Repair or Replace?

That is the Question

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Much of the North American infrastructure was constructed 30 to 50 plus years ago, without the knowledge or expectation that the structures would continue to be in use today and into the future. Only within the last two decades or so have engineers been routinely charged with the explicit goal of achieving 50, 75, and even 100 year service lives. With funds for construction of new facilities and infrastructure difficult to obtain, the profession is being asked to design repairs to extend the useful service life of existing structures. Moreover, the issue of sustainability is becoming an important consideration to the owner's decision of repair versus replacement.

The present value of concrete infrastructure is about $8 trillion. Of course, extending the service life of all types of infrastructure, including concrete structures, requires an investment in maintenance and repair. It is estimated that about $20 billion is spent annually on repair of concrete structures. And, according to the Strategic Development Council (SDC) (**www.concretesdc.org**), the amount spent on concrete repair to that spent on the total volume of concrete placed annually is about 30% and growing.

The SDC's *Vision 2020: A Vision for the Concrete Repair, Protection, and Strengthening Industry* (May 2006) identified the development of a "concrete repair code" as one of the major objectives in the concrete repair industry. In response to this call for consistency in safety, quality, and integrity, the American Concrete Institute (ACI) formed a technical committee to develop a code for the evaluation, repair, and rehabilitation of concrete buildings. The requirement for consensus national standards demonstrates the growing importance of concrete repair within the construction industry.

To illustrate some of the major issues confronting engineers in a concrete repair project, a case study of a 25 year-old concrete foundation showing signs of deterioration is presented. The decision to repair, as opposed to replace, is discussed in the context of the required extension of service life.

Structural Description

The project, located in the Caribbean, consisted of a concrete foundation originally intended to support power-generating machinery. The portion of interest was an individual section approximately 90 feet (27 m) long by 16 feet (4.8 m) wide with an estimated thickness of five feet (1.5 m). The exact dimensions and structural details were largely unknown, since design drawings were no longer available. *Figure 1* shows the initial site.

During site preparation for new power-generation equipment, staining believed to be associated with petroleum by-products was observed on the concrete surface. Exploratory demolition revealed stain penetration to approximately 20 inches (51 cm) from the top surface. During this exploratory chipping, as well as additional chipping for installation of the equipment, facility personnel reported "weak" and "crumbling" concrete. Several sections of reinforcing steel revealed at the top portion of the foundation exhibited corrosion damage, including extensive section loss. These instances raised concerns of sustained structural performance.

Figure 1: Foundation Site. Note staining and chipped areas to the right of the generator

The nature of the reported problems indicated two general categories of distress: deterioration of the concrete materials and potential deterioration of the reinforcing steel. Though the decision to repair a structure or to replace it is often made during the latter phases of a rehabilitation project, it is important to consider this possibility early so that sufficient data is collected during field operations to facilitate an informed analysis. A regimen of multiple nondestructive testing (NDT) techniques was used to collect data, since no single test method is an effective gauge for overall structural performance.

Field Evaluation of the Structure

The field evaluation included the use of surface penetrating radar (SPR) to identify the content and configuration of unexposed reinforcing steel, impact-echo testing to identify potential corrosion-related concrete delaminations, half-cell corrosion testing (ASTM C876) to identify areas of active corrosion, and visual inspection. Concrete core samples were collected from stained and unstained areas. Selected core samples were petrographically examined (ASTM C856) to assess the impact of the petroleum on the concrete micro-structure, and identify the status of delayed deleterious phenomena such as alkali-silica reaction (ASR) and delayed ettringite formation (DEF). Selected samples were used for measuring concrete strength (ASTM C42), as well as chloride content (AASHTO, T 260-93).

Analysis of Results

Laboratory testing of physical samples provided quantifiable results from the fieldwork. Testing indicated an average concrete compressive strength of approximately 3,300 psi (23 MPa). Chloride content from collected samples indicated a general trend of lower concentrations at greater depth from exposed surfaces; however, even the lowest measured concentration was approximately 1,000 ppm, nearly three times the minimum value theoretically required for initiation of corrosion activity. Rapid chloride permeability testing (ASTM C1202-97) indicated very high chloride permeability in excess of 6,600 coulombs in less than 2 hours, well in excess of the ASTM classification of greater than 4,000 coulombs as "high". Petrographic examination of materials within the stained region did not identify any apparent adverse effects of the staining on the concrete microstructure. However, in samples from both stained and unstained regions, ASR and DEF activity were clearly identified (*Figure 2*). From these results, it appeared that the concrete was deteriorating, though no manifestation of deterioration was apparent by normal visual inspection.

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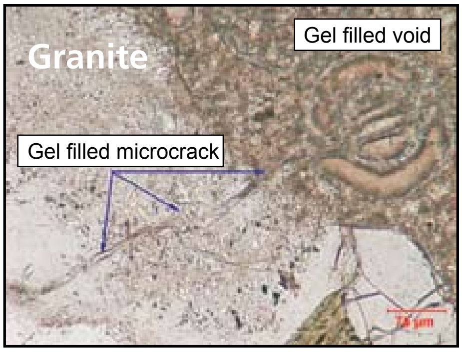

Figure 2: ASR gel identified by petrography (Photograph courtesy of David Rothstein, Ph.D., R.G.)

Additional field testing did not indicate pervasive corrosion damage. Half-cell testing identified some areas with high probability of active corrosion, but this was not a universal trend. Examination of exposed reinforcing steel did not indicate extensive section loss. Impact-echo testing did not identify delaminations consistent with corrosion-related activity.

