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SELECTING APPROPRIATE LONG TERM SOLUTIONS FOR REINFORCED CONCRETE BRIDGE COMPONENTS IN CORROSIVE ENVIRONMENTS

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ABSTRACT

Bridge Engineers and maintenance personnel must often decide what to do with a bridge in order to meet future traffic projections, maintain conformance with current safety standards or otherwise keep the structure in service for decades into the future. Selecting the most technically viable and cost-effective solution for a concrete bridge structure in a corrosive environment is a formidable task. The alternatives typically span the extremes of 'doing nothing' to 'complete replacement of the structure'. Most often however, some type of corrosion prevention/rehabilitation measure is deemed more appropriate and, in these cases, another decision regarding the specific approach to be employed needs to be made. In any case, the process has historically been arduous, with no procedures, standards, or other information available to assist in the analysis. This paper presents a scientifically based, step by step process that has evolved over the last decade for selecting a technically viable and cost-effective solution for a given bridge structure in a corrosive environment. The methodology, which includes mathematical deterioration modeling, is illustrated through three case studies. The case studies also indicate that the new process will have a positive impact on the use of cathodic protection technology.

INTRODUCTION

Among the many problems facing our nation's infrastructure is continued deterioration of reinforced concrete bridges; particularly that associated with corrosion of reinforcing steel. Engineers are constantly faced with the challenge of detecting concrete deterioration, corrosion in its early stages (that is, in undamaged areas), and providing appropriate maintenance in the form of repair and long-term protection for a structure to extend its service life. The ever-increasing cost of repairing, rehabilitating, and replacing deteriorated reinforced concrete bridges is a source of major concern for highway agencies. The costs are astronomical, approaching 3.5 to 4 trillion dollars, according to the Secretary of Transportation's report presented to the United States Congress[1] in 1993.

The initial response of the construction industry to this problem has been to repair the damage caused by the corrosion process. Unfortunately, many still rely on this strategy to combat corrosion of reinforced concrete structures. Repair is defined as a process implemented to restore a concrete component to an acceptable level of service[2]. For example, a column can be repaired to restore its structural capacity to the original design conditions. Repair methods alone, however, do little to address the cause of deterioration. Therefore, in the case of reinforcing steel corrosion, simple repairs typically fail within a few years since nothing is done to mitigate or stop the primary deterioration mechanism.

More often than not, to effectively combat corrosion of reinforcing steel in concrete, some form of rehabilitation is required. Rehabilitation is defined as procedures implemented to restore a concrete component to an acceptable level of service while simultaneously mitigating or stopping the process responsible for the deterioration[2]. Because rehabilitation includes addressing the deterioration process itself, the additional service life realized is typically much greater than for the repair only case.

Assessment and selection of a cost-effective solution for a deteriorated bridge structure can be a challenging task and several methodologies have been proposed or employed for this purpose. This paper focuses on a comprehensive approach termed 'SALTS' (Selecting Appropriate Long Term Solutions), that has evolved over the past decade. This methodology incorporates a multifaceted, logistical approach to analyzing potential future courses of action, from 'doing nothing' to 'repair' to 'rehabilitation' to 'complete replacement'. The resulting information allows one to directly compare alternative solutions so that objective and well-founded decisions for allocating maintenance resources can be made. In each case, a solution, specifically tailored to the target structure, is developed. Upon utilizing the 'SALTS' process, it becomes very apparent that one solution is not and cannot be the most appropriate or cost-effective for all concrete bridge structures in a given State, or DOT district, or even within the same county.

The facets of 'SALTS' involve:

- Obtaining information on the condition of the structure and its environment.
- Applying engineering analysis to the information.
- Identifying options that are technically viable for that particular structure.
- Performing life cycle cost analyses.
- Defining the most cost-effective alternative for rehabilitating the structure.

