CIVIL ENGINEERING PRACTICE™ • JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS SECTION/ASCE

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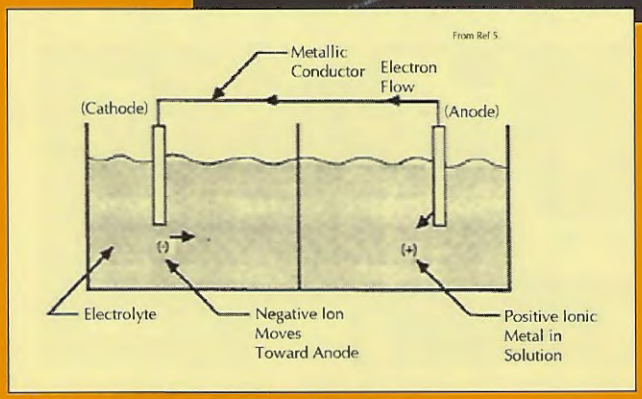
Repairing the Storrow Drive Tunnel



Chloride Contamination in Reinforced Concrete

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CONTENTS

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Erratum

The photograph of Boston Harbor and the Tobin Bridge in the Fall/Winter 2009 issue (Vol. 24, No. 2) was taken by Eric Pheifer.

Our Infrastructure Is a National Treasure!

Every time my wife and I travel through Europe on vacation, it is inevitable that half of the cathedrals we visit are shrouded in scaffolding as part of a major repair and rehabilitation effort. These structures are hundreds of years old and are considered to be national treasures. Ever think that infrastructure is also national treasure? Certainly life would be poorer without our bridges, tunnels, highways, sewers, water mains, harbors and airports. Not many people other than civil engineers would ever think of a sewer as a national treasure, but maybe that is the proper context for the huge investment we as a nation have made over the past decades. As a sampling, in the Boston area, well over $25 billion was spent over the past twenty years on just a few projects (Big Dig, Logan Airport, MWRA's Deer Island, new water treatment, new pipelines and MBTA commuter rail). That sort of expenditure certainly qualifies those infrastructure projects as national treasures! What about all the other infrastructure systems built over the past century and that are still in vital use? These precious, vital treasures that keep our modern society running are together worth many trillions of dollars!

But what do we do with our new treasures? We largely ignore them. Maintenance is not headline-grabbing news the way opening a new highway tunnel or bridge is. When state and city budgets were flush in earlier years, did we repaint our bridges? Ha! The Longfellow Bridge has not been painted for over fifty years! Certainly there were a few good years in that time. But now with budget deficits requiring expenditure cuts in nearly every state, most cities and, of course, the federal government, it seems that our long-neglected public infrastructure is becoming the elephant in the living room. How long can we as a society continue to ignore it? The answer is that we can't anymore.

BSCES President Danielle Spicer's message to the BSCES membership in a recent *BSCES Newsletter* discussed the issues of our deteriorating infrastructure. We have all heard that ASCE has given America's infrastructure a grade of D on its 2009 report card. I am afraid, however, that our elected officials, who have the responsibility of authorizing the funding for major infrastructure projects and setting national priorities, will take the same view of that grade as some of my students and say that "a D is still passing, so what's the problem?" Just how far is it from the D on the ASCE infrastructure report card to the feared F? A student failing a course in college is a lot different than having a bridge over a river fail! Remember, the I-35 bridge in Minneapolis was thought to have another ten years of useful life when it collapsed in 2007. That bridge collapse should have been a national wake-up call, but have we simply pushed the snooze button?

Our national treasure infrastructure is in crisis, and needs strong leadership and active support to reverse the trend of stretching the last breath of useful life out of these systems with lower

and lower expenditures. The cost to rehabilitate a bridge increases by a factor of 5 for each successively lower serviceability grade. Why do we permit our bridges to rust away and the concrete to deteriorate and spall off to expose rebar? Maybe it has been decided that the original outer layer of rebar really wasn't necessary? Come on, civil engineers know better than that. We must harken to the call of our decaying infrastructure, and get involved in the process. Our professional responsibility requires us to be innovative and creative on project work, and design sustainable systems. We must become more pro-active in lobbying for the programs and funding needed to repair, rehabilitate and rebuild our national treasures. We all have to educate the general population on the dire consequences of delayed maintenance. It will cost so much more to rebuild it in the future if we do not repaint it today! Talk about a debt being passed on to future generations. Add several trillion dollars for rebuilding America's infrastructure onto the $14 trillion and rising national debt. Civil engineers must be involved throughout all phases of infrastructure repair, renovation and replacement.

It is appropos that infrastructure maintenance and rehabilitation measures are the two technical topics in this edition of *Civil Engineering Practice*. In his thorough report on the Storrow Drive underpass tunnels, Michael McCall chronicles the problems that stemmed from the tunnels' original design and construction, and which have continued to plague the tunnel throughout its nearly sixty-year life. The serious problems identified in tunnel inspections and condition assessments have been addressed with the recently completed repairs, which added five years to the life of this important transportation link. In several instances, the construction processes actually used were a collaboration with the project civil engineer. However, the story of the Storrow Drive underpass is far from over. These repairs only bought five years' time. What's next? The clock is running. Societal and political arguments and decisions have to be made. The time is opportune for civil engineers to get involved in the process of setting the future course for this transportation link.

Our second article by Iplikcioglu, Lin, Soleimani, Svetieva and Zhao reports on chloride contamination of reinforced concrete — its origin, detection and prevention. Concrete is a major component of infrastructure, and everywhere we travel, the deterioration of concrete is apparent. Several methods of detection investigation and subsequent rehabilitation, as well as methods to mitigate future deterioration, are presented — all have application to current and future rehabilitation of the concrete elements in our decaying infrastructure.

So as you journey around the country, pay close attention to our aging and poorly maintained national treasures. And keep those water and sewer pipes in your thoughts, too. Our infrastructure is worth trillions of dollars. We must keep up with its maintenance, or watch as it slowly decays. And as you watch out for decaying concrete and steel, watch out also for timely, practice-oriented topics on maintaining other parts of our aging infrastructure or other intriguing practice issues in civil engineering today. Perhaps you have worked on such a project, and would want to write a short technical piece, or maybe you can point the Editorial Board in the direction of another member who might be more appropriate to write about it. Please contact me or any member of the Editorial Board with ideas, suggestions or draft papers on an aspect of civil engineering practice that you would like to see in your BSCES journal.

Sincerely yours,

Jim Lambrechts

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Interim Repairs of the Storrow Drive Tunnel

Overcoming political and engineering obstacles, performing interim repairs rehabilitated a critical structure in such a manner that lays the groundwork for prolonging its useful life.

MICHAEL MCCALL

The Commonwealth of Massachusetts Department of Conservation and Recreation (DCR) recently completed a program of structural and waterproofing repairs to the Storrow Drive Tunnel in Boston, which is intended to extend the service life of the tunnel by about five years. The project was successfully completed with minimal disruption to traffic on the parkway, and with minimal disturbance of residential, commercial and recreational activities in the surrounding neighborhoods and parkland. The success of the project can be attributed to careful planning, design and execution of the work, and was aided by all the project participants' adherence to an effective partnering strategy.

Description & Brief History of the Tunnel

The Storrow Drive Tunnel is part of a complex restricted parkway interchange system that actually incorporates two tunnels: the main tunnel carrying underground eastbound through traffic on Storrow Drive; and a much smaller tunnel — the Berkeley Street Underpass — that carries underground traffic entering Storrow Drive eastbound from an on-ramp originating at the end of Berkeley Street (see Figure 1).

The main tunnel has an overall length of 1,280 feet and carries two lanes of eastbound traffic underground and two lanes of westbound traffic on the roof of the structure. The lower part of the structure is basically a reinforced concrete boat section. A portion (760 feet) of this tunnel is covered with a 9- to 10-inch thick reinforced concrete slab supported on steel roof beams with spans varying from 35 feet at the west portal to 63 feet at the mid-tunnel portal for the exit to Otter/Arlington Street. The other 520 feet in the eastern section is covered with a 24- to 36-inch thick simple-span reinforced concrete slab with a constant span of 35 feet.

The small tunnel is about 140 feet long and has a 10-inch thick roof deck supported on steel beams, similar to the western part of the main tunnel, except that it spans a shorter length (27 feet) between walls. It carries one lane of underground eastbound traffic entering Storrow Drive from Berkeley Street and two lanes of traffic exiting Storrow Drive to Otter/Arlington Street over its roof. (This short section of road between Storrow Drive and Beacon Street is actually part of today's

FIGURE 1. Storrow Drive interchange aerial view looking north.

David G. Mugar Way, but historically it was known as Otter Street, and many people today think of it as an extension of Arlington Street, so we will use the Otter/Arlington designation herein.)

The tunnel system was constructed by the Metropolitan District Commission (MDC), the predecessor agency of today's DCR, between 1950 and 1953 as part of the creation of the Storrow Drive parkway, and the associated modifications to the Esplanade park through which it traverses. The interchange in the area of the tunnel originally had all the eastbound exit ramps and entrance ramps that exist today (exit to Clarendon Street, exit to Otter/Arlington Street, entrance from Berkeley Street and entrance from Otter/Arlington Street) and an additional eastbound exit to Dartmouth Street, which is now blocked by concrete barriers. However, the original configuration had no westbound exit to Otter/Arlington Street as it exists today. Instead, there was a westbound entrance from Otter/Arlington Street, and the only westbound exit was to Clarendon Street via a short collector distributor road between Berkeley Street and Clarendon Street. This collector distributor, running parallel to the parkway mainline, also served as the westbound entrance ramp from Berkeley Street (see Figure 2).

One can only imagine the chaos that must have existed on this collector distributor road as the massive 1950s-era automobiles merged, diverged, weaved and probably impacted each other regularly as they tried to exit and

FIGURE 2. Storrow Drive interchange near completion, early 1950s.

enter the westbound parkway over the short distance between Berkeley Street and Clarendon Street. The direct entrance to the westbound roadway from Otter/Arlington Street must also have been a thrilling and sometimes dangerous experience since there was no acceleration lane and entering traffic had to merge instantly with through traffic in the left lane, which was negotiating a sharp right-hand turn on the parkway, and some of which was preparing to make that treacherous exit to the collector distributor road just ahead at Berkeley Street.

In 1954, to alleviate what must have at times resembled a Back Bay demolition derby, the interchange ramps were reconfigured to what we see today (minus the Dartmouth Street exit). At the same time, the structure of the roofs that are supported on steel beams — in the western part of the main tunnel, and in the small tunnel — was modified by the addition of steel diaphragm beams rigidly connected to the original steel roof beams. The diaphragms were added along several parallel lines running perpendicular to the roof beams in the main tunnel, and along a single central line in the small tunnel. The diaphragms were only added under areas that carried roadway traffic on the newly configured ramps and parkway atop the tunnel roofs.

No records exist for the reasoning behind adding the diaphragms. Some have speculated that concerns for buoyancy forces led engineers to intentionally increase the dead load on the tunnel roofs and invert slabs due to concerns about tunnel float-out during flooding conditions. (The thickness of the overburden over the tunnel roofs is up to 2 feet in median areas.) Major hurricanes, Carol in 1954 and Connie/Diane in 1955 (two separate hurricanes, but only about a week apart), hit New England and Boston around this time and caused widespread flooding and flood-related damage. Did engineers at the time see other structures float out of the ground during these floods, calculate that the Storrow Drive tunnels had insufficient float-out resistance, and decide to add dead load (pavement) to help keep them in the ground? Probably not, since the plans for adding diaphragms are dated

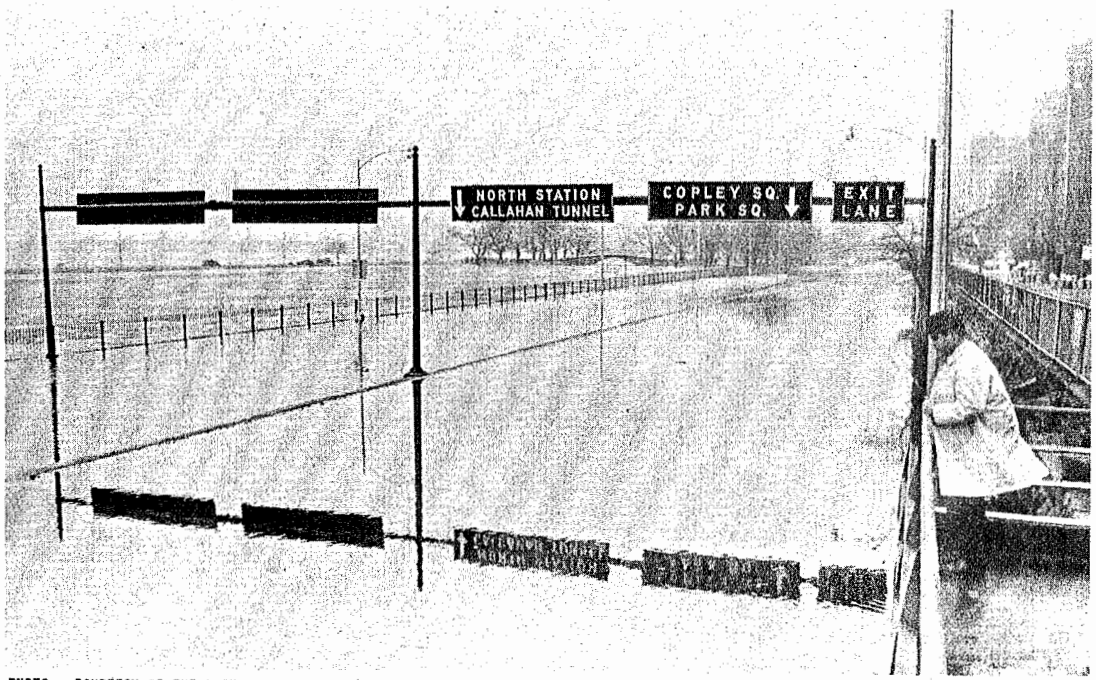

PHOTO - COURTESY OF THE BOSTON GLOBE

Storrow Drive March 1968
(Looking East)

FIGURE 3. Flooded parkway near the tunnel in March 1968.

February 1954, many months before Hurricane Carol hit. Did engineers perhaps observe actual signs of float-out in the tunnels during the hurricane-related flooding? Again, probably not, since an interesting archival photograph from the aftermath of a smaller 1968 hurricane seems to show the main tunnel completely submerged by a Charles River Basin that had overflowed its banks, and a totally submerged tunnel would indicate no risk of float-out (see Figure 3).

Was the structure inadvertently constructed at too low of an elevation, or did the structure settle during or after construction? Either, or both, of these circumstances is probable, because recent survey data show top-of-curb and bottom-of-steel elevations in the tunnel that are a few inches lower than the elevations indicated on the original plans.

Did the original grading of the roadway system, or the 1954 reconfiguration of the ramps, result in the unusually thick pavement and soil overburden? This outcome is also likely because the wide expanse of flat-topped tunnel roofs, and their approach slabs under adjacent roadways, would naturally result in large overburden depths at roadways and medians due to required slopes for surface drainage to catch basins located outside the footprint of the tunnel structures. An early 1950s archival photograph taken near the completion of the original construction clearly shows the placement of a granite curb with its top set about 2 feet above the top of the tunnel roof (see Figure 2). The structural plans for the original tunnel construction consistently show only a uniform 2.5 inches of pavement over the top of the flat-topped tunnel roofs, an impossible condition to achieve in reality because of the necessary grading (for drainage) of the roadways over the tunnels. Thus, it appears that the original structural and civil designs for the interchange were not adequately coordinated, and this lack of coordination was likely the primary reason for the excessive thickness of overburden on the tunnel roofs.

DCR engineers have also speculated that the original construction project may have been conceived with a mud/working slab, but due to economics the final structure did not include this element.

It is clear that the rigidly connected diaphragms were added to help distribute live load over more roof beams than would occur without the diaphragms. This concern for improved live-load distribution almost certainly arose from the engineers' awareness that the unintended thicknesses of pavement and other overburden materials had increased the dead-load stresses in the roof beams to very high levels. This concern would probably have been greater if the parkway was not restricted to pleasure vehicles and excluded all heavy trucks and buses; however, relatively heavy snow plows are now used on Storrow Drive over the tunnel.

During the first few years of its service life, the tunnel experienced heavy leakage of groundwater into the structure, mainly through expansion joints and construction joints in the walls and invert slabs, and also through cracks in these concrete elements. In addition, surface runoff water on the roadways and parkland above the main tunnel leaked freely into the tunnel through roof-slab expansion joints, which inexplicably had been designed and constructed without seals of any kind. This leakage was so severe that the tunnel earned the nickname "Fink's car wash" after the name of the MDC Director of Parks Engineering at the time.

An elaborate system of gutters attached to the roof beams, tile-faced masonry walls offset from the structural concrete walls and steel under-drains placed within the thickened asphalt pavement over the invert slabs was constructed in the main tunnel to conceal the leaking water and direct it into the tunnel's main drainage and pumping system. In addition, large quantities of liquid asphalt were pumped through small holes cored in the tunnel walls and invert slabs at all joints and significant cracks in an attempt to form a waterproof membrane around the most vulnerable locations in the concrete boat section below the groundwater table. This system was completed by about 1960 and, although it was never fully successful in preventing problems associated with the leakage, it was effective enough to alleviate the most serious operational problems for many years. However, the system created a huge maintenance problem for the MDC, which had to deal with these issues as best as it could with maintenance funding that often fell short of what was actually needed. Over the following decades, the system fell into disrepair and the tunnel's concrete and steel components suffered increasingly severe deterioration.

Background to the Interim Repairs Project

In 1992, inspections and engineering studies of the Storrow Drive Tunnel revealed significant structural deterioration and other problems related to deficiencies in the original design and construction of the tunnel. In 1993, to address the most immediate safety concerns, emergency repairs were performed at four expansion joints in the tunnel's concrete roof. Also, a sagging section of the tunnel roof that is supported by the longest-spanning steel beams was permanently shored with new steel columns. In 2001, the DCR, then still known as the MDC, awarded a design contract to an engineering consulting firm to re-evaluate the structure's condition, to assess options for rehabilitating the tunnel and to perform preliminary and final design of the selected rehabilitation option. A report with preliminary recommendations was completed in 2002. The 2002 report included preliminary traffic management plans, which, for some rehabilitation options, included temporarily routing eastbound and westbound traffic onto temporary roads in the Esplanade parkland adjacent to the tunnel that is currently covered with grass and trees. Up to eighty trees between Arlington Street and Clarendon Street would have to be removed to implement this traffic management scheme (see Figure 4).

Also in 2002, the DCR was being sued by property owners in Boston's Beacon Hill and Back Bay neighborhoods who claimed that groundwater leaking into the Storrow Drive Tunnel had caused a general drawdown of groundwater levels in areas of those neighbor-

FIGURE 4. Proposed detour road option within the Esplanade.

hoods adjacent to the tunnel. The lawsuits claimed that this lowering of groundwater levels had resulted in the deterioration of wood piles that support historic masonry buildings, and that significant damage to these buildings had occurred due to structural settlement associated with the pile deterioration. Although they acknowledged that leakage of groundwater into the tunnel had been a problem since the tunnel's original construction, the DCR prevailed in court in 2005 by showing that the volume of water that leaks into the tunnel could not be solely or even mainly responsible for the significant lowering of groundwater levels, which had actually occurred in widespread areas of the city over the entire course of the twentieth century.

With this legal obstacle behind them, the DCR pressed forward with the evaluation of the tunnel rehabilitation options. It became clear that the cost of any major rehabilitation option was so great, and the tunnel was located in such a sensitive area, that alternatives other than rehabilitation or replication of the existing tunnels should be considered. In 2006, the DCR committed to a public participation process, as part of the preparation of an Environmental Impact Report, that laid wide-open the range of long-term options to be considered by its staff, its consultants, the cities of Boston and Cambridge, other state agencies, the political establishment and the general public. Transportation and Landscape Advisory Committees were organized during the tenure of a DCR Commissioner who had vowed, if at all possible, to avoid implementing the preliminary traffic management plan that involved temporary roads in the parkland adjacent to the tunnel. The two advisory committees effectively joined into a single committee over the course of about a dozen public meetings held in 2006 and 2007. Design options multiplied as participants championed variations of the four basic options considered at the outset, which were:

- *Option A:* Rehabilitate the existing eastbound tunnel, providing an additional forty-year structure life.

- *Option B:* Eliminate the tunnel and establish a two-way, signal-controlled surface parkway like Memorial Drive in Cambridge.
- *Option C:* Reconfigure the interchange to incorporate a westbound tunnel and an eastbound, signal-controlled surface road (seventy-five-year structure life).
- *Option D:* Replace the existing eastbound tunnel with two new tunnels for both eastbound and westbound traffic, with new parkland on top of the new tunnels (seventy-five-year structure life).