The combined evaluation of the concrete materials and the reinforcing steel indicated that, while some instances of active corrosion were evident, the structural behavior had reached a plateau: damage related to corrosion and chemical attack was not overwhelmingly apparent, but the conditions were highly conducive to the development and propagation of these deleterious activities. With this conclusion in mind, the issue of project service life was considered in the context of subsequent construction.

Cost-Benefit Analysis

The desired service life of the equipment to be installed was approximately 15 years. Consequently, the service life extension of the current structure would be a minimum of 15 years, and any repairs would have to be appropriately effected for maintaining that service life. A properly designed new structure could certainly provide adequate performance for 15 years, at a minimum. Thus, it became prudent to perform a cost-benefit analysis of the repair and replacement options.

Based on a present value assumption of repair costs at 45 percent of replacement costs, an economic analysis was performed using service lives of 15 years and 30 years for repaired and replaced structures, respectively. Using estimates for interest and inflation rates, the repair option resulted in a present value cost 44 percent less than the replacement option. Variables that were not quantified by the economic analysis included the time required for repair/replacement, the potential benefit of service life exceeding 15 years, and availability of materials. Based on the conclusion from the cost-benefit analysis, the Owner and Contractor decided to repair the existing structure with the intent of providing a minimum of 15 years of continued service life.

Remediation Design

The repair design included replacing the top layer of the foundation, thereby "capping" the underlying material from further ingress of moisture and chemicals contributing to external and internal deterioration. The design required minimizing the potential effect of establishing new "macro-cell" corrosion wherein the presence of new reinforcing steel would accelerate corrosion of reinforcement in the existing structure.

Design repairs included removing the top surface of the foundation beneath critical machinery. In total, this included approximately 60 feet (18 m) of the 90 foot (27 m) structure. Concrete was removed to 3 inches (7.6 cm) beneath the depth of the existing reinforcing steel, with a minimum depth of 12 inches (30.5 cm). Based on the assumption that the design of the original foundation was adequate for in-service structural performance, the repair design included replacement of an equivalent area of steel for the removed reinforcing steel at the top of the foundation. Three-inches (7.6 cm) of concrete cover was specified due to the marine environment and previously observed distress. *Figure 3* shows the final configuration of the replaced reinforcing steel.

Figure 3: Replacement reinforcing bars in place

Across the interface between the new and existing concrete, hooked dowels were placed at spacings determined from analysis of seismic and operational dynamic loads (*Figure 4*). To account for the potential of diminished bond between the new and existing concrete, dowel spacing was decreased corresponding to an assumed reduced contribution of shear friction. At the vertical interface between the existing concrete and the repair concrete (*left edge of Figure 3*), horizontal dowels were installed with sheathed ends in the new concrete to allow for shrinkage.

The repair concrete was specified to minimize the potential for corrosion activity and chemical attack in the new materials. Specifications included a relatively low water-cement concrete mix (0.45), using Type II cement and a high-range water reducer. In response to findings from petrographic examination suggesting that the very high chloride content of the existing structure was largely the result of chlorides admixed from local marine aggregates, the repair concrete incorporated imported granitic fine aggregates and local limestone coarse aggregates, both with low chloride contents.

Complementing the new concrete materials, the repair design used embedded galvanic anodes for corrosion protection. Disc-type anodes, shown in *Figure 4*, were installed at spacings recommended by the manufacturer. While similar protection could have been achieved using a corrosion-inhibiting admixture or a combination of materials, the pre-installation of the anodes offered greater benefits in quality control for this application and location, while still minimizing potential corrosion acceleration due to dissimilarities between the new and existing materials.

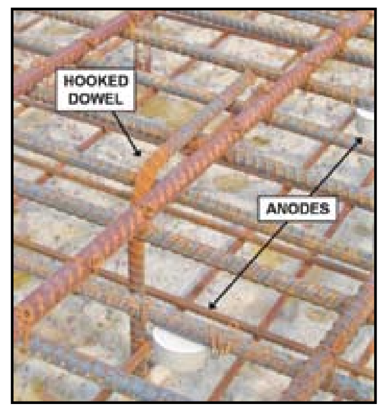

Figure 4: Hooked dowel and disc-type anodes prior to installation of repair concrete

Quality control of the repair operations consisted of testing of the repair surface after removal of the old concrete, inspection of the site before placing the new concrete, observation during repair operations, and strength testing of the new concrete. The repair surface was evaluated using pull-bond testing (ASTM C1583-04). The average pull-bond strength from test locations throughout the repair area was 173 psi (1.19 MPa), greater than the 150 psi (1.03 MPa) value specified by design. This step helped to ensure good bond and minimize the effects of "bruising" from chipping operations. New concrete was delivered to the site by truck and was placed by crane (*Figure 5*). The concrete was installed successfully, with operations concluding immediately before an afternoon rainstorm. Subsequent testing of the sample cylinders indicated an average 28-day compressive strength of 6,400 psi (44 MPa), well in excess of the 4,000 psi (28 MPa) design specification.

Figure 5: Final stages of repair construction

Conclusions

This project represents a successful "economic" use of technology and materials. Successful rehabilitation requires an appropriate paradigm by which aging infrastructure can provide an opportunity to implement new technologies, including NDT techniques and construction materials and methods. This perspective requires thorough consideration of how all involved parties are affected by the decision to repair or replace entire structures and subcomponents. Furthermore, subsequent remediation design must make economic use of materials, as measured by both project costs and tolls on natural resources. Even as new skyscrapers and signature bridges appear, they are best considered in the context of the disproportionately larger number of more modest and less venerated structures. As time passes, these modest structures will demand equal attention from public and private sectors, who will ultimately have to answer the question whether to "repair or replace".▪

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