The Florida Department of Transportation (FDOT) has been pro-active in the implementation of this approach as evidenced by completed projects on the Bryant Patton Bridges[3,4], the Venetian Causeway[5], and the Escambia Bay Bridges[6]. These case studies are discussed later in this paper. The FDOT intends to adopt the 'SALTS' philosophy as a standard procedure in their strategic planning and decision making process for the upkeep of their bridges.

'SALTS' METHODOLOGY

The process of selecting an appropriate long-term solution for a given concrete bridge component in a corrosive environment is represented in Figure 1. The first step involves a "Review of Historical Information" on the structure. This includes a careful examination of bridge drawings, previous bridge inspection reports, any existing survey reports, and available information on the environmental conditions at the bridge site. Acquired information should include the location, size, type and age of the structure, any unusual design features, environmental exposure conditions, reinforcing steel details, type of reinforcement, repair and maintenance history, and presence of corrosion protection systems.

This information is then used to develop a specific scope for a thorough "Condition Survey" of the structure. A typical condition survey involves two interrelated parts: (a) corrosion evaluation and (b) concrete evaluation. The objective of the condition survey is to determine the cause, extent (in terms of total area effected), and magnitude (in terms of severity) of the problem and to provide necessary data for development of a deterioration model. Based on the specific scope developed for the target structure, some or all of the surveys and tests listed below are utilized in the condition survey.

Corrosion Evaluation Test Methods:

1. Visual Inspection including Crack and Spall Surveys

2. Delamination Survey
3. Depth of Concrete Cover Measurement
4. Chloride Ion Analysis
5. Depth of Carbonation Testing
6. Electrical Continuity Testing
7. Concrete Relative Humidity and Resistivity Measurements
8. Corrosion Potential Mapping
9. Corrosion Rate Measurement
10. Determination of Cross Section Loss on Reinforcing Steel
11. Measurement of Concrete and/or Corrosion Product pH

Concrete Evaluation Test Methods:

1. Visual Inspection
2. Petrographic Analysis
3. Compressive Strength Testing
4. Chloride Permeability Testing
5. Measurement of Specific Gravity, Absorption, and Voids in Hardened Concrete

The above have been standardized by the American Society for Testing and Materials, the American Association of State Highway and Transportation Officials, the American Concrete Institute, or are test methods commonly adopted by engineers involved in conducting condition surveys. Detailed information regarding these test and survey techniques is available[7,8].

"Data Analyses" of the field and laboratory test results and "Development of a Deterioration Model" follow the condition survey. When using the 'SALTS' approach, statistical distributions of the condition survey data are determined and used in the deterioration model. For bridge structures, this often includes separate analyses for different portions of the structure, that is the deck, superstructure and substructure. Also, when the bridge structure is located in a marine environment, the substructure components may be further divided by elevation. Thus, the tidal, splash, and above splash zones are analyzed independently. In addition to the statistical analyses, chloride profile data are used to obtain apparent chloride diffusion coefficients, surface concentrations, and bulk concentrations.

Deterioration modeling can then be accomplished in several ways, although the fundamental basis of most models relates to the corrosion initiation-propagation concept first established by Tuutti[9]. The three-phase process shown in Figure 2 represents this concept. During the corrosion initiation phase (Phase I), chloride concentration builds up at the steel surface until the threshold value needed to trigger the onset of active corrosion is reached. This phase begins when the structure is constructed (t_0) and ends when corrosion of the reinforcing steel initiates (t_i). At this point, no corrosion-induced damage has occurred. The corrosion propagation stage is divided into two phases (Phases II and III). Phase II begins at time t_i and ends when the first signs of external concrete damage are observed (that is, at time t_d). The corrosion process continues (often in an accelerated manner) in Phase III and the associated concrete damage worsens until remedial measures are taken or the structure reaches the end of its functional service life.