The advisory committee and the other public participants were unable to come to a consensus on which variations of the options should be pursued during the final environmental permitting process. However, at the end of this phase of the public participation process, an enhanced version of Option A (rehabilitation to provide a sixty-year structure life) seemed to emerge as the default choice of the advisory committee and of the public and institutional entities involved in the discussions. This realization led the DCR, now under the leadership of a new commissioner, to again start thinking about traffic management options for constructing the rehabilitation, which would involve removing and replacing the entire roofs of the tunnels. The DCR felt that the advantages of the temporary detour roads, in terms of easing transportation and reducing project cost and duration, should not be ignored. The DCR also clearly recognized the environmental and societal impacts of cutting down eighty trees to build temporary detour roads. However, the reintroduction of the temporary detour roads at the final meeting of the advisory committee ignited a firestorm of negative public opinion fueled by committee members and others who felt betrayed by the return of this traffic management option, which they believed had been taken "off the table" by the previous DCR Commissioner. The results of this meeting effectively made the project politically too hot to handle and the long-term fate of the tunnel remains uncertain today.

During the many years of institutional inaction, lawsuits, public debate and political wrangling about the long-term fate of the Storrow Drive Tunnel, the condition of the structure continued to deteriorate. For example, the DCR discovered and repaired a large fatigue crack in the web of a steel roof beam at a diaphragm connection in January 2004. And in 2005 structural engineers at the consulting firm that evaluated the structure expressed their opinion to the DCR that deteriorated concrete portions of the tunnel structure were likely to experience localized structural failure within five years if significant remedial action were not taken. The DCR directed the consulting firm to make additional detailed inspections and studies of the tunnel's structural condition. Based on the results of these activities, DCR and the engineering consultant prioritized the necessary repair work into emergency, immediate and interim categories. This prioritization of the short-term repair needs occurred concurrently with the public participation process described above, and the DCR kept the public informed of the latest inspection results and their plans for the short-term repairs during the public meetings.

"Emergency" repairs were made as soon as possible after critical conditions were identified. The main emergency repair consisted of chipping loose concrete off the soffit of the roof slabs. The only other emergency repair was at a cantilevered concrete slab at the top of a boat section wall near the west portal of the Berkeley Street Underpass, where loose deteriorated concrete threatened to fall onto the roadway.

"Immediate" repairs consisted of the following:

- Removal of deteriorated concrete and construction of an up-turned concrete beam over the west portal (the impact-damaged steel roof beam was left in place) (see Figures 5 and 6);
- Installation of hanger supports to prevent reinforcing bars falling at a location where long lengths of bars had been exposed in 1993 but the concrete repair was never completed;
- Installation of structural retrofits (steel plates and angles) at approximately twenty fatigue-prone connections in the

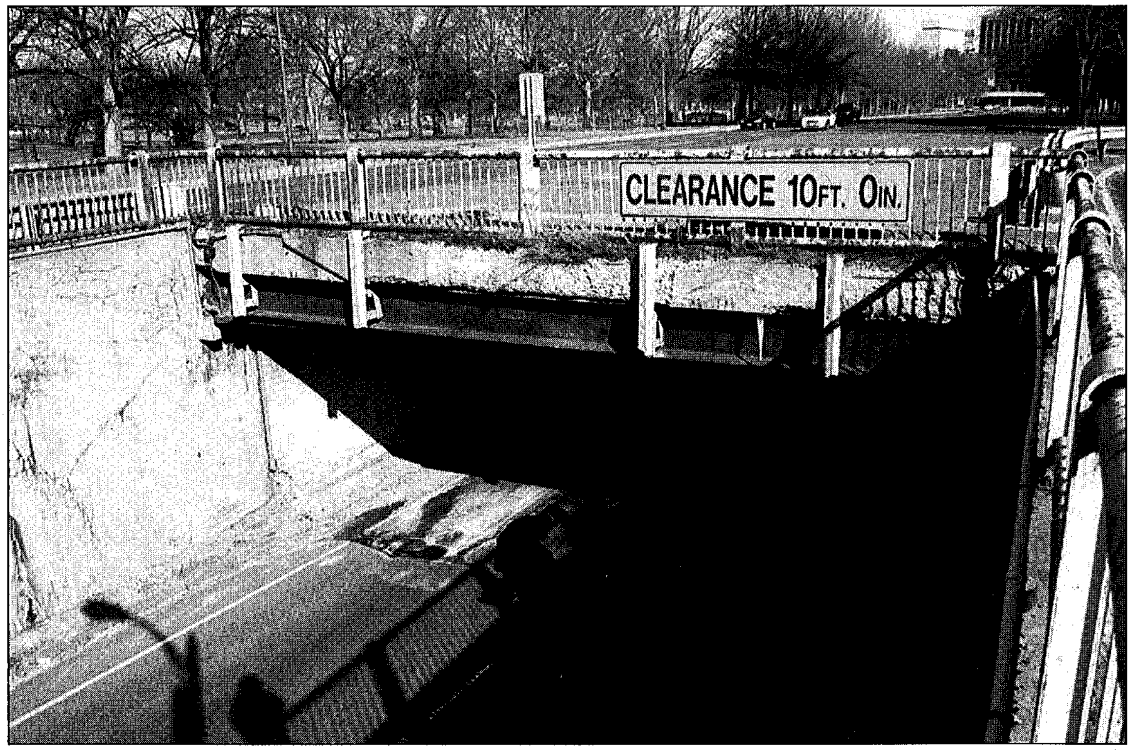

FIGURE 5. West Portal before repairs.

steel beam roof framing system (see Figure 7);
- Structural stabilization of tilting sections of tile-faced masonry walls; and,
- Cleaning and repair of the under-drain system.

The "interim" repairs would need to address structural and waterproofing issues that threatened public safety during the additional years of service life the tunnels had to provide while the long-term issues were being addressed through the ongoing design and permitting processes. However, the assessment of the optimal duration of this short-term extension of the tunnels' service life was not a straightforward matter.

Evaluating the Structural Problems

The 1992 Condition Investigation & Structural Evaluation. The engineering consulting firm performed a fairly comprehensive condition investigation and structural review of the tunnel structures in 1992. The condition investigation included the following field and laboratory investigations:

- Visual inspection of concrete roof slab soffits, steel roof beams, concrete tunnel walls (only visible in the small tunnel and through a few small inspection ports in the masonry walls of the main tunnel) and concrete boat section walls.
- Visual inspection of the topside of concrete roof slabs at test pits made through the soil and asphalt overburden.
- Extraction of asphalt pavement cores over the tunnel invert slabs for pavement thickness determination.
- Extraction of concrete cores from roof slabs and invert slabs to determine chloride ion content, carbonation front penetration, alkali-silica reactivity, strength and quality of the concrete.
- Measurement of the remaining thickness of steel roof beam sections by micrometers and by ultrasonic thickness gages to determine the loss of effective thickness caused by corrosion.

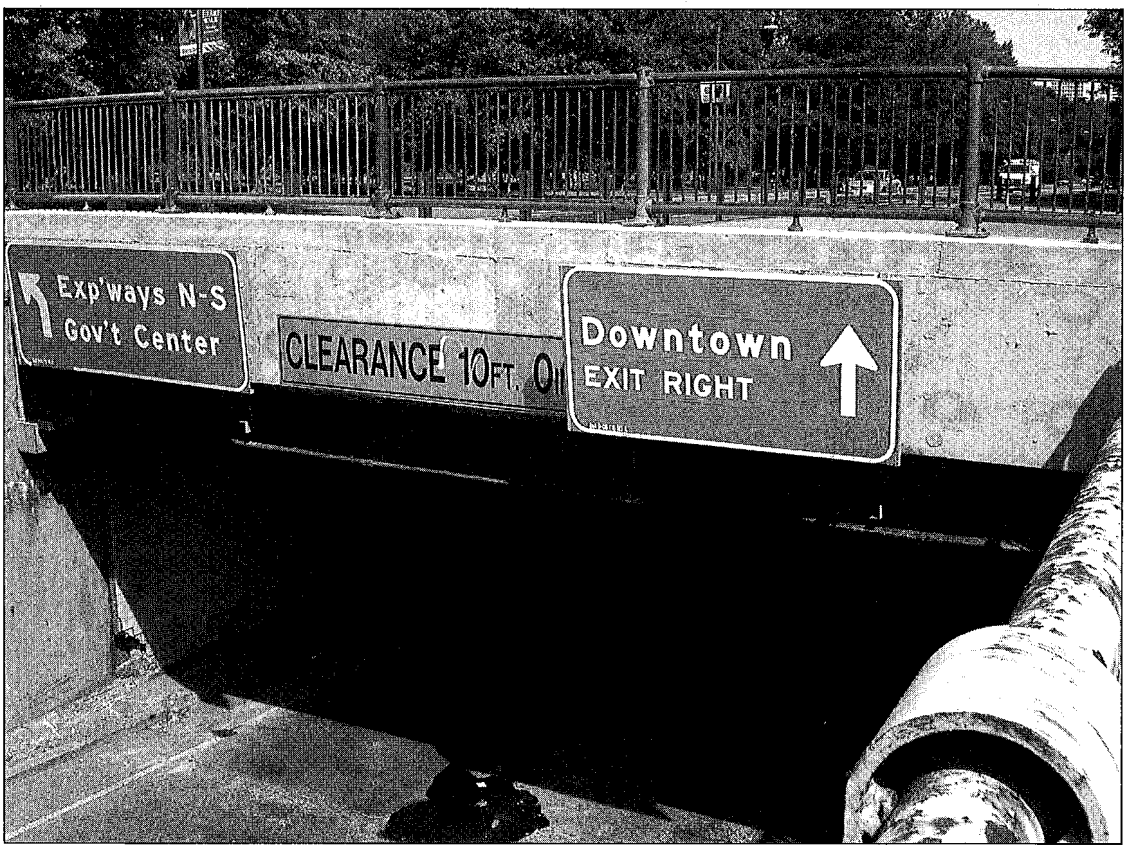

FIGURE 6. West Portal after repairs.

- Measurement of the deflected shape of the steel roof beams (variation from a straight line between beam seats) and of the invert slabs.

The 1992 work also included the following analyses:

- Live-load rating of tunnel roof slabs and roof beams. (The tunnel roofs are considered to be bridges in the Commonwealth of Massachusetts' inventory of transportation structures.)
- Structural analysis of tunnel walls and invert slabs under the effects of dead loads, hydrostatic pressures and soil pressures.

The following were the main findings from the 1992 work:

- The original design did not provide for the amount of asphalt and earth overburden load that is currently on the steel beam portions of both tunnels. However, this weight provides significant additional resistance to overall tunnel float-out. However, the additional weight produces stresses in the steel beams that exceed the normal design allowable stress. Some steel roof beams have experienced significant section loss due to corrosion. Thus, the roof loads must be reduced, or the beams must be strengthened, or a combination of these measures must be developed.
- The top midspan reinforcement in the invert slabs and the outside wall reinforcement are subject to excessive tensile stress. The invert slabs have an inadequate safety factor to resist the effects of the upward hydrostatic and foundation bearing pressures because the outside wall reinforcement was not extended far

FIGURE 7. Bolted steel retrofit of a fatigue-prone diaphragm connection.

enough above the base slab to develop the restraining edge moments assumed in the design of the invert slab.

- The thick reinforced concrete roof slabs of the eastern portion of main tunnel require restoration at all expansion joints because they are severely deteriorated, with disintegrating concrete and large lengths of reinforcing bar hanging free of embedment in concrete. The loss of reinforcing at these locations reduces the strength of the slabs in this section of the tunnel. Also, the conditions at these joints pose a threat to traffic below from falling concrete and reinforcing steel.
- The walls of the open boat sections require restoration because they have extensive cracking with some leakage and unsightly efflorescence. Four wall panels have extensive disintegration of concrete surfaces (see Figure 8).
- The pumps that remove drainage water from the tunnels' two wet wells are old, in poor condition and require replacement.
- The lighting system needs to be upgraded.

The 2001 Re-evaluation & Rehabilitation Studies. The 2001 re-evaluation included the following additional structural investigations:

- Performing ground penetrating radar (GPR) surveys of the topside of the tunnel roofs and of the soffit of the main tunnel's roof slab to determine pavement thicknesses and comparative concrete conditions.
- Extraction, testing and petrographic evaluation of twenty-four additional cores from roof slabs at areas that had been surveyed by GPR.
- Determinating pavement thickness on the invert slabs by impact-echo tech-

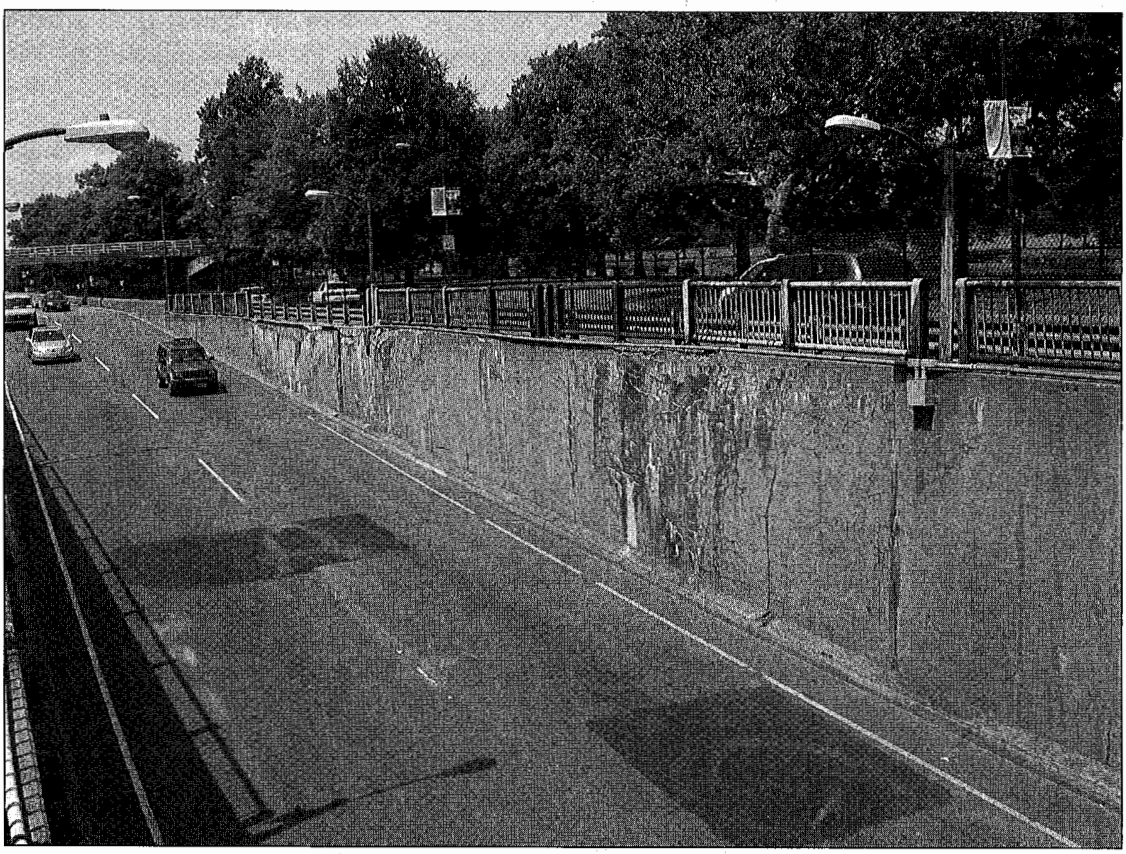

FIGURE 8. Northwest boat section wall and railings before repairs.

niques (correlated by representative pavement cores).

- Examination of a tunnel wall for concrete cracks, using spectral analysis of surface waves (SASW), at areas indicated to be overstressed to the point of plastic hinge formation by the 1992 analyses. (The cracks were expected to be located on the back surfaces of the wall and test pits were initially attempted to allow visual inspection of these areas, but the practical problems and cost of making deep test pits below the groundwater table in the Esplanade adjacent to the active westbound parkway proved insurmountable, so the non-destructive SASW technique was used instead.)
- Making twenty-five soil borings adjacent to the tunnel, installing groundwater observation wells at these locations and performing frequent monitoring of groundwater levels at these wells.

The following were the main findings from the 2001 work:

- The majority of the roof slab concrete appears to be in relatively good condition, with no large-scale or widespread areas of delaminations or exposed reinforcing bars resulting from corrosion or concrete deterioration. The causes of the present areas of deterioration and the main threats to the long-term durability of the concrete are chloride-related corrosion, damage from freezing and thawing, and alkali-silica reaction. These conditions exist mainly in the areas of roof expansion joints, construction joints and through-cracks. The concrete could be repaired at these locations by removing the affected concrete and replacing it. Even with this replacement, future repairs at the edges of the repaired areas and in the repairs themselves must be

anticipated at regular (ten- to fifteen-year) intervals in the future.

- The concrete in areas that are not presently distressed may become distressed in the future, mainly due to the relatively high chloride contents in the concrete. The present deep cover is preventing widespread corrosion by limiting the oxygen availability to the reinforcing bars. To further prevent corrosion at the majority of the structure, the elimination of water sources from the concrete is the best preventative measure that can be taken at this time. The elimination of water will also help to prevent or limit future damage due to alkali-silica reaction, as well as freezing and thawing.
- A contractor to the engineering consultant performed SASW, a non-destructive stress-wave-based test, on the inside surface of the north wall to detect the suspected cracks. Access to the inside surface of the wall was gained through two 17-inch wide by 64-inch high openings in the false-tiled wall. The locations of the two tests coincided with sections of tunnel walls predicted to be seriously overstressed in the 1992 analyses. Although a horizontal crack in the back of the wall (the side against the soil) was expected due to the predicted overstressed condition, the SASW tests found no cracks, indicating that the results of the 1992 analyses may have been overly conservative regarding the formation of plastic hinges in the tunnel walls and the resulting high bending moments in the tunnel invert slabs.
- Groundwater levels adjacent to the tunnels were generally around elevation 103 feet, or about 4 to 5 feet below the normal maintained level of the Charles River Basin.

The 2001 work also included a study of several concepts for rehabilitating the tunnels, which were presented in the 2002 report.

The 2006 & 2007 Re-inspections & Condition Investigation Update. The 2006 inspection focused on providing data to update the condition assessment of the concrete roof slabs and the steel roof beams. Many new areas of delaminated concrete were identified on the underside of the roof slabs, which prompted extensive chipping operations to remove loose concrete (following the tragic ceiling collapse in Boston's I-90 tunnel that summer). Updated measurements of remaining thicknesses of flanges and webs of the roof beams led to the realization that the original shape of the steel wide-flange sections included slight thickness tapers of the flanges. This new information led to a re-assessment of the 1992 estimates of section losses, which were found to be slightly exaggerated. However, the new data also indicated that, for several roof beams below leaking expansion joints, the rate of section loss between 1992 and 2006 was greater than the corrosion rate prior to 1992.

The 2007 inspection concentrated on assessing the condition of the concrete retaining walls in the boat sections at the entrances and exits of the tunnels. Previous inspections had not obtained much data in this regard, because of the understanding that these walls would essentially be reconstructed as part of a major rehabilitation project. However, with the delay of the major project, the crumbling condition of portions of the boat section walls became a primary safety concern. To determine if the service life of these walls could be effectively extended, and to quantify the extent of the required repairs, a program of non-destructive field testing — correlated with laboratory material testing and petrographic examination of concrete cores — was carried out on several of these walls. The engineering consultant used impulse-response (IR) techniques to efficiently obtain condition data on large areas of walls that had highly varied levels of deterioration as determined by visual inspection. By testing and examining cores extracted from areas showing different conditions, it was determined that areas showing little visual evidence of deterioration were likely not experiencing significant hidden distress below the surface, and that visible signs of deterioration were a good indicator of serious distress in the underlying concrete.

Additional soundings of the underside of the concrete roof slabs made during the 2007

inspection revealed that a significant amount of additional concrete delamination had occurred since the 2006 inspection and remediation, indicating that roof slabs were rapidly deteriorating in vulnerable areas around expansion joints, construction joints and cracks where water was leaking through the roof. Together with the ongoing concern for the development of additional fatigue cracks in the roof beams, this observation led the DCR to implement monthly inspections of the tunnel roofs.

Developing the Scope of Interim Repairs

Three levels of service life were considered for an interim-repair project: eighteen months, five years and ten years.

An eighteen-month plan would be intended to address the most immediate structural safety concerns, but not to provide significant upgrades of the tunnel's structural, waterproofing, lighting, drainage or pumping systems. This plan would require the immediate resumption of the environmental permitting process and final design of the major rehabilitation project. An underlying assumption in the adoption of an eighteen-month plan was that the major tunnel rehabilitation project would precede other major projects in the Charles River Basin — most notably the Longfellow Bridge restoration and the Craigie Drawbridge and Craigie Dam bridge replacements — that were also in preliminary design and going through their own environmental review processes.

A five-year plan would provide significant upgrades to the tunnel's major systems, but would not bring these systems into full compliance with current design codes and modern standards for new tunnels. This plan would allow the environmental permitting process to be delayed for a few years while an evaluation was conducted of a synchronized planning approach to all the major bridge and infrastructure projects in the Charles River Basin, from the Elliot Bridge to the Craigie Dam. This option would probably allow the Craigie and Longfellow bridge projects to be constructed before the major Storrow Drive Tunnel rehabilitation project.

A ten-year plan would also not bring all the tunnel's major systems into full compliance with current design standards, but it would provide the maximum feasible improvement to these systems in the context of a partial-rehabilitation program. In addition to strengthening all deficient structural systems, this plan would replace all electrical, lighting and pumping systems with new components. It also would include provision of a new groundwater recharge system along the full length of the southern side of the tunnels. Most of the other major infrastructure rehabilitation projects in the Charles River Basin could be accomplished before the tunnel project under the ten-year plan.

