means of minimizing disruptions to the traveling public.*

2. *VDOT's Structure & Bridge Division should use epoxy concrete overlays or other protective wearing surface applications with low transport properties as a means to protect the lightweight concrete in its bridge decks, particularly in panel-to-panel joints, from exposure to deleterious contaminants.*

3. *VDOT's Structure & Bridge Division should consider using longitudinal post-tensioning once closure pours have been placed between deck panels.*

4. *VTRC should continue research on the design of and the material used in the pour-back locations at the expansion joints in order to reduce costly maintenance issues.*

COSTS AND BENEFITS ASSESSMENT

Construction Costs Using Precast Concrete Decks

The initial construction costs associated with precast, full-depth concrete deck panels are almost certainly greater than those associated with traditional cast-in-place decks. The added costs stem from the fabrication process as well as from shipping and placing the precast elements. On average, recent sources (Balakrishna 2006; Hayes, Seay, Mattern, and Mattern, 2007; Wenzlick. 2005) have reported additional premium costs for the precast deck panels on the order of $26/ft^2 ($280/m^2). For a bridge similar to the Woodrow Wilson Bridge, which originally had approximately 550,000 ft^2 (51,100 m^2) of new deck, the $26/ ft^2 ($280/m^2) additional premium would be $13.7 million today.

On the other hand, precasting is known to permit better quality control during production, thus producing a more durable product. Further, using the precast, full-depth concrete deck panels saved more than 4 months in construction time. Thus, the cost premium does not reflect the construction cost savings realized by the reduction in construction time and equipment.

Post-tensioning costs would be additive costs compared to that of conventionally reinforced panel construction. Conversely, formwork and shoring costs are greatly reduced when precast deck panels are employed. Observations reported herein of the post-tensioned panels in the Woodrow Wilson Bridge showed that the post-tensioning prevented joints between the deck panels from leaking, thereby reducing long-term maintenance costs related to deck and superstructure corrosion. Therefore, very little maintenance of the deck panels was required over the course of the 23-year service life. Thus, the post-tensioning system may be more beneficial than non-prestressed deck panels with regard to the overall life-cycle costs, despite the greater initial cost.

As a point of comparison, for a similar project let by the Missouri Department of Transportation (MoDOT) in 2004, the precast deck costs were $56/ft^2$ ($600/m^2$) as compared with MoDOT's average cost of $32 to $40/ft^2$ ($345 to $430/m^2$) for conventional cast-in-place concrete (Wenzlick, 2005). However, the project manager considered those costs to be artificially high because of the lack of experience with such applications in the locality and the size and orientation of the subject structure, i.e., entirely over water. He also cited significantly reduced construction and traffic control costs attributable to the reduced construction time using the precast method. A similar replacement of a 6,970 ft^2 (648 m^2) deck with a precast deck in 2004 by the Ohio Department of Transportation on the West Sandusky Street Bridge over I-75 in Findlay, Ohio, resulted in a 5-week construction time savings for a $200,000, or $28.69/ft^2$ ($309/m^2$), additional construction cost (LeBlanc, 2006).

Road User Cost Savings of Using Precast Concrete

The difference in road user costs can be compared between two construction scenarios: the factual case (precast scenario), in which precast concrete panels were used to replace the deck of the Woodrow Wilson Bridge; and a counterfactual case (cast-in-place scenario), in which the new deck would have been conventionally formed and cast in place. The road user costs accrue because of reductions in the average speed of the vehicles traversing the work zone and/or the queue that forms when the traffic flow exceeds the throughput capacity of the work zone. In the precast scenario, two lanes of the Woodrow Wilson Bridge were closed every night for about 8 months (Lutz and Scalia, 1984). In the cast-in-place scenario, Lutz and Scalia estimated that one-half of the Woodrow Wilson Bridge would have been closed more or less continuously for 12 months. For the purposes of this analysis, it is assumed that two 10-ft-wide (0.93 m) lanes in each direction would have been maintained through the work zone. As I-95 is a beltway rather than a radial route, the directional split is close to 50:50 and a three-lane traffic maintenance plan would have been inappropriate.

Computing Travel Time Delay

The *Highway Capacity Manual* (HCM) (Transportation Research Board, 2000) recommends that the throughput capacity of three 12-ft (3.7 m) lanes designed to accommodate a free-flow speed of 55 mph (89 km/hr) be assumed to be 6,600 veh/hr per lane if there are no shoulder obstructions or other adverse conditions. The HCM recommends that the capacity of a work zone with a single lane open to traffic be assumed to be 1,500 veh/hr, with approximately a 10 mph (16 km/hr) reduction in mean speed. The capacity for two narrow lanes through a long-term work zone, crossing over to the side normally used for travel in the other direction, is estimated to be 3,000 veh/hr, with a 10 mph (16 km/hr) reduction in speed.

Cost of Travel Time Delay

Chui and McFarland (1986) estimated the average value of time for passenger vehicles on four-lane divided highways to be $10.40/hr (in 1985 dollars). They estimated the value of time for trucks to be $19.00/hr. At 2007 prices, these values would be $20.13/hr and $36.78/hr, respectively (Bureau of Labor Statistics, 2007).

Summary

Traffic volume on the Woodrow Wilson Bridge is reported to have averaged 105,680 vehicles per day in 1983 (Virginia Department of Highway and Transportation, 1983). Given the assumed typical hourly distribution of this traffic (Texas Transportation Institute, 1993), plus the capacity parameters previously provided, it is possible to compute the mean travel speed and the length of the queue, if any, for each hour of a typical day of traffic under each of the two scenarios. The road user costs computed for these scenarios can be regarded only as upper limits on the possible user costs, however, it is certain that some travelers would have chosen alternate travel times or alternate routes through the Washington, D.C., area in order to reduce the added travel costs.

Under these assumptions, the precast scenario is estimated to have imposed an additional 191 veh-hr of delay per day for the duration of the work on the Woodrow Wilson Bridge, at a daily cost of $4,290 at present-year prices. Over the 8-month duration of the work zone, the cumulative road user cost increment amounted to approximately $1 million.

It is estimated that the cast-in-place scenario would have imposed an additional 65,514 veh-hr of delay per day, creating backups tens of miles or kilometers long during the afternoon peak period, at a *daily* cost of $1.4 million. The cumulative road user cost would amount to more than $540 million over the 1-year duration of the work zone. It is obvious that a great many travelers would have altered their travel habits rather than face a delay of 45 min in the morning and 2 hr or more in the afternoon, but a network analysis of the sort needed to obtain a more realistic estimate of the true impact of the cast-in-place scenario is beyond the scope of this study.

Conclusions

It is possible to conclude that the pre-cast scenario imposed far less disruption of travel in the Washington, D.C., area than the cast-in-place scenario would have done, a difference worth hundreds of thousands of dollars per day at 2007 prices. Thus, taking into account the total life cycle costs that include road user, construction time, construction safety, and maintenance costs, full-depth precast concrete deck panels will prove to be the more economical alternative.

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