In the 'SALTS' methodology, the primary purpose of developing a deterioration model is to quantitatively predict future corrosion-induced concrete damage as a function of structure age. Generally, deterioration models are a set of mathematical relationships between corrosion condition data and remaining service life, future condition of the structure and/or estimated future damage. Two reported methods use the concept described above in terms of a chloride diffusion period, corrosion-induced cracking period, and additional damage development period[2]. Another model uses the condition of a structure at two different points in time to predict the future condition of the structure[10]. The models developed for FDOT[11,12] include the use of statistical distributions of concrete cover, chloride diffusion coefficients, and surface chloride concentrations to estimate the distribution of time for corrosion initiation and appearance of external damage over bridge substructure components. The output of these models is a damage function indicating the amount and location of repairs needed as a function of structure age.

The output of any deterioration model should ultimately provide insight regarding the optimum time to repair or rehabilitate a bridge structure or select components of a structure. This information is particularly critical when conducting life cycle cost analyses.

The condition survey data, output from the deterioration model, and the allowable damage which can exist on a particular bridge component before it must be repaired are used in the next step of the methodology, "Review and Selection of Potential Alternative Solutions Based on Technical Viability and Desired Service Life." In this part of the process, a number of technically viable options are defined. The number and type of applicable solutions are also dependent on the type of structural component being considered (for example, a deck versus superstructure components versus substructure components). Typically, the selected options can be broadly categorized as follows:

Do Nothing - Sometimes no remedial measures are needed to meet the functionality and service life requirements for a given structure. More often, however, remedial measures can only be postponed for some period of time.

Repair - Examples include patching, overlays, encasement, jacketing, sealers, and membranes.

Rehabilitate - Examples include patching, overlays, encasement, jacketing, cathodic protection, electrochemical chloride extraction, and corrosion inhibitors.

Replace – Although complete replacement of a structure is sometimes the optimum solution, partial or selective replacement is often a viable alternative. This category also includes measures where severely damaged structural components remain in place, but with supplemental supporting members installed (for example, crutch bents).

Depending on the exact procedures followed, some potential solutions fall into more than one category. For example, overlaying a concrete bridge deck may be categorized as a repair or rehabilitation approach depending on the degree to which all chloride contaminated concrete is removed prior to placing the overlay.

Combinations of the above options should also be considered. For example, partial or selective replacement can be used in conjunction with some type of rehabilitation measure on other structural components. Another example involves utilizing cathodic protection in areas that have suffered the severest corrosion and conventional repairs in areas where corrosion has only recently initiated. On the other hand, the analyses may show that the overall best approach is to repair or rehabilitate only those components that have experienced a critical level of damage with future measures taken on additional concrete elements as needed.

The final list of potentially acceptable solutions is then used in the last step of the 'SALTS' process, "Life Cycle Cost Analysis (LCCA)." This stage involves identifying and quantifying all significant costs of maintaining the structure over a specific length of time. Life cycle cost analyses allow one to compare and evaluate the total costs of competing solutions based on the anticipated life of each solution and the desired service life of the structure. The value of a potential solution includes not only consideration of what it costs to acquire it, but also the cost to maintain it over a specified time period. Since the owner of a structure pays the life cycle cost, the value of each solution being considered must be estimated. Therefore, to perform LCCA, one must estimate the initial cost, maintenance cost, and service life for each alternative selected earlier in the process. Also, the amount of concrete damage that can be sustained prior to implementing some type of remedial measure must be estimated. For these purposes, the most critical information required from the deterioration model includes quantified damage predictions and the time at which the extent of damage will reach allowable limits.

Basically, LCCA involves calculating equivalent costs for each potential solution over a given time frame. Equivalent costs are typically developed by equating all costs to a common time baseline using a discount rate to adjust for variable expenditure years. One must also hold the economic conditions constant while the cost consequences of each alternative are being developed. That is, the same economic factors must be applied to each alternative using a uniform methodology. Of primary interest in the life cycle cost analysis is to calculate the total net present value of all future costs (that is, calculate costs occurring at different times to equivalent present costs in present dollars) for each potential solution over a given time period. This information can then be used to identify the "Optimum Solution" for the target structure and time period investigated.