The Analysis of Costs & Risks

To evaluate the relative costs and benefits of these three interim plans, the DCR and its design consultants prepared a matrix of costs and another related matrix of corresponding risks. These two matrices were intentionally limited to one page each so they would be useful to top-level planners and policy-makers as well as the engineers at the project management level. As such, the matrices included only the following broad categories under which all related information was consolidated:

- Tunnel Roof (Beams & Slabs)
- Tunnel Walls
- Boat Section Walls & Invert Slabs
- Tunnel Invert Slabs & Under-Drain System
- Mechanical Systems — Pumping & Groundwater Recharge
- Lighting System
- Traffic & Noise Control
- Engineering
- Ongoing Structural Inspection Program

Within each main category, no more than eight major sub-items were included for the overall evaluation of costs and risks. The cost estimating for this exercise was necessarily approximate, but was helped by taking a contractor-style, bottoms-up approach to the material, labor, equipment and incidental costs. These bottoms-up estimates were roughly verified by updating the costs associ-

ated with the extensive quantification of major work items that had already been accomplished during the evaluation of rehabilitation concepts in 2001-2002. These cost estimates were as follows:

- Eighteen-month plan: $1.8 million
- Five-year plan: $11 million
- Ten-year plan: $23 million

The risks were addressed qualitatively on a scale that included Low, Medium, and High risks associated with deferring the major sub-items of work, and just a few shades of "grey" between these major classifications. Risks to public safety were given top priority, but the matrix also included other types of risks, such as the potential for further litigation related to groundwater levels.

Using the risk matrix made it instantly clear to planners and decision-makers why some repairs could not reasonably be deferred for even eighteen months, but rather had to be done as "immediate" work, such as the retrofitting of the fatigue-prone connections in the steel roof beams and the cleaning/repairing of the pavement-embedded under-drain system. It also showed moderate to high levels of risks to public safety for deferring certain structural repairs and waterproofing upgrades for even five more years, which is not surprising since the matrix was composed by the same engineers who had already warned that the tunnel was likely to experience a localized structural failure within that time frame.

The Peer Review Process

During the evaluation process by the DCR and its design team, voices from outside the project-design team were promoting the idea of delaying a major rehabilitation project for at least ten years. These were primarily the voices of those who believed that Storrow Drive could be eliminated, or have its traffic capacity significantly reduced, without adversely affecting the city and regional transportation systems. These groups seemed convinced that it was wasteful to spend millions of dollars on rehabilitating or replicating a tunnel and interchange system that they believed represented the poor transportation/urban planning of a 1950s automobile-centered mentality. Members of these groups included a former governor, a former state secretary of transportation, and a former associate commissioner of MassHighway. With such formidable critics, the DCR decided to subject the design team's work on the long-term project, and on the interim-repairs project, to a rigorous peer review process.

The DCR hired a team of local engineering firms to review the technical approaches that the design team had used, and they also retained a national strategic/logistics-planning firm to conduct a more quantitative and comprehensive risk analysis of the options the design team had prepared for both the long-term and interim projects. The DCR and the design team met with the peer reviewers and made copies of all project-related documents, from 1992 to the present, available to them.

The Selection of the Five-Year Plan

Using a utility-valuation approach that considered such factors as public safety, economic efficiency, parkland preservation, financial feasibility and regulatory compliance, the peer reviewers concluded that the most favorable long-term alternative was to robustly rehabilitate the tunnel system in its current configuration (i.e., the enhanced Option A). They also concluded that beginning construction of the major rehabilitation within eighteen months was the most favorable timeframe, but that waiting five years (after performing the necessary interim repairs) was a close second. The peer review team also made some specific technical recommendations for improving the effectiveness of the interim repairs. Considering all the recommendations from its internal staff, its consulting design team, its peer-review team and the outside voices, the DCR decided to proceed with the five-year interim-repairs plan, incorporating the technical recommendations of the peer reviewers.

The final scope of repairs and system upgrades in the five-year plan included the following major work items:

- Waterproofing the boat section walls of the inclines leading into and out of both

tunnels to reduce leakage of groundwater through these walls.

- Repairing damaged concrete in the boat section walls.
- Installing new steel railings on top of the boat section walls.
- Waterproofing the expansion joints and construction joints in the tunnel roof slabs from the topside.
- Repairing damaged concrete on the underside (soffits) of the tunnel roof slabs.
- Cleaning and painting steel roof beams at expansion joints.
- Removing the tile-faced "false" masonry walls in the main tunnel.
- Upgrading the lighting system in the main tunnel to the maximum extent possible within the constraints of the existing electrical supply system.
- Coating the concrete structural walls in the main tunnel with a white, breathable paint system — mainly for increased brightness and reflectivity in conjunction with the new lighting system.
- Providing a permanent connection of the tunnel's west pump station to existing under-utilized groundwater recharge chambers below Back Street. (The Boston Water and Sewer Commission installed the chambers in 2007 during a water/sewer upgrade project along Back Street, with the intention that they could be used for accepting discharges from the tunnel drainage system.)
- Utilizing a noise-monitoring and noise control system during construction. All construction was specified to occur during overnight periods, except for jackhammering activities on the surface road, which could only occur between 10 A.M. and 2 P.M.
- Developing a traffic management plan for use during construction.

Designing the Interim Repairs

Concrete Wall Waterproofing. The design team consulted with several waterproofing contractors and relied on its experience with designing and evaluating other below-grade waterproofing projects to determine the optimal program for waterproofing the concrete walls. The selected system consisted of hydrophilic polyurethane grout for injection into cracks and joints, and hydrophobic polyurethane grout for general injection through the boat section walls along their entire length, up to the maximum groundwater level expected during the service life of the repairs (see Figure 9). Both forms of grout are injected through ports in holes drilled through the concrete walls. The hydrophilic grout fills any voids in the cracks and joints and is intended to extend slightly into the soil behind the wall in localized areas around the cracks and joints. The hydrophobic grout is injected at regularly spaced ports and is intended to form a curtain wall of grout behind, and in intimate contact with, the concrete walls.

Concrete Repairs to Boat Section Walls. At all boat section walls that had existing railings that needed replacement, the design team chose to demolish a minimum of 18 inches of the tops of the walls, regardless of condition, and rebuild them with adequate dowels and new reinforcement to accept the new crash-tested railings. In addition, deteriorated concrete on vertical wall surfaces was specified to be removed to a minimum depth of 2 inches behind the existing outer mat of reinforcing steel.

The design team chose to require hydrodemolition for the preparation of the vertical concrete surfaces that needed to be repaired. This requirement was based on the team's experience with various surface preparation methods and on research that indicates that even small chipping hammers leave microcracks and abrasions in the prepared concrete surface, which adversely affects the bond of the new concrete to the existing concrete; whereas hydrodemolition results in much less surface damage and has the potential for a better bond. No bonding agent was specified; the concrete surfaces simply had to be wetted to a saturated, surface-dry condition immediately prior to the placement of new concrete.

New reinforcing steel was called for where existing bars were seriously corroded, and in all repair areas a mesh of galvanized welded

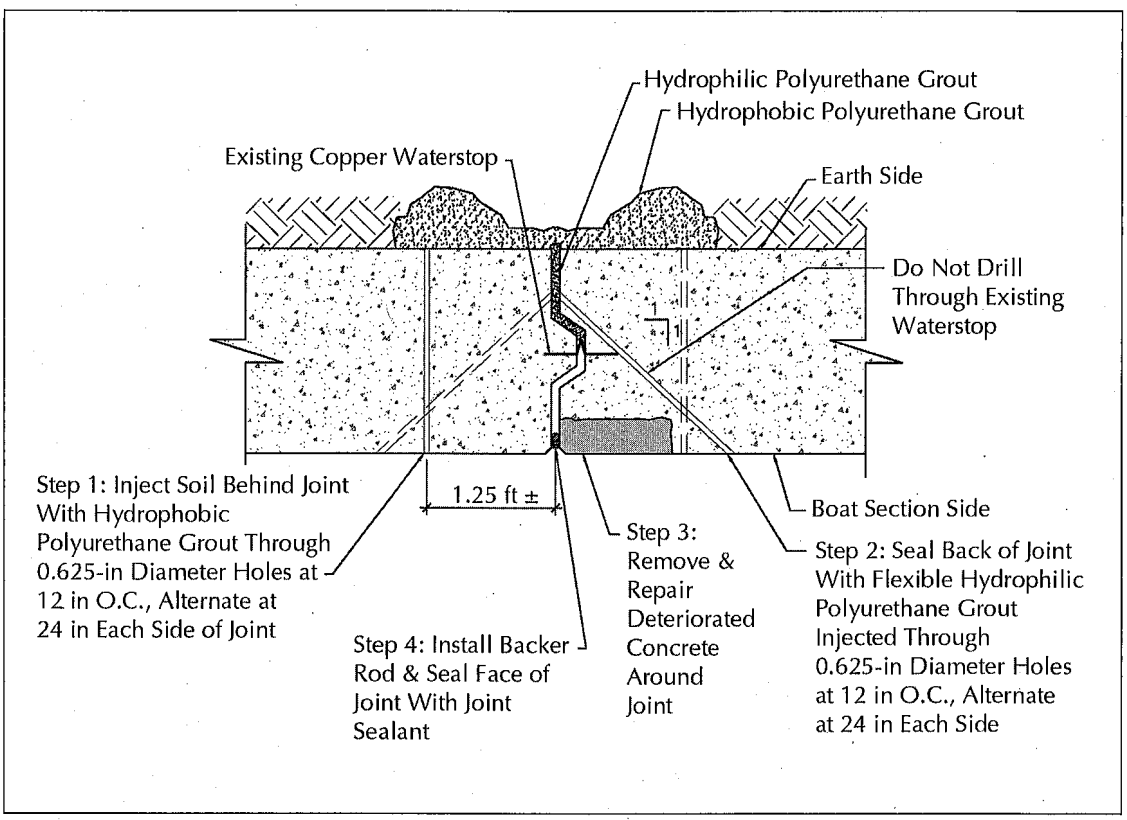

FIGURE 9. Polyurethane waterproofing detail at wall joints.

wire reinforcement was specified to be added on the outside face of the existing mat of reinforcing. Any existing steel reinforcing bars exposed and retained in the repair areas were specified to be blast cleaned and coated with a zinc-rich paint. New bars were specified to be uncoated and left black, although the contractor chose to also coat these bars with the zinc-rich paint.

The specified repair concrete for small, shallow wall repairs was a premixed, bagged specialty concrete, which includes shrinkage-reducing agents in its mix. The state's normal 4,000 pounds per square inch, 0.75-inch aggregate concrete was specified for areas having larger placement volumes, such as the top-of-wall and through-wall repairs. All concrete repairs in the walls were specified to be formed, with placement to be acheived by pouring the concrete (for the large-volume repairs) or by either pouring or form-and-pump methods at shallower vertical surface repairs.

Steel Railing Replacements. The existing steel railings consisted of I-section posts, top and bottom pipe-section rails, and closely spaced vertical pickets between the rails. These railings were in various states of disrepair: some sections with severe impact damage had been previously replaced with highway guard rail, some sections with less severe damage had been left in place and some sections had no damage other than general corrosion. The existing railings appeared to have poor geometry in terms of impact resistance, and to have no particular historic significance, so the design team chose to replace all the existing steel railings wherever there was a real need to prevent vehicles from crashing through the railing at the top of a boat section wall and falling onto the roadway below. The new railing is Massachusetts' standard S3-TL4 bridge railing, which is crash tested to a performance level that is conservative but appropriately safe for the Storrow Drive parkway. This railing has I-section posts and vertical pickets

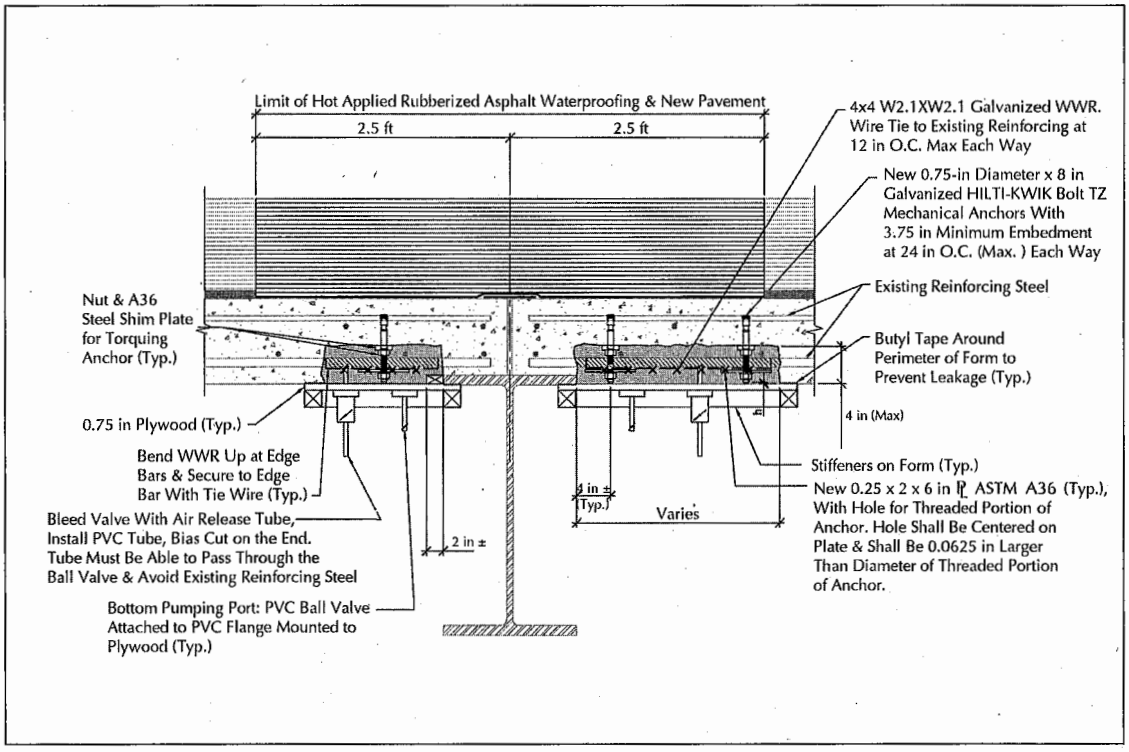

FIGURE 10. Partial depth repair detail at the concrete roof slab.

similar to the old railing, with the only significant visual difference being the extra horizontal rail (three total, which are square tube sections) on the new railing.

Concrete Roof Soffit Repairs. Hydrodemolition was also specified for surface preparation of soffit repairs in the concrete roof slabs. Deeper soffit repairs were called to be accomplished in a manner similar to the vertical surface repairs in the boat section walls (see Figure 10). Shallow repairs that did not fully expose the bottom layer of reinforcing bars were also allowed in the soffits. The same premixed specialty concrete specified for wall repairs was specified for the soffits. Only form-and-pump placement methods were allowed for all types of concrete soffit repairs, with a suggested method shown on the contract plans.

To reduce the chance of soffit repairs debonding and falling onto the roadway below, the design team called for mechanical anchors to be installed in the existing substrate concrete at all soffit repairs, whether they were deep or shallow. The anchors were either the undercut type, which were used wherever the required embedment depth could be achieved, or a tension zone expansion type (tested and approved for use in cracked concrete) where sufficient depth was not available for undercut anchors. Small steel plates attached to the bottom end of these anchors supported a mat of welded wire reinforcement that resulted in a mechanical system that would hold the patch in place if the bond between the patch and the substrate failed. Although the calculated load on each anchor was small, and would only be present in the event of a bond failure of the patch, the designers called for 100 percent load testing of the installed anchors to a test load of two to four times the maximum design load. Chemical anchors were not allowed as substitutes for the mechanical anchors.

Topside Waterproofing at Roof Joints. The designers selected a roof joint waterproofing system that could be installed during the short work windows available during nighttime lane closures of Storrow Drive westbound. A more elaborate, redundant and durable joint-

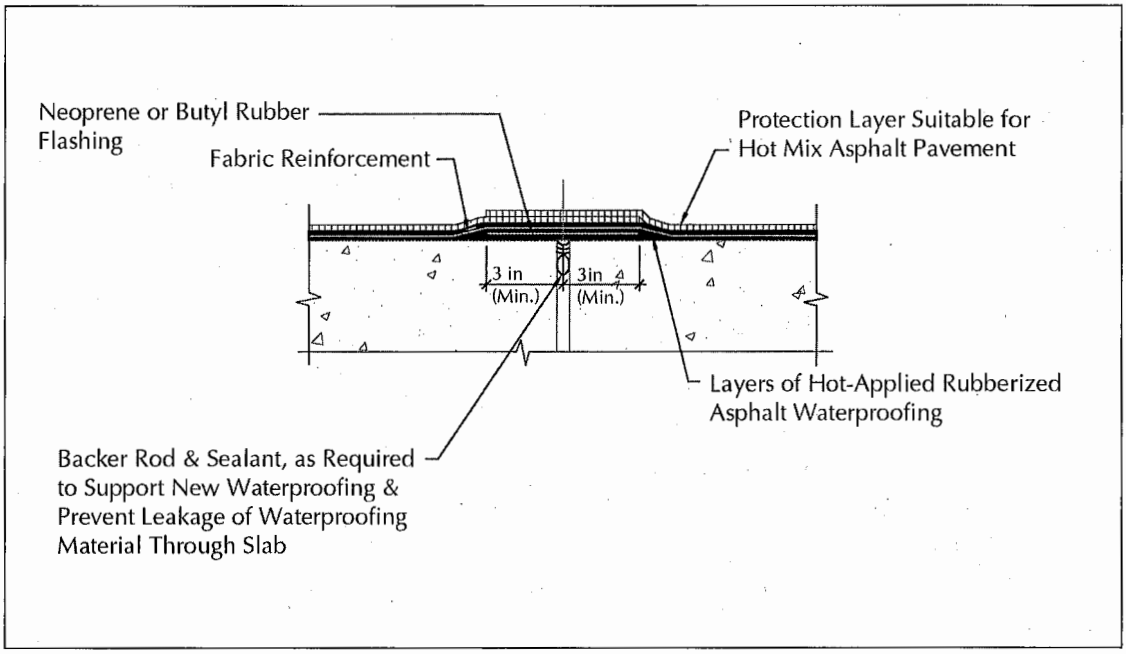

FIGURE 11. Waterproofing detail at the roof slab expansion joints.

sealing system was considered, but was not selected due to the short-term nature of the repair and the practical limit on installation time. The selected system was a hot-applied rubberized asphalt waterproofing system that incorporated a flexible flashing to accommodate expansion joint movement and fabric reinforcement to provide added strength and durability at both construction joints and expansion joints (see Figure 11). This system could be effectively installed during the time available to excavate and patch the areas over the joints during the overnight lane closures. It was also tolerant of the expected surface imperfections in the top of the concrete roof slabs.

The Cleaning & Painting of Steel Roof Beams at the Expansion Joints. General cleaning was required to remove dirt, debris and loose material from all steel roof beams. For roof beams below the expansion joints in the roof of the main tunnel further washing and mechanical cleaning were specified to achieve at least a power-tool level of surface preparation (SP-3 per the Society for Protective Coatings [SSPC] standards). The most recently installed paint system was still mainly intact on roof beams away from the expansion joints, but it had almost completely failed at the expansion joint beams, which had suffered significant section loss due to corrosion of the unprotected steel. Therefore, only the beams at the expansion joints were specified to be painted with a high-performance paint system, and the other beams were not to be painted during the intended five-year service life extension of the interim repairs.

False Wall Removal. The design team had originally considered only making a few exploratory openings in the tile-faced masonry "false" walls during the five-year interim-repairs project. However, inspections of these walls during the repair work preceding the interim project revealed that a few sections of the false wall were tilting outward, were possibly restrained from further tilting only by incidental contact with the lighting cable raceway, and could be pushed back away from the raceway by hand. Those sections were immediately stabilized with temporary anchors and strong-backs, but this discovery gave the final impetus to the decision to entirely remove all the false walls during the interim-repairs project. Doing so would finally allow a complete visual inspection and evaluation of the leaking concrete walls that had been concealed for

so long. The design team recognized that the false walls in the main tunnel may have had a beneficial effect in protecting the concrete walls from water and salt being splashed on them from the roadway, but the walls in the small tunnel showed no signs of deterioration from this long-term exposure, so full removal of the false walls was specified.

Tunnel Wall Painting. Along with the decision to remove the false walls in the main tunnel came the opportunity to improve the visibility for drivers by increasing the brightness in the tunnel, which would be accomplished primarily by improving the lighting system. However, it was decided to paint the exposed walls with a bright white coating to enhance the effectiveness of the new lighting and to provide improved aesthetics. Of course, the surface of the concrete walls would have to be properly repaired and cleaned to provide a good substrate for any coating system, and there were many unknowns associated with this work due to the presence of the false walls. However, the DCR chose to deal with these uncertainties during the interim-repairs project rather than further delaying the project in order to remove the false walls and evaluate the structural concrete walls. The specified coating was an environmentally friendly mineral (silicate) exterior paint system with a reflective additive (glass beads) to enhance reflectivity. The chosen system does not form a vapor or moisture barrier and is breathable to avoid the possibility of its blocking leaking water and thus forming blisters or bubbles in the coating. The designers specified rigorous procedures using prototype mock-up panels to demonstrate the effectiveness of surface preparation and material application procedures used by the contractor.