EXPERTISE REQUIRED FOR A BRIDGE REHABILITATION ASSESSMENT PROJECT

Implementation of the approach described above for selecting appropriate long-term solutions for concrete bridge structures in corrosive environments requires expertise in several areas including:

- Corrosion Engineering
- Concrete Materials
- Mathematical Modeling
- Engineering Economics
- Structural/Civil Engineering

Consequently, to ensure that the appropriate expertise is available for each aspect of the project, a team approach is typically employed.

CASE STUDIES

Three case studies involved in the evolution of the 'SALTS' methodology are discussed below. These projects were completed in 1992, 1994, and 1998 respectively.

Bryant Patton Bridges[3,4]

The Bryant Patton Bridges were constructed in 1965 at St. George Island, Florida. The bridges span almost 4.0-km (2.5 miles) of the Apalachicola Bay in Franklin County. Due to concerns raised in Inspection Reports regarding significant corrosion induced damage on prestressed piles, a project was initiated to evaluate the corrosion status of the piles, determine the most appropriate repair approach, and design a long-term protection system. The primary objective of the project was to ascertain the most appropriate and cost-effective corrosion protection system that would require minimal maintenance and that could be expected to provide a life extension in the range of 15 to 25 years. All protection systems were to be considered in the selection process, including electrochemical and non-electrochemical techniques.

The bridges have a total of 856 standard 50.8-cm (20.0-in.) square prestressed pilings. All piles were constructed with 20 prestressed tendons, spirally wrapped with a #5 gauge wire. A total of 160 piles were reported deficient in 1990 with an additional 17 piles found to be deficient in 1991. Some of these piles were judged to have deteriorated to the point where replacement or additional new piles may be required. Deterioration of concrete due to corrosion of the spiral wire and prestressed strands was the most significant problem. In some cases, cracks induced in the piles due to improper driving techniques during construction served to accelerate the corrosion process. The corrosion-induced concrete damage was concentrated primarily in the splash zone. Previous repairs utilizing fiberglass jackets filled with either cementitious or polymer modified material were not effective, and many were found to have completely failed. Corrosion of embedded steel was found to have continued even after the repairs.

All available historical information on the bridges was reviewed and a limited corrosion condition evaluation was conducted on a total of four piles. Sampling and testing included collection of core samples for chloride content analysis, visual and delamination surveys, corrosion potential measurements, concrete resistivity measurements, concrete cover measurements, electrical continuity testing, and concrete pH measurements. Based on the results of the corrosion condition evaluation and the desired life extension, 10 different rehabilitation approaches were identified as being technically viable. These included various combinations of repair methods and long term protection methods. Another approach involving standard pile jackets was also included for comparison purposes. In addition, the various alternatives considered repairing and protecting only deficient piles immediately with all remaining piles being repaired and protected 10 years later versus repairing and protecting all piles at the same time. Similar to all approaches was the need for crutch bents in select locations.

To determine the most appropriate and cost-effective means to repair and protect the piles, life cycle cost analyses were performed to rank the alternative solutions based on the Present Value of initial construction costs and subsequent repair and maintenance costs for 30 years of additional service life. Installation of crutch bents was not considered in the comparative life cycle cost analysis since this was common to all selected alternatives. The most technically and economically appropriate solution was then recommended. This consisted of repairing (using conventional gunite) and cathodically protecting (using a perforated zinc sheet/bulk anode system) deficient piles now. The cathodic protection system was intended to provide corrosion protection in the tidal and splash zones while periodic conventional repairs where to be installed in the unprotected portion (that is, above the splash zone) of the piles. The recommended approach was implemented and all associated work was completed by mid-1995.