Performance Specification Items

Several major items of work were not designed prior to awarding the construction contract; rather, the design team prepared performance specifications that required the contractor's engineering team to prepare the final detailed plans for the work described below.

The Lighting System Upgrade. The tunnel's original lighting system had gradually deteriorated and failed over the years, and the replacement system was installed in more or less of an ad hoc, piece-meal basis. The result was a system that had inadequate levels of light for daytime conditions at the entrance portal and an outdated electrical distribution system. The design team solicited proposals for designing a replacement lighting system that would be fully compliant with current lighting and electrical codes, but the DCR decided that this approach was too expensive and not required for the five-year interim-repairs plan. Instead, the DCR specified a particular lighting fixture to use for the replacement system, and the designers prepared a performance specification requiring these fixtures to be arranged in the tunnel to maximize the light levels in the tunnel within the limitations of the existing power service, while not exceeding maximum levels as recommended by the Illuminating Engineering Society (IES) Standard, IES RP-22, for tunnel lighting. The contractor was required to have a professional electrical engineer survey existing lighting levels and prepare design plans for the placement and wiring of the new fixtures. A testing program of the new system was specified to demonstrate acceptable levels of illumination, system safety and compliance with applicable electrical codes.

The Connection of the Tunnel Pump Discharge to the Back Street Groundwater Recharge System. The design team's studies related to groundwater levels in the area of the tunnel had led to the conclusion that returning the water from the tunnel's drainage system to the surrounding soil would probably only have minimal effects on these groundwater levels. The Boston Water and Sewer Commission tested their Back Street recharge chambers in 2007 by pumping water from hydrants into the chambers over an extended period. The volume of water pumped each day into the chambers was approximately equal to the average volume pumped daily from the tunnel's west wet well into the Charles River; unfortunately, this pumping had no significant effect on raising groundwater levels in the area. Nevertheless, the potential good will of Back Bay property owners and the environmental benefits to be gained by pumping tunnel drainage water into recharge chambers rather than into the

Charles River were judged to be too significant to not attempt improving existing conditions by making the connection between the tunnel pump station and the Back Street recharge system. However, the design consultants and the DCR could not come to an agreement on the best way to make this connection, and the proper way to have it environmentally permitted, in time to complete the design and permitting before the planned advertising date for the interim-repairs project. Therefore, the location and intent of the connection was described in the contract documents, as a performance specification, and the contractor was required to employ a registered professional engineer to prepare the final detailed design, and to obtain the necessary environmental and other permits, for construction of the connection and its control systems.

The Noise Evaluation & Control Plan. The DCR prepared performance specifications for the interim-repairs project that it adapted from recent similar projects in which construction noise was effectively monitored and controlled. The specifications required that noise monitoring and development of a detailed mitigation and control plan be done by an experienced acoustical engineer. Noise barriers were required in specific work areas, such as at the top of boat section walls being repaired and at locations of the topside roof slab waterproofing, and their composition had to conform with barriers that had been used successfully on other noisy projects in urban settings. Maximum noise levels that could not be exceeded were described for each type of construction equipment that might be used on the project, with certain maximum levels tied to the ambient noise levels in the surrounding community that were to be established by a comprehensive noise-monitoring program. The overall approach to noise control as specified was similar to, and adapted from, the procedures that had been used successfully in Boston during the Central Artery/Tunnel Project.

The Traffic Management Plans. The traffic management plan developed by the consulting team included a detour for eastbound full-closures of Storrow Drive that used city of Boston streets. This detour would only be used between the hours of midnight and 5 A.M., and it was the most direct route for traffic heading into downtown Boston. However, the DCR rejected the use of this detour and required eastbound traffic to be detoured to Memorial Drive to avoid the use of city streets. The consulting team would not endorse the Memorial Drive detour, so the final design of the traffic management was handled with a performance specification that required the contractor to prepare the final eastbound detour plans in accordance with DCR direction. The DCR also decided not to allow the westbound full closure of Storrow Drive and its associated detour that had been designed using city streets. The traffic management plans for single-lane closures designed by the team's traffic engineering consultant were retained in the contract documents, and the contract was written with language requiring bidders to assume that all work would be performed during single-lane closures, with no change in bid prices if the DCR decided to allow full roadway closures.

Constructing the Interim Repairs

The construction contract was advertised with the design provisions described above. After the initial low bidder withdrew its bid, the $11.5 million contract was awarded to the same contractor that was working on the DCR Ramp H reconstruction project at the Bowker Interchange on Storrow Drive, just to the west of the Storrow Drive Tunnel.

The Partnering Approach. During the design phase of the project, the DCR had requested that the design team provide full-time on-site engineering presence during all non-routine repair work, which included most of the work in the project. The purpose of having this engineer on site was not to be a substitute for the DCR's resident engineer but, rather, to assist the resident engineer and provide immediate technical advice instead of having to coordinate technical questions among night-working construction personnel and day-working engineers. The engineering consultant provided this field engineer, who agreed to work the night shift for the nearly year-long construction project. Despite initial concerns expressed by the collective bargaining unit representing state engineers and sci-

entists, the DCR's resident engineer and the engineering consultant's field engineer worked so effectively with each other and the contractor that all parties involved in the project agreed that the field engineer's nearly continuous presence was a welcomed and valuable contribution that significantly aided the project's success.

DCR, contractor and consultant personnel conducted weekly site meetings to address engineering, construction and management issues during the initial months of the project. These meetings proved valuable for planning and scheduling the upcoming construction and monitoring work, and kept all parties informed and aware of issues with the ongoing work. With all parties having an on-site presence during execution of the work, there were few surprises, and when unforeseen conditions were encountered or disputes arose concerning how to best perform the work, these issues were more easily addressed and resolved in a generally uncontentious manner by this group of professionals who had learned to work together "in the trenches" as a partnering team.

Wall Waterproofing Repairs. The contract documents required manufacturer's representatives to assist the contractor in establishing acceptable installation techniques for waterproofing the walls and roof slabs. Over the course of a few nights, the waterproofing manufacturer and the contractor worked out specific procedures for effectively injecting hydrophobic and hydrophilic polyurethane grout. The presence of the liquid asphalt from the 1950s waterproofing attempts resulted in some unexpected installation issues especially for the hydrophobic grout. The old asphalt seemed to cause the new hydrophobic grout to travel quickly under its installation pressure to the top of the wall, and the initial installations seemed to produce more grout uselessly squirting out from behind the wall than penetrating into the soil behind the wall to form an effective grout curtain. This condition was investigated by taking concrete cores through a few areas where the grout had been installed. At all locations, a thin layer of hydrophobic grout was present at the back of the wall, but, unexpectedly, no asphalt was encountered; rather, material beyond the grout was a fine cohesive soil. Liquid asphalt was oozing out of old form tie holes at a few locations adjacent to where cores were taken, so it was clear that this material was still present behind the wall, but it was not clear exactly where it was or how it was affecting the grout installation. The contractor and the manufacturer's representative used the results of these trial installations and investigations to develop appropriate locations, sequences and procedures for installing both the hydrophobic and hydrophilic grouts to most effectively stop the active leaks. These procedures did not completely stop all leaks, but it was estimated that approximately 90 percent of the active leaks were stopped. The effects of the old liquid asphalt on the long-term effectiveness of the wall waterproofing remain unclear, especially given the observation of the seasonal change in the viscosity of the asphalt with temperature changes, but the design and construction team felt that the combination of the relatively rigid hydrophobic curtain wall and the relatively flexible hydrophilic filling of localized cracks and joints would provide the best chance of long-term waterproofing success.

Concrete Wall Repairs. The contract documents called for robotic hydrodemolition for the full depth of concrete removal at areas of walls that had to be repaired. Hand-held lances were only allowed for localized clean-up of areas not easily accessible by the robotic equipment. The contractor proposed alternate methods: using chipping hammers to remove the outer deteriorated concrete; and using hydrodemolition lances — and not robotic equipment — to accomplish the final 0.5 to 1 inch of concrete removal. The design team required the contractor to demonstrate the effectiveness of these alternate concrete demolition methods in a trial area of the first wall, which happened to be the most deteriorated boat section wall. The designers and the DCR were satisfied with the results and the contractor fine-tuned the demolition procedures using a combination of chipping and hydrodemolition on the next production areas on the first wall.

The designers believed that the contractor was being overly aggressive in chipping away

concrete in some of the subsequent production areas on the first wall. Structural evaluation of concrete removal near the bottom of the taller sections of this wall revealed that it would be safe to perform general removal of up to 6 inches of the outer surface, but the contractor had removed localized areas up to 8 inches in depth. The design and construction team agreed to limit further demolition work to the 6-inch maximum depth. However, larger than expected areas needed general removal of this amount of concrete, and the designers were concerned about the stability of these large patch areas if bond failure occurred in the future. To address these concerns, the contractor installed hooked epoxy-adhered dowels spaced no more than 3 feet on center to provide a more reliable mechanical connection between the substrate and the large patch.

The larger than expected areas of wall repairs made the use of the bagged, premixed specialty concrete impractical for the vertical surface repairs. The contractor proposed using the regular 4,000-pounds per square inch, 0.75-inch mix for the vertical surfaces and for the top-of-wall replacement, to be formed and poured in a series of sections along the length of the wall with no horizontal construction joint between the vertical surfaces and the top of the wall. The designers allowed this implementation, as long as a specific arrangement of control joints was provided, the length of pours along the wall did not exceed 40 feet, placements were coordinated with existing construction joints and a shrinkage-reducing admixture was added to the regular concrete mix.

The contractor also proposed to use a mobile, volumetric concrete mixer and a concrete pump, rather than regular ready-mix trucks, to supply the concrete to the site during the nighttime work shifts (see Figure 12). The DCR and the design team required the contractor to conduct trial mixing of the proposed concrete, with the shrinkage-reducing admixture, in the proposed mixer at an off-site non-production location. The initial trial was not completely successful: the mobile volumetric mixer did not seem to provide adequate mixing of the concrete materials, and the resulting concrete test cylinders exhibited excessive bleed water. In addition, the results of several batches using the mixer settings the contractor had pre-established to produce the specified properties were not consistent, especially with respect to slump and air content. The contractor performed a second trial, this time using a steeper angle on the mixer's dispensing chute and sending the mix through the proposed concrete pump before taking samples for testing. This trial was successful in demonstrating the contractor's ability to consistently produce concrete with the specified properties. However, it was clear from the demonstrations that the settings on the volumetric mixer would need to be calibrated on a nightly basis, which the contractor agreed to do with the assistance of an independent concrete-testing laboratory. The production use of the mobile volumetric mixer was generally successful, after the contractor and its subcontractors figured out that they needed to have sufficient supplies of sand and stone stored at the site to eliminate the possibility of running out of concrete during a large pour.

To allow placement of the larger than expected areas of concrete, while limiting the free-fall of the concrete within the forms, the contractor devised a stepped forming system that allowed form "windows" to be temporarily removed during placement of lower sections and put back in place without disrupting the placement or producing an inconsistent finish on the wall. The windows were required because the pump hose was too large to fit into the space between the forms/reinforcing and the existing concrete substrate. The system was not ideal, but it was successful in limiting the concrete free-fall to no more than 4 feet.

Concrete Roof Soffit Repairs. The contract requirements for participation in trial installations by manufacturer's representatives for roof soffit repairs were similar to those for the wall repairs. The design team had experience with the specified undercut and tension-zone expansion anchors during the previously performed "immediate" repairs in the tunnel and from other projects. The contractor and the manufacturer did not agree that 100 percent anchor testing was necessary, but this requirement was strictly enforced and none of the anchors failed the test during the course of the

FIGURE 12. Wall repair concrete placement using the mobile mixer.

project. The contractor experimented with methods for saturating the concrete substrate, which is an essential requirement for successful overhead repairs using the form-and-pump method. In just a few trials, the successful methods were identified and the work proceeded efficiently using the specified specialty concrete that included a shrinkage reducer in the premixed components.

Hydrodemolition was successfully used to remove deteriorated soffit concrete and prepare the substrate for the form-and-pump repairs. Similar to the wall repairs, the soffit repair surface preparation was accomplished with a hand-held lance rather than robotic equipment. Unlike at the wall repairs, the operator of the hydrodemolition lance could not stand on the roadway while performing the work on the roof soffit. On the roadway, the operator could stabilize himself against the force produced by the 10,000 to 15,000 pounds per square inch pressures at the lance tip. But, while working from a mechanized work platform, there was inadequate built-in resistance to these forces, so the contractor devised an adjustable, rotatable stand for the lance that resisted these forces so the operator just had to concentrate on aiming the lance, which was critical because a misfired waterjet could be lethal at the operating pressures being used. However, the rotatable stand proved prone to frequent operational breakdowns, so the contractor devised an alternate method to stabilize the work platform that allowed a very muscular operator to safely brace himself against the pressure of the hydrodemolition lance.

Topside Roof Joint Waterproofing. In 2007, the DCR had their on-call maintenance repair contractor perform a trial installation of the hot-applied rubberized asphalt waterproofing system over the west-most expansion joint in the roof of the main tunnel. This joint was not one that had shown excessive leakage or deterioration to the roof slab or steel beam below it (although some of these problems were evident), but it was located entirely within the median of the interchange, which

FIGURE 13. Waterproofing and protection board at a roof slab expansion joint.

made it ideal for a trial application since traffic control was not an issue (see Figure 13). The trial used one of the three similar systems that were specified for this work in the interim-repairs contract, and it was generally successful. The concrete roof at this joint, over Beam 10 (the tenth from the west portal), was in fair to good condition and was easily prepared to receive the waterproofing system by complete removal of the old asphalt and membrane-waterproofing system, followed by cleaning and flame-drying of the exposed concrete surface. The original joint-filler/forming material was still in place at this joint, and it was providing unexpectedly good resistance to leakage through the joint, at least during the warm weather that was prevalent during the trial installation. The only significant problem that occurred during this installation was at the southerly end of the joint, where a fairly large tree had grown in the soil over the south wall of the tunnel. The soil was excavated close to the trunk of this tree, but the trunk was not undermined in order to not damage the tree. Doing so left the last 2 to 3 feet of the joint unprotected by the new waterproofing system. At the designer's suggestion, the contractor injected polyurethane grout into the joint in this otherwise unprotected area, although there was no good way to verify that this last area of the joint was filled. However, later observations from inside the tunnel did not show significant leakage though any part of the roof expansion joint at Beam 10.

The roof joint waterproofing work in the interim-repairs contract closely followed the example set by the 2007 trial installation, although it was performed by a different subcontractor who selected one of the other alternate proprietary systems that had been specified. The next joint in line to be waterproofed was an expansion joint over Beam 18. This joint also unfortunately had a tree growing

over the south end, which was dealt with in a similar fashion to the trial installation. This joint was also almost completely contained within the median and was waterproofed with similar ease and success as the Beam 10 location. However, as the work continued from west to east, the construction team encountered increasingly difficult work conditions as the joints extended partially or completely under active roadways.

During the design phase, it was anticipated that some of the pavement removal work in these areas would need to be done with jackhammers, which the DCR had agreed not to allow during night work due to their loud operating noise, which might disturb neighbors' sleep. In addition, due to traffic constraints, the daytime use of jackhammers was limited to the hours between 10 A.M. and 2 P.M. The interim-repairs contractor chose not to try to work within that narrow daytime window and decided to use less noisy equipment to remove the pavement over the joints during the regular nighttime work shifts (9 P.M. to 5 A.M.). This equipment consisted mainly of backhoe-mounted digging and scraping tools, which were not as precise in removing specific areas of pavement as jackhammers. Even though these excavation methods required more pavement to be removed at each location, they met the requirements of the strict noise control plan and were able to accomplish the excavations in time for the waterproofing to be installed and the area to be backfilled with compacted asphalt over a complete lane-wide work area in a single night shift. The only significant problem caused by the time restrictions on the roof waterproofing work was when significant concrete deterioration was discovered in the top of the roof slab. The narrow work window did not allow for complete preparation and repair of these defects and, therefore, some small areas of loose concrete simply had to be removed and flooded with the rubberized asphalt rather than properly repaired. This less than optimal repair procedure was limited to just a few locations of shallow concrete removal and each instance was reviewed and approved by the on-site structural engineer from the design team. Having the design engineer on site allowed the contractor to proceed with confidence in the structural integrity of the remaining roof slab.

The work was generally successful in stopping existing leakage, but when a few areas below newly waterproofed joints were observed to be wet from inside the tunnel, the contractor used polyurethane grout injection from the bottom side to attempt to seal these few problem areas. In total, after all roof repairs and waterproofing work was completed, the final results of the construction efforts were judged to have stopped about 90 percent of the leakage through the roof — about the same success rate as for the boat section walls.

The Cleaning & Painting of the Steel Roof Beams. The contractor requested a substitution of surface preparation and coating systems for the seven steel beams below the expansion joints in the main tunnel roof. Instead of power-tool cleaning and a conventional zinc-rich primer, epoxy intermediate coat and urethane top coat, the subcontractor involved with the cleaning and painting proposed ultra-high-pressure waterjet cleaning followed by the application of a moisture-cured paint system consisting of a mica-containing zinc primer and a urethane top coat. The DCR had recently used this system on a few bridge-painting projects and were happy with the results. The designers and contractor agreed that it was a suitable system for the intended five-year service-life extension, so this substitution was allowed with no change in the contract bid price for cleaning and painting. The subcontractor performed the cleaning and painting work during the full closures of the eastbound roadway through the tunnel. The work was performed from a vehicle that enabled complete containment of the water spray and removed materials above the footprint of the vehicle during the cleaning operations to remove the lead-based paint. The vehicle was moved sequentially along the length of a beam to complete the cleaning operations in two or three night shifts per beam, and the painting operations followed the cleaning work as closely as possible. The subcontractor was careful to follow the manufacturer's instructions for applying the moisture-cured coating system, because moisture-cured sys-

A view of the finished project, with the repaired boat section walls and new railings at the west approach.

tems (similar to other coating systems) cannot be applied effectively on wet surfaces, and the conditions inside the tunnel were highly variable with respect to dew points and the formation of moisture on the steel beams.

The Connection to the Groundwater Recharge System. The contractor retained a civil engineering firm to design the connection to the Back Street groundwater recharge chambers. The design of the mechanical system to make the connection was relatively straightforward; the new connecting pipes were routed between the steel roof beams in the main tunnel. But the permitting process was even more involved and time consuming than the design team anticipated, and the proposed design was not approved until about ten months after the construction contract was awarded. The construction of the connection was completed in a few days.

The Noise Control Plan & Its Implementation. The contractor hired an experienced acoustical engineer to develop the noise control plan. The acoustical engineer performed the specified monitoring program to determine ambient noise levels during day and night conditions and prepared a report describing the program to control construction noise below the specified maximum levels. The contractor installed noise barriers adjacent to work areas. The barriers consisted of standard concrete median barriers (Jersey barriers) with 8-foot-high fences mounted atop them. The fences had medium density overlay (MDO) plywood panels attached to them, with sound-absorbing blankets attached to the panels. Early in the construction project, there was one complaint lodged with the project concerning excessive noise during the night work. Although the noise-monitoring program showed that the established maximum noise levels were not being exceeded, the contractor made a few simple adjustments to the night-time operation of construction equipment to further lower noise levels, and there was not another official noise complaint for the remaining duration of the project.

The Traffic Control Plan & Its Implementation. The firm designing the groundwater recharge system was also retained by the contractor to prepare the traffic management plan for the full eastbound closure of Storrow Drive during the nighttime work shifts. The detour directed eastbound traffic to Memorial Drive via the River Street Bridge. Initially, only single-lane closures were used, but the anticipated benefits of full eastbound closures for expediting the work inside the tunnel and boat sections could not be ignored, so the eastbound full closures were implemented first on a trial basis and finally on a permanent basis. The night shifts generally started with a single lane closure at 9 P.M., and the full closure was imposed as early as 11 P.M., though sometimes later if a Red Sox game ended late. The single-lane closures would generally back traffic up to about the Massachusetts Avenue bridge for the first hour or so, but traffic volumes usually decreased enough between 10 P.M. and 11 P.M. so that the single lane of traffic flowed relatively smoothly, though slower than usual, through the tunnel during the final hour before the full eastbound shut-down. Only single-lane closures were used on the westbound roadway. Also, the eastbound and westbound on-ramps from Berkeley Street were frequently closed to allow work over the tunnel on these ramps. The potential negative effects of the partial and full shut-downs of Storrow Drive and its ramps were alleviated in part by the DCR's program of issuing traffic advisories via the Internet and other news media so that few drivers were surprised by the lane closures and detours.

Additional Structural Evaluations Performed During Construction

During the course of the interim-repairs project, as portions of the structure were exposed and observed in ways never before possible, the DCR and design team observed that some structural conditions were not as poor as previously supposed. These observations prompted the DCR to ask the design team to further evaluate these conditions, especially related to the feasibility of further extending the service life of the repaired tunnel beyond the previously intended five years.

Metallurgical Evaluation of the Steel Roof Beams at Intermediate Diaphragm Connections. During the installation of the fatigue retrofits at the terminal connections of the diaphragms to the steel roof beams, which actually occurred slightly prior to the start of the interim-repairs project, the engineering consultant performed metallurgical inspections of the roof beams at the intermediate connections of the diaphragms. All connections of the terminal diaphragms to the roof beams had been retrofit with bolted plates and angles before or during the metallurgical inspections. However, DCR inspectors observed radial lines of rust that emanate into the roof-beam web from many of the intermediate diaphragm top-flange-to-web connections (where diaphragms frame into both sides of a roof beam). Fifteen locations were identified where significant anomalies, such as deep pits or sharp grooves, exist in the surface of the steel near the toe of welds connecting intermediate diaphragm top flanges to the webs of the roof beams. The engineering consultant determined that these anomalies are likely locations of stress raisers and could possibly initiate stress-induced cracks or distortion-induced cracks in the roof beam webs, but the radial lines of rust that emanate from many of these connections are not likely signs of debilitating distress.

Load Tests of Roof Beams in the Main Tunnel Near Beam 52. During construction of the interim repairs, the engineering consultant and its instrumentation subcontractor performed live-load tests and monitoring of ambient environmental conditions of the main tunnel roof near Beam 52 as a demonstration of the feasibility of a more extensive program of long-term monitoring of the roof structure. The results of this work showed that stresses induced in the steel roof beams by ambient traffic loading are very small. Also, the maximum, live-load, strong-axis-bending stresses induced by a 15-ton test truck were much lower than would be expected for a non-composite steel beam. These small bending stresses indicate full composite action is occurring between the deck slab and Beam 52, at least for the static conditions tested. From comparisons of strain measurements at midspan and at the end of this beam, it was estimated that

the fixity provided by the beam's attachment to the wall results in a live-load negative moment at the end of the beam that is about 8 percent of the corresponding live-load positive moment at midspan. Measured out-of-plane strains and corresponding stresses in the roof beam webs due to live loads were very small; however, strains and stresses in the webs due to large ambient temperature changes were more significant.

Following the load tests, the instrumentation was left in place for six months as a trial installation of a longer-term monitoring program of the tunnel's roof structure.

Structural Analyses of the Roof to Correlate Field Measurements. The engineering consultant performed computer-based analyses of the tunnel roof system (beams, diaphragms and slab) and correlated the analyses with the field results of the load tests and monitoring program to develop a tool that could be used to evaluate the short-term and long-term structural performance of the highly stressed roof system. The structural analyses of the roof generally confirmed the findings of the instrumented load tests, except that the bending moments, deflections and stresses predicted by the computer model are about 20 to 30 percent higher than the measured responses from the load tests. The sources of this discrepancy between measured and predicted results remain unclear.

The analyses confirm the relatively wide distribution of individual vehicle wheel loads to several roof beams, as indicated by the load-test results, which is likely assisted by the rigidly connected diaphragms. The strong-axis bending stresses from live load predicted by the computer model are small compared to the computed dead-load stresses. Computed out-of-plane bending stresses in the roof beam webs due to displacements caused by temperature changes were of a similar magnitude to stresses computed from the measured strains during the instrumented load testing and monitoring program. Although these stresses are significant, they are not of a magnitude that would be expected to cause structural distress in the absence of stress raisers and distortion-induced fatigue. Unfortunately, these adverse conditions do exist at a few locations (as described above).

Inspection, Repair & Coating of Concrete Walls in the Main Tunnel (Previously Hidden by False Walls). After the false walls were demolished and removed, engineers conducted visual inspections and performed hammer sounding of the structural concrete tunnel walls. The walls had vertical cracks regularly spaced at 10 to 20 feet along their full length. Many of these cracks were relatively wide, had been repaired before the false walls were constructed and showed signs of long-term leakage. Other vertical cracks were narrower, dry and showed no evidence of leakage. Most of the construction and expansion joints showed signs of long-term, extensive leakage, indicating that the copper waterstops used in the original construction were not effective. These wider cracks, and the joints, had been injected with liquid asphalt before the false walls were constructed, and the signs of leakage indicated that this waterproofing attempt was generally ineffective.

The engineers found only small, isolated areas of hollow-sounding concrete, which were marked for repair as part of the interim-repairs project. Since the condition of the tunnel's structural walls was generally better than expected, the DCR decided to add this repair work as an extra work order to the interim-repairs contract, with the intention of extending the service life of the tunnel even longer than the previously intended five years. Additional waterproofing by grout injection was also added to the interim-repairs project to address the leaking cracks in the walls. Approximately $3 million additional wall repair work was performed to correct the localized concrete distress and to waterproof the cracks and joints in the tunnel walls.

After the wall repair work was performed, the walls were cleaned and coated with the specified mineral-based white paint. The contractor prepared the required mock-up sections showing coatings with and without the specified glass beads. It was determined from these mock-ups that the beads did not consistently enhance the reflectivity and aesthetics of the coating system. The areas with beads also tended to hold dirt particles more adherently than areas without the beads, so the production coating was performed without the glass bead additive.

Discussion of Additional Investigations & Repairs. From previous investigations, and from the results of the additional investigations described herein, the DCR and designers expect that a ten-year extension of the tunnels' service life can probably be achieved if the following issues are addressed during special annual inspections and maintenance:

- Leakage of groundwater through tunnel walls and through invert slabs of boat sections and tunnels.
- Further corrosion and potential yielding of steel in the highly stressed roof beams in the main tunnel and the small tunnel.
- Potential cracking of webs of roof beams at the fatigue-prone connections of the intermediate diaphragms to the roof beams in the main tunnel and the small tunnel.

The design team was less concerned with the thick, reinforced concrete roof slabs in the eastern portion of the main tunnel that are slightly overstressed from the effects of dead load only, but which would have nearly the AASHTO-prescribed ultimate-strength capacity if some overburden were removed. Further inspection and testing will need to focus on overhead spalls in the thick slabs, and in the thinner slabs in the steel roof beam section of the tunnel. Monitoring of long-term deflections of the tunnel roofs was recommended as part of the special inspections and during a long-term instrumented monitoring program.

Conclusions From Additional Investigations. The repairs to the tunnel walls will likely mitigate significant water-related damage to these walls for the next ten years.

The overstressed conditions of the tunnel roofs can be greatly improved by removing some of the dead-load overburden if further evaluation of construction risks and float-out risks prove positive. The dead-load stresses in the steel roof beams will still exceed the AASHTO allowable stresses, but, given the favorable results from testing to date and the apparent composite behavior, frequent inspection of all beams and instrumented monitoring of selected beams should allow sufficient management of the risks associated with the overstressed conditions for the next ten years.

The steel-cracking risk associated with the fatigue-prone details in the roof beams can also likely be managed for the next ten years by instrumentation, monitoring and frequent inspection.

The reduction of dead load will likely allow the thick roof slabs in the eastern portion of the main tunnel to meet the current AASHTO ultimate-strength (LRFD) design criteria for reinforced concrete bridge members.

The Next Steps

Because the steel roof beams in both tunnels will remain highly stressed due to the effects of dead load during the period between the interim repairs and the proposed major rehabilitation or replacement project, the DCR asked the design team to further evaluate the risks for this period and to develop a plan for mitigating any significant risks that are so identified. These risks and the recommended mitigation are:

- To address the risk of steel yielding and excessive deflections of roof beams, perform additional load testing and long-term automated monitoring of tunnel roof beam strains, temperatures and displacements. This work will expand on the trial load tests and monitoring that were performed during the interim-repairs project to demonstrate the feasibility of the data-collection equipment and data-acquisition/reduction systems. The long-term monitoring is proposed to be complementary to the continued frequent inspections of the tunnel roof, not a substitute for this regular inspection.
- To address the risk of a decreasing factor of safety against float-out of the tunnel structure, perform long-term monitoring of groundwater levels near the tunnels using automated or semi-automated equipment. The success of the waterproofing repairs and contribution of tunnel drainage to the Back Street recharge system could possibly have the beneficial effect (for nearby property owners) of raising groundwater levels in the vicinity of the tunnel. The negative effect of the possible change is to reduce the factor of

safety against float-out of the tunnel, and particularly the uncovered boat section structure near the main tunnel's west portal. In order to respond to any changes in this factor of safety, the proposed system will allow monitoring of the affected groundwater levels using electronic gages placed into the array of observation wells that had already been installed.

- To reduce the dead load on the tunnel roofs, evaluate and possibly implement permanent dead-load reduction by the removal of some pavement and other overburden from over the tunnel roofs. This proposal requires careful evaluation of the existing grading and drainage regime in the roads and ramps over and around the tunnels. Construction loads on the tunnel roofs must also be closely scrutinized. However, even a few inches of net pavement removal would significantly reduce the dead-load stresses in the overstressed roof beams.
- Continue evaluating the transportation needs of the entire Charles River Basin, and determine how and when the permanent tunnel rehabilitation or replacement best fits into these overall needs. The evaluation, design and permitting processes must continue over the next five to ten years so that the permanent project can proceed when the life span of the interim repairs "runs out."

As of this writing, the semi-automated groundwater-monitoring equipment has been installed, the program of load testing and monitoring is under development and the final design of the pavement removal and regrading is underway.

NOTES — *The engineering consulting firm Simpson Gumpertz & Heger Inc. (SGH) evaluated and re-evaluated the tunnel's condition, assessed options for its rehabilitation and designed the selected interim repairs option. Infrasense, Inc., subcontractor to SGH, performed GPR surveys of the top and bottom surfaces of the tunnel roofs. Olson Engineering, Inc., subcontractor to SGH, performed SASW on the inside surface of the north tunnel wall to detect suspected cracks. Geocomp Corporation, subcontractor to SGH, provided the instrumentation system for load testing and monitoring the tunnel roofs. Members of the SGH design team included Paul Kelley, Atis Liepins, Gregory Imbaro and Diego Arabbo. Subconsultants to SGH included the following: BETA Group, Inc. (traffic engineering); Bryant Associates, Inc. (roadway civil engineering); Carol R. Johnson Associates (landscape architecture); CDW Consultants, Inc. (environmental permitting); Epsilon Associates, Inc. (MEPA process permitting); GEI Consultants, Inc. (geotechnical engineering); Jacobs Engineering Group (tunnel ventilation studies); Peter Martin Consulting (cost estimating); Regina Villa Associates, Inc. (public participation facilitation); and SEI Companies (mechanical, electrical and plumbing). The contractor for the interim repairs project was SPS New England. The specified repair concrete for small, shallow wall and soffit repairs was premixed BASF LA40 PMAC repair mortar. The selected wall waterproofing products were manufactured by DeNeef. The hot-poured rubberized asphalt products for waterproofing the topside of the tunnel roofs were manufactured by American Hydrotech and by the Henry Company. The contractor retained the engineering consulting firm Vanasse Hangen Brustlin, Inc., to design the connection to the Back Street groundwater chambers, as well as to prepare the alternate traffic management plan for the full eastbound closure of Storrow Drive during nighttime work shifts. The author wishes to thank David Lenhardt, DCR Supervisor of Bridges and Parkways, for reviewing this article. Lenhardt managed the tunnel project for the Commonwealth of Massachusetts for many years.*

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The Evaluation of Chloride Contamination in Reinforced Concrete

Selecting the optimal method for reducing or preventing the corrosion of reinforcing steel in concrete structures depends on a range of factors.

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Reinforced concrete makes use of steel to provide the tensile and bending strength needed for concrete structures. The steel helps prevent the failure of concrete structures that are subjected to tensile and flexural stresses due to dead, live, wind and temperature loads. However, when the steel reinforcement corrodes, the formation of rust leads to a loss of bond between the steel and the concrete, and results in subsequent delamination and spalling. The cross-sectional area of steel can be reduced, leading to a reduction in strength capacity. If left unchecked, the integrity of the structure can be affected. This reduction in strength capacity is especially detrimental to the performance of tensioned strands in prestressed concrete. The main cause of the deterioration of reinforced concrete is the corrosion of the reinforcement steel, usually caused by either carbonation or chloride contamination of the concrete. This deterioration of reinforced concrete occurs in two stages: commencement and propagation. The commencement and propagation of corrosion in reinforced concrete structures depends on many factors: permeability of concrete, cover thickness, environmental aggressiveness, type of loads and duration of application, design errors, reactants, etc.

Concrete covers the rebar, providing them with a non-corrosive environment because of its high alkality. When corrosion is initiated by chloride contamination, it causes a reduction of concrete cover. Corrosion causes more deterioration in low-quality concrete and in concrete with poor compaction and poor curing. Once the chloride penetrates through the protective concrete cover, corrosion is initiated if the chloride concentration exceeds a critical value.

Corrosion of reinforcement results in the formation of rust that has two to four times the volume of the original steel. The increas-

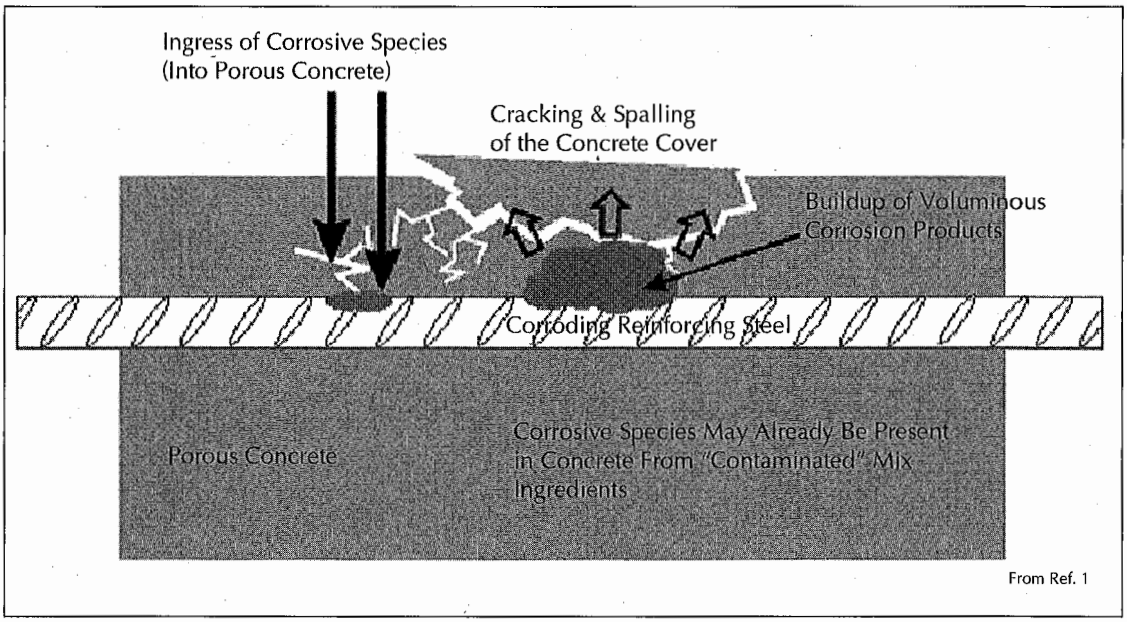

FIGURE 1. Corrosion damage in reinforced concrete.

ing volume of rust causes concrete tensile stresses that may be the main cause for internal microcracking, external longitudinal cracking and eventually spalling (see Figure 1). Corrosion also produces holes in the surface of the reinforcing steel, reducing strength capacity due to the reduced cross-sectional area. All these factors may lead to serviceability failure and/or loss of structural integrity.

Rebar corrosion has been blamed for concrete deterioration, especially on bridge decks. Currently, the most commonly used repair technique has been the mechanical removal and replacement of chloride-contaminated concrete, which is inherently destructive. Replacement treats only symptoms. The other treatment methods deal with the actual cause(s) of corrosion. For example, one other commonly used non-destructive repair method is half-cell protection.

Methods for Determining the Extent of Reinforcement Corrosion

Visual Inspection. Visual inspection (VI) includes all unaided inspection and evaluation techniques that use only very basic tools (for example, flashlights, sounding hammers, tape measures, plumb bob, etc.).

VI is a process of examining and evaluating systems by using sensory systems with only mechanical enhancements to sensory input, such as magnifiers and probes. The inspection process can be done just by looking, listening, touching, smelling and applying some load. It depends highly on the inspector's knowledge of the structure and the inspector's past experience.

The accuracy of VI relies on five factors:

- Subject factors;
- Physical factors;
- Task factors;
- Environmental factors; and,
- Organizational factors.

Subject factors include the inspector's visual acuity, color vision, eye movement, age, experience, personality, sex, intelligence, training, etc. Physical and environmental factors cover lighting, aids (magnifiers, overlays, viewing screen, closed circuit television, automatic scanner, etc.), background noise, workplace design, etc. Task factors include inspection time, pace, density of observations, spatial relationship of observations, fault probability, the object's complexity, etc. Organizational factors range from the number of inspec-

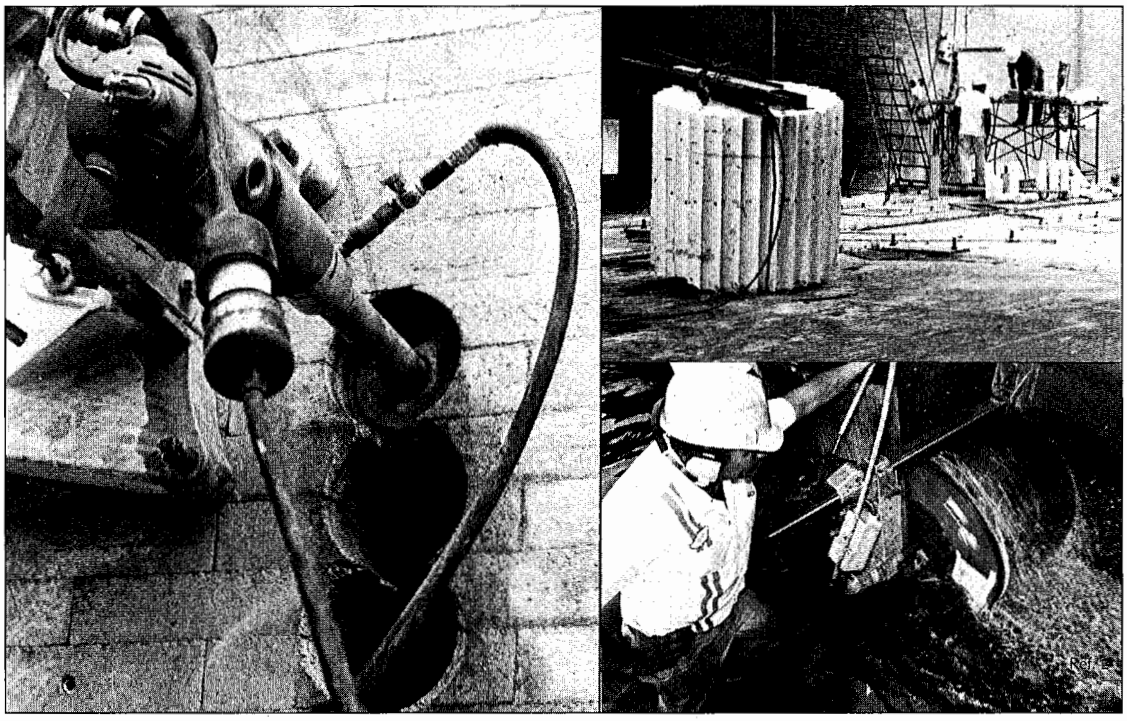

FIGURE 2. Side-by-side or stitch coring (left); extracting one large core (top right); and in standing water (bottom right). (From Ref. 2.)

tors, their instruction, supervision, training, standards, budget, alertness, to social factors such as motivation, incentives, etc.

An advantage of VI is that it is inexpensive and easy to execute.

Disadvantages include:

- All the external environment factors such as the light, color, wind speed, traffic and the inspectors' visual acuity and training can all have negative effects on VI accuracy. Even the choice of inspection tools will affect accuracy.
- Further, VI has limitations due to its own nature. It cannot be used to identify small in-depth cracks and time-dependent fatigue.

Coring. Core drilling is an economical, quick and clean process whose application has greatly expanded in recent years. Special equipment enables the drilling of holes up to 48 inches in diameter (see Figure 2). Concrete core drill bits can range in diameter from 0.5 to 72 inches (1.25 to 183 centimeters) in diameter, and drilling depths can be virtually unlimited due to the use of extensions. A completely solid and cylindrical boulder or core of concrete is removed from the hole after the drilling has been completed. Concrete coring is most commonly used for what is known as a utility penetration.

A typical core drilling could have slabs being drilled under load using a 3-inch (75 millimeter) diameter diamond drill bit. If water is used as a coolant, it can affect the vibrating wire gauge readings. Depending on the ambient humidity and dryness of the concrete, the cooling water could also cause the concrete, and in particular the core, to expand.

Core drilling methods are used widely in underground utilities construction, most commonly for manhole taps, underground vault taps and wherever sewer, water, steam, air or communication lines pass through a concrete or brick structure.

An advantage to core drilling is that it is a fast and direct way to observe the interior corrosion status of a structure.

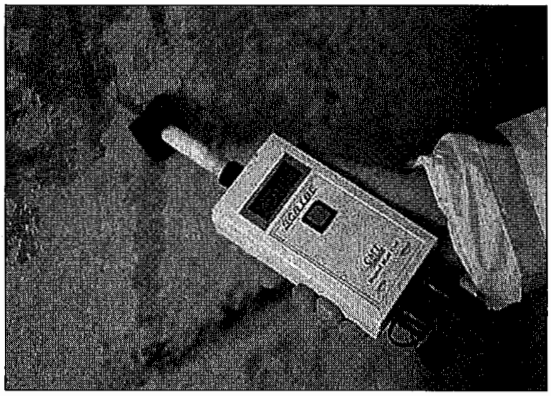

FIGURE 3. Half-cell potential testing equipment. (Courtesy of Hammond Concrete Testing, Ref. 3.)

The disadvantages of core drilling are:

- It will likely harm the structure's integrity: high-impact vibration causes microfractures of concrete. (There also is the potential for damage to nearby structures and equipment.)
- The work environment is usually not very comfortable or that brief: loud noise; extremely dusty and time-consuming clean-up.

Electrical Resistance Testing. One of the most common electronic methods of monitoring corrosion is using the electrical resistance (ER) technique. This technique measures the change in electrical resistance of a conducting element, and then uses this datum to calculate the corrosion rate. ER monitoring is an applicable measurement technique in either wet or dry conditions.

ER probes operate on a straightforward basis. As the surface area of a conductive metal element subject to corrosion is reduced, its conductivity will be lowered and its electrical resistance increased. Readings from the sensing element are relative to a non-corroding reference element sealed within the body of the probe. Small changes in resistance can be detected by a sensitive measuring instrument.

A separate reference element allows testing of the internal integrity of the probe. Through careful design, the reference element also serves in eliminating thermal variations. Since temperature variations can have an effect on resistance, the reference element is subjected to the same temperature as the exposed element, which ensures consistent measurement.

Probe elements are made of a material similar to that of the pipe or vessel in which it is placed in order to simulate as closely as possible the corrosive environment. By taking periodic readings over a fixed time interval, the rate of metal removal can be determined and a corrosion rate in mils/year can be calculated.

The advantages of ER consist of:

- ER probes are the most flexible of the electronic probes in terms of their possible applications. Because the technique does not depend on an electrochemical reaction, a current-carrying electrolyte is not necessary.
- ER monitoring can be used to measure virtually all environments, such as solids, non-aqueous liquids, gases and vapors — all of which can be of high or low conductivity.
- ER probes can rapidly indicate changes in corrosion rates, and will show when the changes took place, which is helpful in evaluating process decisions.

Half-Cell Potential. Normally, concrete will protect its reinforcing bars but if chloride penetrates into the concrete, it will deteriorate. Therefore, early monitoring before cracking can be observed visually is very important. The half-cell potential measurement technique is a non-destructive one for evaluating and estimating corrosion in reinforced concrete (see Figure 3). This technqiue also yields the probability of corrosion. Its drawback is that this method measures potential on the surface of the concrete and not on the surface of the bars. After employing the half-cell potential method, polarization resistance can be used to determine the corrosion rate.

In order to determine the corrosion rate, numerical methods like the boundary element method (BEM) or inverse boundary element method (IBEM) can be used. BEM is used for distributions and current flows of rebar and

the IBEM is used for experimental results to identify the corrosion states in order to compensate for half-cell potential. The half-cell potential method is more successful than BEM but IBEM will yield a better result.

If the concrete is assumed to be homogeneous, the potential can be obtained by using the BEM method at the boundary:[4]

$$cu(x) = \int_S \left[G(x,y)\frac{\partial u}{\partial n}(y) - \frac{\partial G}{\partial n}(x,y)u(y) \right] dS$$

And by using the IBEM method at the interface of the concrete and rebar:

$$u(x) = \int_S \left[G(x,y)\frac{\partial u}{\partial n}(y) - \frac{\partial G}{\partial n}(x,y)u(y) \right] dS$$

Where:

$u(x)$ = the potential;
c = the shape factor;
y = the point on the boundary, S, of concrete; and,
G = fundamental solution.

Two reinforced concrete slabs were tested — one of them an ordinary concrete slab and the other with voids.[4] A portable corrosion meter with a silver chloride reference electrode was used to measure the resistivity, polarization rate and half-cell potential of the concrete on its surface. The test was done in dry and wet conditions. The half-cell potential was measured after 28 days of concrete immersion in fresh water. Then the specimens were dried and put in a sodium chloride solution. Afterwards, the concrete covering the rebar was physically removed in order to visually note the rate of corrosion.

Corrosion proceeds in three stages: no visible rust, little rust and heavy rust. A visual inspection of the specimen can be used to determine the corrosion stage. The distribution of surface potential can be influenced by the positioning of the electrode located inside the concrete and near the rebar. The values of cathode and anode would be the same but the values around or at the center of the voids can differ and rebar located near voids can experience more corrosion than the measurement data would indicate.

Half-cell potential measurements provided negative potentials under the wet condition in both concrete specimens. In the intact slab, the corrosion area only occurred in the right corner and some parts of the rebar were corroded, with a little corrosion commencing in other areas. In the other specimen with the voids, negative potentials were found on the right edge and all of the rebar was corroded.

By comparing the visual inspection and the results of the half-cell potential measurement, it can be concluded that they are in poor agreement. In the intact specimen, the visual inspection noted that there was minor corrosion observed near the anode. Also, there was no corrosion on the rebars of anodic and cathode areas of the voids specimen. Using half-cell potential without compensating for it by applying IBEM can yield false results. Therefore, IBEM should be used to refine half-cell potential by taking into account the resistivity of concrete around edges and voids. In that way, the more negative values obtained by half-cell potential, in comparison with actual corrosion, can be avoided.

ASTM C876 — 09 Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete can be used at any time during concrete member life. This test can be used to estimate the electrical corrosion potential of uncoated reinforcing steel. However, solely relying on half-cell data for determining what bridge deck areas need repair is not recommended. The ASTM criteria for half-cell potentials can be used for the interpretation of data but the criteria should be modified with more data to take into account specific site conditions. The Washington State Department of Transportation, in cooperation with the Federal Highway Administration, tested bare concrete decks with different evaluation techniques, including half-cell potential.[5] The test was done with copper electrode immersed in copper sulfate solution. The magnitude of electrical potential was measured to determine the state of corrosion of the reinforcing steel. Normal differences in conditions, ground location and variant seasonal surveys that represent questionable corrosion were not significant. The half-cell potential was not sensitive to these factors.

The flow of electrical current from one metal to another depends on their natural

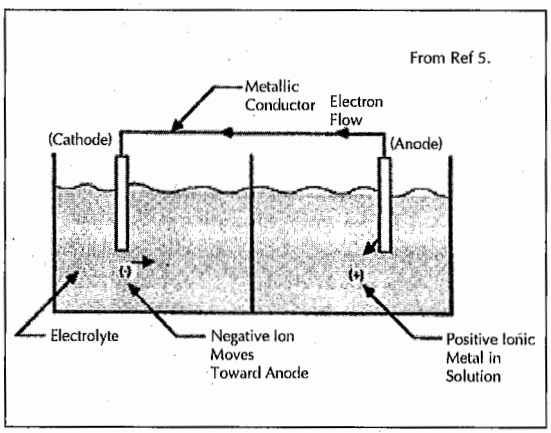

FIGURE 4. A basic corrosion cell.

electromotive force. The anode emits electrons and the cathode receives them, with corrosion forming in the anode. With both metals connected, an electrical flow can be observed, creating a corrosion cell (see Figure 4).[5]

Samples with half-cell potential measurements from copper-copper sulfate that are more negative than –0.35 volt have a 90 percent possibility of corrosion. Samples that are less negative than –0.2 volt have a 90 percent possibility of passivity. Data between –0.35 and –0.2 volt are attributed to uncertain conditions.[5]

Many highway agencies use this method for determining bridge deck conditions, together with other evaluation techniques. But there are two concerns with its use. The first concern centers on the reliability of the half-cell potential method and the second deals with the dependence of half-cell data with concrete deterioration and chloride contamination. However, the deterioration of concrete and the observation of rust in bars can confirm the presence of corrosion. Also, it should be noted that not all corrosion is destructive (causing deterioration) and not all corrosion activity will result in heavy corrosion.

The advantages of the half-cell method include:

- Identification of areas of potential corrosion;
- Testing is conducted using light-weight, portable equipment; and
- The method is inexpensive and quick.

The disadvantages to the half-cell method include:

- Requires direct connection to reinforcement;
- Electrical continuity of the reinforcement in the structure is required (therefore the method is not suitable for epoxy-coated bars); and,
- Concrete needs to be moist.

Linear Polarization. Due to the widespread corrosion of reinforcing steel in concrete structures, there has been a concerted demand for the development of non-destructive techniques to acquire an accurate assessment of the condition of reinforced concrete structures. To address this need, the linear polarization resistance (LPR) measurement technique has become a well-established method of determining the instantaneous corrosion rate of reinforcing steel in concrete. The technique is quick and non-intrusive, resulting in only localized damage to the concrete cover so that an electrical connection can be made to the reinforcing steel.

LPR test data yield more detailed information than a simple potential survey. The LPR data provide a means to acquire a more detailed assessment of the structural condition and is a major tool in deciding on the optimum remedial strategy that should be adopted. Thus, it is imperative to obtain accurate LPR measurements. In LPR measurements, the reinforcing steel is perturbed by a small amount from its equilibrium potential, which can be accomplished potentio-statically by changing the potential of the reinforcing steel by a fixed amount, E, and monitoring the current decay, I, after a fixed time. Alternatively, it can be done galvano-statically by applying a small fixed current, I, to the reinforcing steel and monitoring the potential change, E, after a fixed interval. In each case, the conditions are selected so that the changes in potential, E, fall within the linear Stern–Geary range of 10 to 30 millivolts. The polarization resistance, R_p, of the steel is then calculated from the equation:

$$R_p = \Delta E/\Delta I$$

Where:

ΔE = Change in voltage

ΔI = Change in current per unit area

In addition, there is an established relationship between the corrosion rate of the anode and the polarization resistance:

$$I_{corr} = B/R_p$$

Where:

I_{corr} = Corrosion current density in area (ft^2)

B = A constant based on polarization curves (taken as 0.026 volts for steel in concrete)

R_p = Polarization in Ω/ft^2

The advantages of LPR include:

- Ability to use portable and lightweight measuring equipment; and,
- Determines the rate of corrosion at the time of the testing.

Its disadvantages include:

- Requires direct electrical connection to the reinforcement and interconnection of all of the reinforcing elements;
- Cannot test decks with epoxy-coated or galvanized rebar;
- Concrete must be uncracked, even and free of coating and visible moisture;
- Results can vary widely across a deck; and,
- Requires knowledgeable personnel in order to interpret the results.

Magnetic Flux Leakage. Magnetic flux leakage (MFL) is a method of non-destructive testing that is used to detect corrosion and pitting in steel structures. A magnet is mounted on a carriage and induces a strong magnetic field in the area of interest. If corrosion is present, a magnetic flux leakage field forms outside the area and an array of sensors positioned between the magnet's poles detects the flux leakage.

MFL measures changes in the path of magnetic force lines, or flux, near a ferromagnetic material with defects. In the presence of a magnetic field, ferromagnetic materials align their electric dipoles with the external field. A high intensity magnetic field has more aligned dipoles. The applied magnetic field needs to be strong enough to overcome noise, as well as the distance between the magnetic source and the ferromagnetic field. When the flux detects corrosion or fractured steel, it leaks out instead of traveling to the south pole of the magnet. The reduction in cross-sectional area of the material causes the leakage. MFL equipment use "Hall Effect" sensors, which react to and develop a voltage difference between the parallel faces of crystal sensors to detect the leakage.

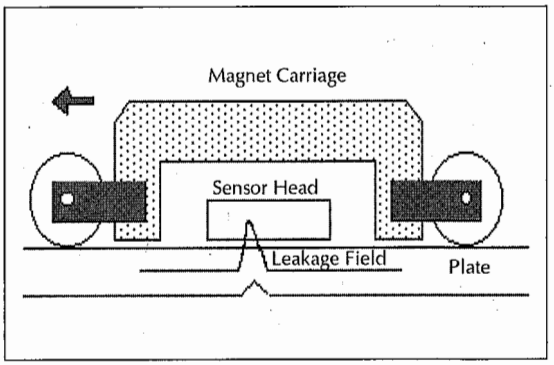

FIGURE 5. Magnetic flux leakage.

This method has certain advantages:

- Mechanical contact with the material or structure under examination is not necessary;
- There is no need to specially prepare the test area (it needs to be cleaned only); and,
- This method detects cracks and subsurface inclusions down to 0.3 millimeters.

Since MFL is a detection technique that detects volumetrical changes, its results report no absolute values but rather relative volumetrical changes. However, it is a very suitable tool for detecting bad spots in plates.

Petrography. Petrography is widely used in order to better diagnose distressed concrete. Usually poor and good quality areas are evaluated. A petrography study provides information about the different properties of materials

used on a site, as well as the effectiveness of the concrete mix.

Identification of aggregates can be made somewhat just from a visual inspection. A large number of minerals can be recognized by observing the general appearance of the concrete in plane polarized light, which splits into two rays when it passes through a certain filter. The light rays are viewed with a polarizing device called a *Nicol prism*. Carbonation and water content, degree of maturity (cement hydration) and bonding within the concrete matrix, discoloration and staining can be determined using this method. This method can also be used to determine segregation caused by excessive vibration, as well as noting if there are any supplemental cementitious materials. A petrographic study can reveal certain conditions of the concrete pavement such as alkali silica reactions (ASR), delayed ettringite formation, freezing of plastic concrete, freeze-thaw distress, aggregate soundness, sulfate attack and contaminants.

The petrography method is also used to determine the air content of cement, parameters of the air void system and the water/cement ratio.

A petrographic microscope, called a polarizing microscope, is used to detect cracks and defects in the concrete. It is a compound, transmitted-light microscope to which components have been added in order to determine the optical properties of translucent substances. The microscope has an ocular that focuses on a virtual image of the subject that is produced in the tube of the microscope by the objective lens.

Corrosion, and the resulting expansion of the reinforcement that causes lateral cracking in the plane in which the reinforcement is situated, can be viewed and determined with this method. Normally, the high pH of concrete protects the reinforcing steel from oxidation. The concrete between the reinforcement and the outer surface of the element thus serves as a barrier to the ingress of chloride ions or carbon dioxide. Construction plans should specify the concrete cover thickness, which is usually around 50 to 68 millimeters. Cracks extending from the exposed surfaces toward the level of the reinforcement may significantly decrease the effectiveness of the cover concrete as a barrier, especially if the cracking is over an extensive area.

An advantage to the petrography method is that it provides insight into existing concrete properties that are related to durability.

The method's disadvantages are that it can take quite a bit of time to conduct a thorough petrographic study and that it requires sample removal.

Corrosion Reduction Options

Patch Option (Coat Rebar). Coating reinforcing bars can help prevent chloride ions from reaching the metal, and, therefore, help protect the reinforcing steel. Epoxy coatings provide protection by acting as a physical barrier that prevents or slows down the arrival of corrodents to the coating/steel interface. Epoxy coating is a barrier system that is designed to keep chlorides and other chemicals that can initiate and sustain corrosion away from the reinforcing steel. It also provides a physical barrier between the steel and the concrete interface.

However, the epoxy does uptake some amount of moisture, which results in a temporary reduction or loss of bond. It is also impacted by the presence of coating damage or defects in the form of holidays (which result from areas that accidentally do not receive coating), mashed areas, narrow cracks and bare areas. The defects in the coating are normally generated during the application of the coating, storage and handling, transportation to site, placement in forms and placement of the concrete. Corrosion on epoxy-coated rebar initiates at defects in the form of crevice corrosion and can spread by undercutting. The rate of corrosion is controlled by the availability of cathodic sites and chloride ions. In addition, the coating may deteriorate over time.

The two national specifications for epoxy-coated reinforcing bars reflect the preceding coating application processes and industry practices. ASTM A775/A775M, which was issued in 1981, establishes the requirements for the epoxy coating of straight bars. The second specification, designated as A934/A934M and adopted in 1995, prescribes the require-

ments for the epoxy coating of pre-fabricated bars. These provisions cover:

- the surface preparation of the bars before the application of the coating;
- recognition of, and permission to use, a pretreatment on the cleaned bars before application of the coating;
- limits on the thickness of the coating;
- limitation of holidays in the coating to an average of one per foot; and,
- prequalification requirements in the form of testing criteria for chemical resistance of the coating, cathodic disbandment, salt spray resistance, chloride permeability, coating flexibility, relative bond strength in concrete, abrasion resistance and impact.

Damaged coating is limited to one percent of the surface area in any one foot length of coated bar. A coated bar with damaged coating not exceeding the one percent limit is acceptable on the condition that all of the damaged coating is properly repaired prior to shipment to the job site.

Durng the late 1980s, in response to concerns about the Florida Keys bridges, a task group within the ASTM Subcommittee on Steel Reinforcement initiated a critical evaluation of the A775/A775M specification. The resulting revisions involved coating thickness, surface preparation, coating continuity, coating flexibility, coating adhesion, permissible amount of damaged coating and the repair of damaged coating. These revisions, adopted in the A775/A775M specification, translated into the fabrication of higher quality epoxy-coated bars. The lower value of the permitted range of coating thickness was raised to 7 mils (0.178 millimeters) from 5 mils (0.127 millimeters). Average readings of 1.5 to 4.0 mils (0.38 to 0.102 millimeters) are required for the maximum roughness depth of the blast profile to assure a suitable anchor pattern. After abrasive blast cleaning, the use of multidirectional high-pressure dry air knives is required to remove dust, grit and other foreign matter from the bar surface. The deposition of oil on the cleaned bars by the air knives was prohibited. A chemical wash or other treatment of the steel surface of the bar can be used to enhance adhesion.

In-line holiday detection is recommended by the A775/A775M specification. The accuracy of the in-line system must be verified by checking with a hand-held detector. The limit on the allowable number of holidays is reduced to an average of one per foot from two per foot. Conducting bend tests of coated bars requires 180-degree bend angles for bars (except for some large bars) instead of the previous bend angle of 120 degrees. In addition, production-coated bars should be evaluated by a cathodic disbandment test.

All coating damage incurred during handling and fabrication must be repaired before shipment to the job site. Damaged coating on a bar must be repaired if the amount of damaged coating exceeds one percent of the total surface area in each one foot length of the bar. Coating the reinforcing bars is relatively inexpensive and appears to provide good corrosion protection for the coated bars in the patches as well as for adjacent bars in the existing concrete.

Patch Option (Discrete Anodes). This method involves the installation of an impressed current cathodic protection system for reinforced concrete structures. The purpose of the cathodic protection system is to protect the steel embedded in the concrete from corroding by passing a small direct current from the anode to the steel. An internal discrete anode system (IDAS) should provide sufficient current to adequately protect the steel according to the current density requirements of the steel and the field-proven ventilation of anodic gases. A minimum anode life expectancy of twenty-five years at the full design anode current density of 900 milliamps per square meter of anode surface area and at least five years of satisfactory operation in one or more similar applications are required. An IDAS should also provide a positive system of locating and positioning the anodes that are being installed in the structure, thus ensuring a high confidence of zero short circuits and uniform current distribution.

The area affected by the discrete anodes and their effectiveness in protecting against

corrosion can be easily disputed. These anodes are also somewhat more expensive than coating. While concrete admixtures are relatively inexpensive and do provide some corrosion protection, their use in conjunction with coated bars may be overkill. Silica fume enhances the concrete properties, but is difficult to place in discrete patches without cracking.

Deck Option (Bonded Overlay). Bonded overlays are most commonly used on deck systems. Low-slump, high-density (LSHD) concrete, latex modified concrete (LMC), silica-fume concrete (SFC) and polymer concrete are materials that can be used to form a bonded overlay on a deck. Due to their smaller degree of permeability than existing concrete, overlays can make the cover over reinforcing bars stronger. In addition, because these concretes increase the weight of the deck, a special calculation at the edges is required. These concretes can be also used in patches. Their permeability and electrical conductivity is less than that of regular concrete. LSHD has 50 percent, SFC and LMC have 10 to 25 percent, and polymer concrete has less than 10 percent of the permeability and electrical conductivity of conventional concrete.

Portland cement overlays are about 2 inches thick and polymer concrete overlays are about 0.25 to 1 inch thick. According to a survey in 2005, many state highway agencies use more than one overlay material.[6] In addition, it is estimated that bonded overlays add twenty to twenty-five years to deck service life.

Deck Option (Membrane). An asphalt topping with a waterproofing membrane is another option that can be used to protect a deck from corrosion. The membrane should be tested for water and chloride permeability, along with tensile strength, durability, toughness, elasticity and temperature susceptibility as they correlate to the degree of adhesion between the membrane and the asphalt. A proper anticorrosive system must ensure a good adhesive force between the anticorrosive coating and the steel bridge deck because it is the strong force of adhesion that adds to decreasing the deterioration in steel. Laboratory tests indicate that water and chloride penetration is virtually eliminated and the effect of reducing corrosion is significant. However, the topping increases the weight of the structure and it is recommended that the deck edge be designed to accommodate the membrane. Using membrane systems can increase the life of the deck for about ten to fifteen years, but the deterioration of asphalt requires frequent replacement, thus limiting the life of the system.

Deck Option (Surface Sealer). Surface sealers include silanes, siloxanes, epoxies, methacrylates and urethanes. The sealant's usefulness in corrosion prevention in concrete pavement depends on the adhesion between sealant's surface and the concrete. Strong adhesion strength is also needed to withstand static and dynamic stresses such as traffic and temperature fluctuations. Applied to the concrete by spraying, silanes and siloxanes penetrate 0.1 to 0.25 inches into it and are widely used on decks exposed to traffic. Epoxies, methacrylates and urethanes form a film sealer and they are applied mostly on substructure elements and on superstructure. These materials are effective in reducing the effect of corrosion. Over 40 percnt of state highway departments use silane, 20 percent use epoxies, 16 percent use linseed oil and another 16 percent do not use sealers.

Corrosion Inhibitors

Many kinds of inhibitors are used to prevent corrosion in reinforced concrete such as:

- calcium nitrite (the traditional commercial inhibitor);
- sodium monofluorophosphate (which is used to prevent the onset of corrosion or to reduce corrosion rates);
- alkanolamines such as diethanolamine, dimethylpropanolamine, monoethanolamine, dimethylethanolamine, methyldiethanolamie and triethanolamine (which are used to reduce steel corrosion rates);
- other organic substances based on ternary mixtures of aldonic acid, benzoic acid and a triazole, carboxylic or bicarboxylic acids, and tannins.

Corrosion inhibitors have been proven effective in steel pipelines, tanks, etc., for

many decades. However, their use as admixtures to concrete is more recent and more limited. Their use in concrete is limited because they cannot be changed or replenished if they are found to be ineffective or have deleterious effects. Therefore, it is essential that the mechanism of inhibition be studied to ensure proper use.

There are numerous compounds that can be investigated in the laboratory as potential corrosion-inhibiting admixtures to concrete. Sodium compounds are found to reduce the compressive strength of concrete, whereas calcium compounds do not. As a matter of fact, calcium compounds are found to increase the compressive strength of concrete. Therefore, calcium salts are more commonly used as anodic inhibitors. Some of the organic compounds affect the air void materials and, as a result, the dosage of void-preventive materials must be modified accordingly. Similarly, when used as a corrosion inhibitor, how the concrete set accelerating properties of calcium nitrite and concrete-set-retarding effects of some of the organics must be understood.

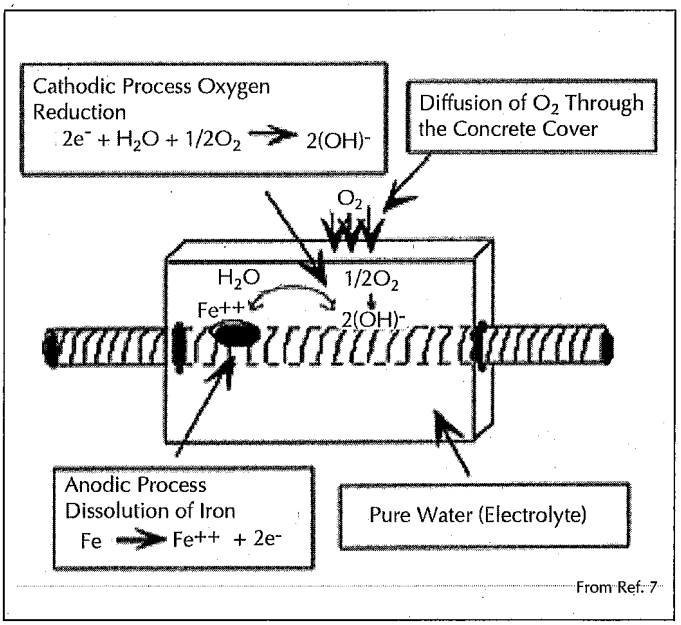

FIGURE 6. Corrosion inhibitors in concrete.

An ideal corrosion inhibitor is defined as "a chemical compound, which, when added in adequate amounts to concrete, can prevent corrosion of embedded steel and has no adverse effect on the properties of concrete."[7] As a result, only the admixed corrosion inhibitors are considered, although migrating chemicals and concrete coatings could have similar effects. When considering the chloride-induced corrosion of steel reinforcement in concrete, the factors that could be affected by an inhibitor are:

- the rate of ingress of chlorides from the environment;
- the degree to which these chlorides are chemically bound or physically trapped in the concrete cover;
- the concentration of chlorides that the steel can tolerate without the breakdown of the inherent passive film;
- the rate of ingress of dissolved oxygen to sustain the cathodic half-cell reaction;
- the electrical resistance of the concrete; and,
- the chemical composition of the electrolyte.

Because of their set accelerating properties, inhibitors tend to increase pore size distribution in concrete. On the other hand, it may be possible that chlorides that have penetrated the concrete from the outside may decrease porosity because of the precipitation of chloro-aluminates in the coarser pores, although that has not been confirmed. Chlorides also can affect the pH value of the pore solution: $CaCl_2$ decreases the pH; whereas NaCl and KCl have the opposite effect. Because of their hygroscopic character, the chlorides tend to cause an increase in water absorption in concrete. However, the relative humidity of dilute salt solutions is usually higher than that of highly concentrated solutions and, thus, the relative humidity of non-saturated concrete containing chlorides may be lower than those with similar levels of water saturation but without chlorides.

Finally, the increased ionic content of the solution due to chlorides will increase its con-

ductivity and, consequently, decrease the electrical resistivity of the concrete.

Therefore, it is far from sufficient to consider only how the inhibitor affects concrete properties. How the chlorides affect the properties of concrete containing inhibitors also must be considered.

In summary, a corrosion-inhibiting admixture can work by:

- increasing the resistance of the steel to breakdown by chlorides;
- generating a barrier film on the steel;
- blocking the infiltration of chlorides;
- blocking the infiltration of oxygen; and,
- absorbing the oxygen dissolved in the pore solution.

Cathodic Protection

The cathodic protection method is an electrochemical technique that can minimize the corrosion of rebar in reinforced concrete. Cathodic protection works by making the metal act as the cathode of an electrochemical cell by placing it in contact with a more easily corroded metal that acts as the anode. In this rehabilitation method, the rebar at risk of corrosion are uniformly electrically polarized with –200 millivolts in order to create ideal protection to prevent the corrosion of unpolarized areas. Designing, maintaining and managing a cathodic protection system for concrete structures has received very little attention in the eighty-year history of the cathodic protection technique but now it is a popular method and it is used in a variety of structures that are subjected to corrosion.

Most commonly, metal (rebar) that comes into contact with concrete is at risk for corrosion and its surface can sustain dissolution and a cathodic conjugate reaction that, in some cases, leads to oxygen reduction or hydrogen evolution. When a metal is polarized positively, its dissolution will be accelerated but the cathodic conjugate reaction will be decelerated and if it is polarized negatively its dissolution will be decelerated but the cathodic conjugate reaction will be accelerated. A polarization between –200 and –300 millivolts is sufficient for cathodic protection. For the successful application of cathodic protection, a uniform distribution over the entire metal (rebar) is required.

The reasons why cathodic protection has not been adopted in more than 350 bridges in North America can be attributed to the uniformity of distribution of the cathodic protection current, the design of the ground bed versus rebar geometry and the locations of electrical contacts, as well as due to the effects of the environment and climate on the distribution of the cathodic protection current.

The cathodic protection of steel in concrete was developed initially in Europe and the United States for buried prestressed concrete water pipelines. In California and other sites in North America, it was used to protect reinforced concrete bridge decks from deicing salt. In the United Kingdom, the method was developed to tackle such different problems as buildings with cast-in chlorides to bridge substructures contaminated with deicing salts and to marine structures and tunnels. In the Middle East, there are lots of problems of corrosion caused by high levels of salinity in soils (such as in marine conditions). In Australia, Japan and Hong Kong, cathodic protection is used extensively. In 1959, the California Department of Transportation, installed an experimental cathodic protection system on a bridge-supported beam. In 1972, a more advanced anode system was installed on the bridge deck that was based on a conventional impressed current cathodic system for pipelines. One problem in adopting this method for U.S. bridge decks is that U.S. decks are not designed for overlays. The anode should not change the profile of the bridge nor should it increase the deadload. The most successful anodes for use in decks are now titanum mixed with oxide coating.

This method is used for long-term protection on corrosion-damaged structures that have significant residual life or on undamaged structures in high chloride environments. In the latter application, there would be a considerable cost saving compared to long-term repair, reconstruction or replacement. This method is useful for highway bridge/tunnel engineers, marine and harbor authorities, building engineers/architects/managers and other professionals and entities who have to

deal with corroding reinforcement. Projects that combine concrete repair and cathodic protection require multidiscipline skills from the civil/structural/concrete engineering professionals involved.

Chloride Extraction

Chloride extraction was developed to treat corrosion damage of reinforced concrete structures by removing the chloride ions from the contamined concrete. Chloride extraction is performed by applying an electric field between the reinforcing steel in the concrete (the cathode) and an externally mounted electrode mesh (the anode). The electrode mesh is embedded in a sprayed-on mixture of potable water and cellulose fiber. During this treatment, the negatively charged chloride ions near the rebar migrate toward the positively charged anode mesh. When the chloride concentration in the concrete has been reduced to an acceptable level (which also means that the pH level of the concrete has increased), the temporary electrolyte media is removed from the structure. This process usually takes between four to eight weeks.

Chloride extraction is a non-destructive method and no concrete removal is necessary. Any noise and dust problems associated with concrete removal or disturbance are almost eliminated, and the process uses enviromentally safe materials. After treatment, some chloride will remain in the concrete: this method does not remove 100 percent of the chloride from the concrete structure; however, most codes accept 0.2 to 0.4 percent chlorides by weight of cement in new, ordinary reinforced concrete. The concrete's ability to protect steel reinforcing from corrosion changes with its pH level; a treated structure may offer added protection against renewed corrosion attack due to the increased pH level, prolonging the useful life of the structure.

It should be noted that using chloride extraction in prestressed concrete structures is not recommended due to uncertainties related to hydrogen production. It may cause potential embrittlement and stress corrosion cracking.

Chloride extraction can be performed under all weather conditions as long as the electrolyte does not freeze. The necessary number of rebar connections depends on the electrical continuity of the rebar, but there should be at least one every 50 square meters as a general requirement. Based on the results of conducting an electrical continuity survey on the proposed structure, the typical connection between two rebar connection points should be ideally less than 1 ohm, but up to 50 ohms may be acceptable. Flexible copper cables are connected to the rebars using self-tapping screws and cable connectors (other suitable methods can be used for connection).

After satisfactory treatment, all equipment is removed and the surface is cleaned. To prevent the possibility of renewed chloride contamination and to promote long-term effectiveness, the surface should be treated with waterproof protective coating, such as sealer or deck coating. It would be benefical to use embedded electrodes to monitor future corrosion activity on the structure.

Brief Summary of Corrosion Reduction Options

All of the corrosion reduction methods could be applicable, depending on the conditions and circumstances of the particular project. Each method presents its own costs and average service life (see Table 1).

Patch Option. Coating the reinforcing bars is relatively inexpensive and appears to provide good corrorion protection. Laboratory tests have shown that even grossly flawed coating provides significant corrosion protection, regardless of whether the coating was epoxy, cementitious or modified cementitious.[8] Two coats provide better protection than one coat due to the fact that the second coat addresses some of the flaws in the first coat. These laboratory tests indicate that applying two coats of a corrosion-inhibiting materials to reinfocing steel in patches would yield best results.[8] Field-coating reinforcing bars may add about 5 to 10 percent to the total patch cost.

Deck Option. Silane surface sealers can reduce water and chloride penetration into deck concrete by 85 to 90 percent, while expoxies and other surface coatings can reduce water and chloride penetration by up to 95 percent. Penetrating sealers on decks

TABLE 1.
Price & Service Life Comparison

Corrosion Reduction Method	Price (Square Foot)	Service Life (Years)
Coating Rebar	$1 to $2	4 to 10
Bonded Overlay	$5 to $25	20 to 25
Membrane	$3 to $5	10 to 15
Surface Sealer	$3 to $4	10 to 20
Cathodic Protection	$10 to $15	15 to 30

may need to applied every five to seven years. Epoxy, methacrylate and urethane sealers on superstructure and substructure elements may need to be reapplied every ten to eighteen years.

Corrosion Inhibitors. Migrating corrosion inhibitors are relatively inexpensive for deck treatment, but the degree of penetration into the concrete is disputed. If the inhibitor does not reach the steel, its effectiveness is questionable.

Cathodic Protection. Cathodic protection stops corrosion activity effectively, but it is very expensive to implement. In 2004, over half of responding state highway agencies had tried cathodic protection, and many reported difficulties with reliability and maintenance.[6] Hovewer, this method could be considered to save costs in comparison to the long-term costs of repair. In addition, applying this method requires expertise from professionals experienced with dealing with corrosion cathodic protection materials.

Chloride Extraction. The effectiveness of chloride extraction is influenced by the quality of the concrete and the depth of the reinforcing steel. The goal of the method is to reduce chloride levels at the reinforcing steel. Chloride extraction commonly requires three to eight weeks perform. The process of the chloride extration is estimated to add ten to twenty years to the member service life.

Alternatives to Steel Reinforcement

Alternatives to steel reinforcement, such as fiber polymer materials, have gained significant attention in the structural engineering field. Fiber reinforced polymer (FRP) bars are made of polymer fibers pulled longitudinally and then held together by a resin. The advantages to their use include: high tensile strength, light weight (which makes them convenient for transportation and installation), resistance to corrosion and effective cost. These benefits make FRP bars a promising solution for the replacement of steel reinforcement.

Glass (GFRP), carbon (CFRP), aramid (AFRP) and basalt (BFRP) fiber polymers have been used in experiments that have been devised to determine how these materials can improve structural stability. (See Table 2 for a summary of properties of these types of fibers.) Loss of strength over time must be taken into consideration in order to quantify the durability of these materials. Degradation of GFRP strength due to moisture and alkali reactions was studied by many researchers and many methods for accelerated aging tests were performed.[9-11] ACI recommends that the strength of bars at the end of service be 70 percent of the initial tensile strength.[12]

Glass fibers are divided in three categories:

- E-glass — used for electrical;
- S-glass — used for high strength; and,
- C-glass for high corrosion resistance applications.

The most commonly glass fiber used in civil structures is E-glass, which is produced from

TABLE 2.
Summary of Properties of FRPs

Typical Properties	Glass Fibers		Aramid Fibers		Carbon Fibers		
	E-Glass	S-Glass	Kevlar 29	Kevlar 49	High Strength	High Modulus	Ultra-High Modulus
Density (g/cm³)	2.6	2.5	1.44	1.44	1.8	1.9	2.0-2.1
Young' Modulus (Gpa)	72	87	83/100	124	230	370	520-620
Tensile Strength (Gpa)	1.72	2.53	2.27	2.27	2.48	1.79	1.03-1.31
Tensile Elongation (%)	2.4	2.9	2.8	1.8	1.1	0.5	0.2

lime-alumina-borosilicate, which is usually obtained from sand.

Aramid fibers are synthetic organic fibers that provide good fatigue and creep resistance. The most common aramid fibers used in structural applications are Kevlar 29 and Kevlar 49.

Carbon fibers are anisotropic materials and have lower thermal expansion coefficients than glass and aramid fibers. They are brittle materials, which makes them critical in joints and connections. As a result, an adhesive bonding should be used with them.

The Gills Creek Bridge in Virginia was constructed to model the behavior of polymer reinforcement. Extensive research was conducted to determine and compare reinforcing materials. The top mat of the deck of one end-span of the Gills Creek Bridge was reinforced with GFRP bars and the bottom mat was reinforced with steel bars. Epoxy coating was applied on the steel reinforcement. The opposite span of the deck was constructed of epoxy-coated steel bars on the top and bottom mats and live load tests were performed on both spans in 2003 (after construction) and again in 2004.[12]

Stresses calculated from tests showed that transverse bars experienced tensile stresses of 75 pounds per square inch, which is below the ACI 440 specified tensile stress of 13.9 kilo-pound-force per square inch and less than the laboratory-measured tensile strength of 109 kilo-pound-force per square inch. Allowable compressive stress is not specified by ACI 440 because rupture from creep is not governing in compression. The compressive stresses of 130 pounds per square inch recorded in this case were small and it was assumed that these stresses could be carried by the concrete.

GFRP offers a low modulus of elasticity, around 6,000 kilo-pound-force per square inch (compared to 29,000 kilo-pound-force per square inch for steel), which could cause deflections and cracks and result in lower shear strength. Tests show that the stress-strain relationship is linear elastic until rupture,[12] which the ACI Committee 440 (2003) guidelines take into account in designing with GFRP-reinforced concrete.[13] The governing factor in the design of GFRP-reinforced concrete tends to limit the cracks by placing sufficient reinforcement and decreasing surface cracks. There needs to be more research on its long-term durability.

Composite materials technology can benefit the building and development of civil engineering structures in terms of offering engineers a wider array of physical properties and technical solutions. In choosing these materials and solutions, their implementation and service costs also must be taken into consideration for their applicability.

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The Gift

The true gift is not just in the giving, but also not looking the gift horse in mouth and finding another splendid way to repurpose an item.

BRIAN BRENNER

Dan was free at last. After eighteen years of living under the watchful gaze of hovering, helicoptering parents, he was a freshman at college. His parents said their tearful good-byes and took off for the long, solitary return trip to New England. Dan watched the station wagon's exhaust fade away. All around him, students unpacked clothes, televisions, computers and other essential bric-a-brac. Classes hadn't started yet so he had a few days to experience college fun with no responsibilities. He was free, free, free! He relaxed in his new dorm room with his new roommate, a kid whose last name ended in "y." Out in the hall there were girls. Someone was tossing a football. The dorm room was neat and spotless, at least for now. The two moms had organized everything, with the clothes carefully folded and each item in its proper place. The floor was not yet piled with dirty laundry or half-eaten food. Dan and his new roommate smiled. That would change.

As he was checking out the new lay of the land (his land), Dan looked in his sock drawer. A gift-wrapped box was hidden below the socks.

The kid whose last name ended in "y" said, "Looks like you got a present."

"It's cookies," Dan said. "My parents baked cookies." He said this in a neutral, play-my-cards-close-to-the-vest tone of voice. Probably he would share the cookies, but Dan was really hungry and he wasn't completely sure that he would share them.

The box had an envelope with a card. Normally, Dan would go straight for the cookies and ignore the card, but that didn't seem appropriate in this case. Even though his folks were gone only an hour, this time they were really gone, so he decided that he should read the card first. That way, it would be more respectful of the gift. The card said:

Dear Dan,

We love you. We are so proud that you're starting your college year. Best of luck. We are looking forward to being grandparents. Just not yet.

Love, Mom and Dad

Dan reread the last line several times and then opened the box. It wasn't cookies. There was an assortment. Since mom and dad weren't sure exactly what type to get, they got everything.

The kid whose last name ended in "y" asked, "What kind of cookies did they make?"

Dan said, in a much different tone of voice, "It's not cookies."

The Point of the Story

It's probably not obvious, but this story is related to the performance of triaxial tests. These loading tests are performed by geotechnical engineers on soil samples. The samples are cut to be cylindrical in shape, about eight inches long or so, and they are placed in a testing device and exposed to different types of stress. The objective of these tests is to measure how the soil reacts under different loading conditions. By doing so, geotechnical engineers can determine soil properties and overall behavior, as well as model foundation and substrata performance during construction.

In order to run the test, the soil sample is placed a chamber filled with water. Clearly, the soil sample must be contained, or the water will mix with the soil and the sample will turn to mud. The most common method to contain the soil is to place it in an impervious membrane. It is possible to purchase membranes specially designed for triaxial tests, but most geotechnical engineers don't bother. Suitable membranes are conveniently available at the local drug store, in shapes and strengths satisfactory for the tests. To get proper results, however, only certain types of these commonly available membranes may be used, and they must be applied using the correct procedures. For some reason, little documentation is available on the proper way to use these membranes for triaxial tests. The key information has been largely transmitted by geotechnical engineers via word of mouth, often in a hushed tone of voice.

Keeping It Simple

Should you need to run a triaxial test of, say, clay, note when selecting a membrane that you really want to use the basic product. You don't want or need frills. In the old days, circa *Summer of '42*, membranes were available in pretty much one shape and type, but today, membranes come with an explosion of options. For geotechnical purposes, you do not want to select the membranes with projections. Ruffles and ridges will interfere with proper performance during triaxial tests. Membranes of different colors may be aesthetically attractive, but the color is not really relevant for the success of your test. However, size does matter when it comes to membranes. A very large sheath may be too large for the soil sample and may fail to properly contain the material. Also, specially coated membranes are not appropriate for triaxial testing and should be avoided. The various coatings can interfere with the results.

It's helpful to have a large supply available, so when purchasing membranes, buy in bulk. Let the sales attendant know that you are a geotechnical engineer performing triaxial tests. All weekend.

Relying on Experience

Back in the lab, when it's time to fit the membranes onto the soil samples, most geotechnical engineers report that experience is key. After much trial and error, novice triaxial testers eventually get the hang of it. It is important to use two membranes instead of one, because keeping water out of the sample is essential. Apparently, membranes often have small nicks and cuts that can lead to water infiltration.

Geotech professors comment (not in writing, of course) that triaxial test instruction has a unique component, with an aspect not present in many other civil engineering topics. In the old days (again, circa *Summer of '42*), virtually all civil engineering students were male, which was simultaneously easier and harder for triaxial test lessons. Nowadays, many more women are studying to become civil engineers. The best teaching method, therefore, is a studiously clinical approach, with an irony-free, straight-lipped delivery.

A Good Course of Action

When Dan's freshmen year ended, his parents piled into the station wagon and drove back down to Maryland. It was the drop-off process only in reverse. All of the students were deconstructing their dorm rooms and dragging the voluminous bric-a-brac to the street.

Fortunately, we borrowed a rooftop carrier to fit all the extra stuff Dan had accumulated. This was where I stuffed Dan's dirty laundry — all four weeks of it. Dan had timed his last wash so that his clean clothes, or some semblance of them, would run out just as we arrived.

Dan had a great year and preparing to leave college was bittersweet. It took many trips from his dorm room to the street and many hours to empty out all his stuff. The loaded station wagon looked like the vehicle at the beginning of the *Beverly Hillbillies*, except that Granny wasn't riding in the back. If it mattered, I suppose that none of the college kids knew who the Beverly Hillbillies were anyway. The dorm room that had been home to the two boys during their first-year adventure was now forlorn and nearly bare. In one corner, with piles of refuse waiting to be tossed, was the empty gift box.

The empty box reminded me of engineering school and my days in the geotech lab. I said to Dan, "I remember Intro to Geotech. That was a great class."

Dan said, "Yes, Dad, I learned a lot in Intro to Geotech. I'm glad I took that course. I enjoyed it."

The kid whose last name ended in "y" was puzzled. He said, "Mr. Brenner, Dan didn't sign up for Intro to Geotechnical Engineering."

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Article Index 1986–2010

The following index covers the Spring 1986 to the Spring/Summer 2010 issues and is grouped by discipline. Articles are alphabetized by title within each grouping. The scope of many articles included here fall across more than one discipline. Articles are not cross-referenced or included in more than one grouping, so it is recommended to scan related groupings when seeking a particular article. Each entry gives the article title (in italics), author(s), issue, and page citation.

Our Infrastructure Is a National Treasure!, James Lambrechts, Spring/Summer 2010, pp. 5-6

Partnering & Its Implementation on the Central Artery/Tunnel Project, Michelle G. Daigle & Ali Touran, Spring/Summer 1998, pp. 49-62

Practice Can Make Perfect, Brian Brenner, Fall/Winter 1993, p. 5

The Practice... of Civil Engineering, James Lambrechts, Fall/Winter 2008, pp. 5-6

The Proactive Engineer: A Vision of Leadership, Eugene J. Fasullo, Spring/Summer 1998, pp. 83-88

Proactive Engineering: A Perspective, Jack K. Lemley, Fall/Winter 1995, pp. 37-42

Providence, Brian Brenner, Fall/Winter 2001, pp. 65-66

Recognizing Engineering Excellence, compiled by Brian Brenner, Fall/Winter 2007, pp. 61-79

Replacement of the Cranston Viaducts Using Spliced Bulb-Tee Girder Technology, Firooz Panah & H. Raymond Palmer, Fall/Winter 2005, pp. 35-50

Risk Modeling & Measurement in Construction, Ali Touran, Spring 1992, pp. 29-46

The Role of Engineers in Creating an Environmentally Sustainable Future, Anthony D. Cortese, Spring/Summer 1999, pp. 29-38

Spectacle Island, Brian Brenner, Spring/Summer 2003, pp. 63-65

Streamlining the Public Works Design Process, Cynthia Chabot, Fall/Winter 1996, pp. 5-6

A Taxpayer's Look at a Sacred Cow: Public Sector Design in Massachusetts Two Decades After the Ward Commission, Edward Moscovitch, Fall/Winter 1996, pp. 7-12

Tewksbury Collection System Expansion: The Development & Implementation of a Major Infrastructure Program, Jerome Selissen & Michael J. Walsh, Spring/Summer 2009, pp. 65-72

Transforming the Engineer into a Manager: Avoiding the Peter Principle, Neal E. Thornberry, Fall 1989, pp. 69-74

20/20 Vision: The Engineering & Construction Industry in the 21st Century, Henry L. Michel, Spring/Summer 2001, pp. 75-79

Understanding the Nature of Engineering Decision-Making, P. Aarne Vesilind, Spring 1987, pp. 7-16

A View From the Academy: Preparing Future Civil Engineers for Practice, Thomas C. Sheahan, Fall/Winter 2004, pp. 65-67

What Happened to Nantucket?, Brian Brenner, Fall/Winter 2002, pp. 66-67

Whatever Happened to John T. Mongan?, Brian Brenner, Fall/Winter 2007, pp. 80-83

When I Was a Contestant on Jeopardy, Brian Brenner, Spring/Summer 2009, pp. 73-75

ENVIRONMENTAL

Applying Continuous-Flow Stirred Tank Reactor Methodology to Mussel Biomonitoring & Effluent Discharge Data, Windsor Sung, Spring/Summer 1999, pp. 63-74

Boston Harbor Cleanup: Use or Abuse of Regulatory Authority?, Donald R.F. Harleman, Spring 1989, pp. 25-32

The Case for Using Chemically Enhanced Primary Treatment in a New Cleanup Plan for Boston Harbor, Donald R.F. Harleman, Shawn Morrissey & Susan Murcott, Spring 1991, pp. 69-84

Chemically Enhanced Wastewater Treatment: An Alternative & Complement to Biological Wastewater Treatment, Ingemar Karlsson & Shawn P. Morrissey, Fall/Winter 1994, pp. 29-38.

Chlorine Dosing at the Ware Disinfection Facility, Windsor Sung, Cynthia Parks, Elizabeth Reilley-Matthews & David Pinksy, Fall/Winter 2001, pp. 51-60

Combined Sewer Overflow Abatement in Boston Harbor, David R. Bingham, Cheryl Breen, Lisa Marx & Michael Collins, Spring/Summer 1994, pp. 83-106

Design of the Deer Island Treatment Plant, John A. Lager, David P. Bova, Robert M. Otoski & Gerald L. Gallinaro, Spring/Summer 1994, pp. 49-66

Drinking Water Quality & Point-of-Use Treatment Studies in Nepal, Andy Bittner, Amer M.A. Khayyat, Kim Luu, Benoit Maag, Susan E. Murcott, Patricia M. Pinto, Junko Sagara & Andrea Wolfe, Spring/Summer 2002, pp. 5-24

The Effectiveness of Municipal Wastewater Treatment, Holly June Stiefel, Fall/Winter 1994, pp. 49-72

Electron Inactivation of Pathogens in Sewage Sludge & Compost: A Comparative Analysis, Samuel R. Maloof, Fall 1988, pp. 37-46

Emerging Biological Treatment Methods: Aerobic and Anaerobic, Ross E. McKinney, Spring 1986, pp. 79-99

Environmental Concerns Imposed by Boston Area Geology, David Woodhouse, Spring 1989, pp. 83-88

An Evaluation of Recycled Tire Shreds as a Substitute for Gravel in Residential Soil Absorption Systems, Sukalyan Sengupta & Heather Miller, Spring/Summer 2004, pp. 33-52

The Feasibility of Real Time Control of Combined Sewer Overflows, Wolfgang Schilling, Fall 1992, pp. 17-26

The History of Leather Industry Waste Contamination in the Aberjona Watershed: A Mass Balance Approach, John L. Durant, Jennifer J. Zemach & Harold F. Hemond, Fall 1990, pp. 41-66

Innovative Wastewater Treatment in the Developing World, Michael R. Bourke, Donald Harleman, Heidi Li, Susan E. Murcott, Gautam Narasimhan & Irene W. Yu, Spring/Summer 2002, pp. 35-34

Investigation and Hydraulic Containment of Chemical Migration: Four Landfills in Niagara Falls, Robert M. Cohen, Richard R. Rabold, Charles R. Faust, James O. Rumbaugh, III, & Jonathan R. Bridge, Spring 1987, pp. 33-58

Landfill Gas: An Asset & a Liability, Michael J. Rossini, Fall/Winter 2001, pp. 41-50

Management & Control of Diffuse Urban Snowmelt Pollution, Vladimir Novotny, Daniel W. Smith & David A. Kuemmel, Fall/Winter 2003, pp. 17-32

Managing the Coastal Plain Aquifers of the Delaware River Basin, David C. Noonan, Spring 1986, pp. 9-22

The Mixing Zone for Combined Sewer Overflows: Testing the Concept as a Basis for Regulation, Thomas Hruby, Fall 1991, pp. 43-54

The New Boston Outfall, Dominique N. Brocard, Brian J. Van Wheele & Lawrence Williamson, Spring/Summer 1994, pp. 33-48

Observations on the Temporal Variations of Dissolved Copper & Zinc in Boston Harbor, Windsor Sung, Spring 1991, pp. 99-110

A Perspective: The Boston Harbor Project, Douglas B. MacDonald, Spring/Summer 1994, pp. 7-9

Planned Facilities for Combined Sewer Overflows: Boston Metropolitan Area, Gene Suhr, Fall 1992, pp. 5-16

Point Toxics Control for Industrial Wastewaters, W. Wesley Eckenfelder, Spring 1988, pp. 98-112

Recycled Paper: A Sound Choice?, Richard Scranton, Spring 1991, p. 4

The Restoration & Treatment of Burlington's Groundwater Supply, Paul C. Millett, Fall/Winter 2000, pp. 83-95

The Scope of the Boston Harbor Project, Dominique N. Brocard, Spring/Summer 1994, pp. 5-6

A Simple Box Model of the Nitrogen Cycle in Boston Harbor and the Massachusetts Bays, E. Eric Adams, Jim W. Hansen, Rafael L. Lago, Pam Clayton & Xueyong Zhang, Fall 1992, pp. 91-103

Simplified Solids-Flux Analysis for the Design of Activated Sludge Wastewater Treatment Systems, Albert B. Pincince, Fall/Winter 1995, pp. 77-90

Smart Growth Strategies for New England, Cynthia Chabot & Brian Brenner, Fall/Winter 2000, pp. 79-82

Sustainable Development Indicators of Some European & Asian River Basins, Susan E. Murcott, Spring/Summer 1999, pp. 57-62

GEOTECHNICAL

The Analysis and Design of the Superconducting Super Collider Underground Structures, Gordon T. Clark & Birger Schmidt, Spring/Summer 1995, pp. 17-36

Anatomy of a Court Trial on Tank Settlements, Charles C. Ladd, Fall/Winter 2004, pp. 45-64

Applying the Finite Element Method to Practical Use in Geotechnical Engineering, J. Michael Duncan, Fall/Winter 1999, pp. 75-80

Back Bay Boston, Part II: Groundwater Levels, Harl P. Aldrich & James R. Lambrechts, Fall 1986, pp. 31-64

Close-In Construction Blasting: Impacts & Mitigation Measures, Andrew F. McKown, Fall 1991, pp. 73-92

Construction of Underground Facilities for the Narragansett Bay Combined Sewer Overflow Program, Phase I, John Kaplin & Geoffrey Hughes, Fall/Winter 2008, pp. 7-32

Deep Foundations Integrity Testing: Techniques & Case Histories, Les R. Chernauskas & Samuel G. Paikowsky, Spring/Summer 1999, pp. 39-56

Deep Well Dewatering for the Greater Cairo Wastewater Project, Robin B. Dill & Mark M. Petersen, Spring/Summer 1993, pp. 13-28

Design & Construction of Deep Stone Columns in Marine Clay at Spectacle Island, Eric M. Klein & Richard F. Tobin, Spring/Summer 1996, pp. 79-94

Developments in Foundation Renovation: Les Promenades de la Cathédrale Project, John Marcovecchio, Spring 1990, pp. 85-97

Developments in Geotechnical Construction Processes for Urban Engineering, Donald A. Bruce, Spring 1988, pp. 49-97

Dilatometer & Cone Penetration Tests on Peat Soil in Carver, Massachusetts, Assem Elsayed, Fall/Winter 2006, pp. 39–52

Dilatometer & Cone Penetration Tests on Peat Soil in Carver, Massachusetts [Supplement], Assem Elsayed, Spring/Summer 2007, pg. 76

Effective Uses of Finite Element Analysis in Geotechnical Engineering, W. Allen Marr, Fall/Winter 1999, pp. 89-98

Evaluation of Liquefaction Potential at a Silt Site in Providence, Rhode Island, A.S. Bradshaw, R.A. Green & C.D.P. Baxter, Spring/Summer 2007, pp. 5-18

Finite Element Analysis of the Combined Effects for Adjoining Braced Excavations, Bashar Altabba & Andrew J. Whittle, Spring/Summer 2003, pp. 5-24

Foundation Considerations for the Expansion & Renovation of the Hynes Auditorium, Edmund G. Johnson & David A. Schoenwolf, Fall 1987, pp. 35-62

From Casagrande's "Calculated Risk" to Reliability-Based Design in Foundation Engineering, Fred H. Kulhawy, Fall/Winter 1996, pp. 43-56

Full-Scale Tiedown Tests for the Central Artery/Tunnel Project, Marco Boscardin, Geraldo R. Iglesia & Mary-Louise Bode, Spring/Summer 1996, pp. 51-78

Geology of the Boston Basin & Vicinity, Patrick J. Barosh, Clifford A. Kaye & David Woodhouse, Spring 1989, pp. 39-52

A Geotechnical Analysis of the Behavior of the Vaiont Slide, A.J. Hendron & F.D. Patton, Fall 1986, pp. 65-130

Geotechnical Characteristics of the Boston Area, Edmund G. Johnson, Spring 1989, pp. 53-64

Geotechnical Design & Construction From 1848 to 1998, S. Trent Parkhill, Fall/Winter 1998, pp. 7-30

Geotechnical Instrumentation for the Central Artery/Tunnel Project: An Overview, John Dunnicliff, Charles Daugherty & Thom Neff, Spring/Summer 1996, pp. 11-20

Geotechnical Instrumentation for Deep Excavations in Boston, Chris M. Erikson, Steven R. Kraemer & Edmund G. Johnson, Spring 1992, pp. 47-66

The Hazard From Earthquakes in the Boston Area, Patrick J. Barosh, Spring 1989, pp. 65-78 [Discussion by William Weiler, Fall 1989, pp. 82-84. Response by author, Fall 1989, pp. pp. 87-89.]

Heathrow Express Cofferdam: Innovation & Delivery Through the Single-Team Approach — Part 1: Design & Construction, Alan J. Powderham, Spring/Summer 2003, pp. 25-40

Immersed Tube Tunnels: Concept, Design & Construction, Thomas R. Kuesel, Spring 1986, pp. 57-78

Innovative Design for Tunnel Exchange & Excavation Support for the CA/T I-90/I-93 Interchange, James R. Lambrechts, Paul A. Roy & Stephen Taylor, Fall/Winter 1999, pp. 43-62

In-Situ Testing for Site Characterization & QA/QC for Deep Dynamic Compaction, Heather J. Miller, Edward L. Hajduk, Kevin P. Stetson, Jean Benoit & Peter J. Conners, Fall/Winter 2007, pp. 19-36

Measures to Minimize the Effects of a Deep Excavation on Two Adjacent Office Buildings: The Abutters' Perspective, Lewis Edgers, Richard Henige, Thomas L. Weinmann & Kenneth B. Wiesner, Spring/Summer 2001, pp. 53-66

Microtremor Measurements to Obtain Resonant Frequencies & Ground Shaking Amplification for Soil Sites in Boston, Kristin E. Hayles, John E. Ebel & Alfredo Urzua, Fall/Winter 2001, pp. 17-36

Modeling the Effects of Soil-Structure Interaction on a Tall Building Bearing on a Mat Foundation, Lewis Edgers, Masoud Sanayei & Joseph L. Alonge, Fall/Winter 2005, pp. 51-68

Observational Evidence for Amplification of Earthquake Ground Motions in Boston & Vicinity, John E. Ebel & Kathleen A. Hart, Fall/Winter 2001, pp. 5-16

The Observational Method — Application Through Progressive Modification, A.J. Powderham, Fall/Winter 1998, pp. 87-110

The Performance of a Remotely Controlled Fiber Glass Pipe Jacking System, Dipak D. Shah, Sajjan K. Jain & Robert W. Prybella, Jr., Spring 1992, pp. 7-28

Pioneers in Soil Mechanics: The Harvard/MIT Heritage, Anni H. Autio & Michael A. McCaffrey, Fall/Winter 2002, pp. 35-48

Pipe Jacking Forces in Soft Ground Construction During Utility Installation Related to Central Artery/Tunnel Project Construction, John M. Pecora, III, & Thomas C. Sheahan, Fall/Winter 2004, pp. 29-44

The Place of Stability Calculations in Evaluating the Safety of Existing Embankment Dams, Ralph B. Peck, Fall 1988, pp. 67-80

The Planning & Implementation of Trenchless Technologies to Restore the St. James Avenue, Boston, Interceptor, Fall/Winter 1998, pp. 77-86

Prediction of Excavation Performance in Clays, Andrew J. Whittle, Fall/Winter 1997, pp. 65-88

Predictions & Observations of Groundwater Conditions During a Deep Well Excavation in Boston, Chris M. Erikson & David A. Schoenwolf, Fall/Winter 1993, pp. 37-52

Pumping Test Program for the Central Artery/Tunnel Project in Downtown Boston, Abdelmadjid M. Lahlaf, Francis D. Leathers & Iqbal Ahmed, Fall/Winter 2000, pp. 63-78

Reducing Seismic Risk in Massachusetts, Steven P. McElligott, James R. Gagnon & Christopher H. Conley, Spring/Summer 1993, pp. 73-90

The Role of Finite Element Methods in Geotechnical Engineering, Andrew J. Whittle, Fall/Winter 1999, pp. 81-88

The Role of Soil-Structure Interaction for Geotechnical & Structural Engineers, Lymon C. Reese, Fall/Winter 2005, pp. 5-34

Seismic Isolation: An Economic Alternative for the Seismic Design & Rehabilitation of Buildings & Bridges, Ronald L. Mayes, Trevor E. Kelly & Lindsay R. Jones, Spring 1990, pp. 7-30

Seismic Response Analysis of Cobble Mountain Reservoir Dam, Alfredo Urzua, John T. Christian, William H. Hover, Ivan A. Hee & Stanley Bemben, Fall/Winter 2002, pp. 7-24

A 70-Foot-Deep Mixed-Face Excavation, Richard M. Simon, Robert J. Palermo & Harry E. Risso, Spring 1991, pp. 57-68

Shear Wave Velocity & S-Factor for Boston Blue Clay, William A. Weiler, Jr., Spring 1991, pp. 85-98

Slurry Wall Construction for a Cut-and-Cover Tunnel, Philip Bonanno, Donald T. Goldberg & Amol R. Mehta, Spring 1987, pp. 75-88

Trenchless Technology Considerations for Sewer Relocation & Construction, Arthur A. Spruch & John Struzziery, Spring/Summer 1996, pp. 95-102

Tunnel Boring Machine Excavation of the Beverly Sewer Tunnel, George W. Hartnell, III, & Andrew F. McKown, Fall 1991, pp. 93-110

Tunneling Projects in the Boston Area, David Woodhouse, Spring 1989, pp. 100-117

Tunneling Through Soft Ground Using Ground Freezing, Helmut Haas, Spring/Summer 2006, pp. 45-70

Underground Engineering for the Central Artery/Tunnel Project, Thom Neff, Spring/Summer 1996, pp. 7-10

An Underpinning Scheme for the Red Line Subway at South Station, Boston, Burton P. Kassap, Spring 1992, pp. 67-86

Understanding-Soil-Behavior Runs Through It, James K. Mitchell, Fall/Winter 1994, pp. 5-28

The Use of Back Analysis to Reduce Slope Failure Risk, James Michael Duncan, Spring/Summer 1999, pp. 75-91

The Use of Slurry Caissons for High-Rise Buildings, James V. Errico & Theodore Von Rosenvinge IV, Fall 1989, pp. 7-22

Using Dynamic Measurements for the Capacity Evaluation of Driven Piles, Samuel G. Paikowsky, Fall/Winter 1995, pp. 61-76

Using Custom Probabilistic Seismic Hazard Analysis Maps Based on U.S. Geological Survey National Seismic Hazard Mapping Procedures, Richard J. Driscoll & Laurie G. Baise, Spring/Summer 2005, pp. 5–18

What Has the Finite Element Method Done for (or to) Geotechnical Engineering?, John T. Christian, Fall/Winter 1999, pp. 73-74

HISTORY

The Boston Harbor Project: History & Planning, Jekabs Vittands, Cheryl Breen & Daniel O'Brien, Spring/Summer 1994, pp. 11-32

BSCES: History & Heritage, Gian S. Lombardo, Fall 1986, pp. 145-157

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Cape Cod Canal, H. Hobart Holly, Spring 1987, pp. 109-113

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The Engineering Center: One Walnut Street, Boston, H. Hobart Holly, Fall 1990, pp. 91-94

In the Footsteps of Giants: The History of the Founders of Earth Pressure Theory From the 17th Century to the Late 19th Century, Nabil M. Hourani, Fall/Winter 1996, pp. 79-92

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Geo.S. Morison, Ch. Eng'r, Francis E. Griggs, Jr., Fall/Winter 2009, pp. 7-40

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