Venetian Causeway[5]

The Venetian Causeway is 4.4-km (2.75 miles) long and consists of a roadway on 11 islands and a series of 12 bridges between the islands. The Venetian islands and causeway are located in Miami and Miami Beach, Florida. The bridges consist of a roadway 11-m (36-ft) wide and a 1.2-m (4.0-ft) sidewalk. Ten of the bridges are fixed low-level concrete arch bridges, and two are bascule bridges with concrete arch approach spans. The spans range from 14.0-m to 18.3-m (46.0-ft to 60.0-ft) long except at expansion joints where 6.1-m (20.0-ft) spans were typically used. The fixed span concrete bridges are constructed of variable depth cast-in-place tee beams. There are five tee beams approximately 2.6-m (8.5-ft) center-to-center in each span. Three diaphragms run in the transverse direction in each span. In 6.1-m (20-ft) spans, three beams and two diaphragms were used. The deck cantilevers 1.0-m (3.3-ft) on each side of the exterior tee beams. The superstructure is supported on square pier columns, which rest on concrete pile caps. The precast piles are founded on coral sand about 3.0-m to 6.1-m (10-ft to 20-ft) below the bay bottom.

The Venetian islands and the bridges were built in the 1920's. Both the concrete arch spans and the bascule spans deteriorated in the harsh marine environment. As a result, the structures have been repeatedly repaired since their construction. Repairs typically included application of gunite, therefore, a significant portion of the girders, diaphragms and bridge deck soffit are covered with gunite layers of varying thickness. In addition to guniting, sometime in the 1950's or 1960's, a black, bituminous coating was applied to the underside of the bridges. Overspray from subsequent gunite applications resulted in gunite being applied directly on top of this coating.

Engineering studies were conducted in 1968, 1979 and 1991. Replacement of the bridges was recommended as the most viable alternative in all of these efforts. As a result of concerns raised by the Venetian Islands Improvement Association regarding replacement of the historic causeway, another engineering study was initiated in late 1992. The primary objective of this effort was to revisit potential preservation and rehabilitation schemes for the causeway. The resulting report provided three rehabilitation alternatives all of which included cathodic protection of the bridge deck soffits, arch girders, pier columns and abutments.

A study was then initiated in 1994 to select and design appropriate cathodic protection systems for the bridges. Since previous investigations focused on defining the best course of action and not details related to selection, design and installation of cathodic protection systems, a final field investigation was performed to obtain the necessary information. The final study included visual and delamination surveys, chloride sampling and analysis, concrete cover measurements, electrical continuity testing, corrosion potential measurements, corrosion rate measurements, cross-section loss measurements on the reinforcing steel, and petrographic analysis.

It was previously determined that the bascule spans on the two bascule bridges, several concrete spans on one bridge, and the cantilevered portion of all the bridge decks were to be replaced. Therefore, no testing or other survey work was conducted in these areas.

The results of the corrosion condition evaluation indicated that corrosion induced damage was limited to about 5% of the areas surveyed in spite of the fact that the chloride ion concentration at most locations was in excess of the threshold value normally assumed to initiate corrosion. Also, corrosion potentials in the majority of the areas tested were found to be in the active range, corrosion rate data indicated high localized corrosion rates, and the cross-section loss on reinforcing steel varied from 0% to 100%. In addition, significant electrical discontinuity was detected between regions of the reinforcing steel in all members.

Although very high levels of chloride ions and active corrosion were found, the extent of concrete damage was relatively lower than expected. This discrepancy was attributed in part to extensive repairs that had been completed on the bridges numerous times over the previous 50 years as well as to generally good cover over the reinforcing steel.

Although the most appropriate solution from a purely corrosion point of view was considered to be installation of a cathodic protection system, this approach was discounted due to excessive costs required to establish an acceptable degree of electrical continuity among the reinforcing steel elements. It was also believed that system installation would be more difficult and costly due to the proximity of the structures to the water. Consequently, the recommended rehabilitation approach involved a conventional concrete repair designed to provide the longest possible service life extension in the harsh environment around the bridges. This included removing all delaminated or otherwise unsound concrete. Additionally, sound concrete was to be removed in both directions along all exposed corroded steel until no corrosion was found on the steel. In all cases, concrete was to be removed at least one inch around the entire circumference of all exposed steel and severely corroded steel was to be replaced. Subsequently, all exposed steel and excavated areas would be cleaned and patched with a high quality, conventional shotcrete or concrete depending on the type of concrete component. The recommendations also called for application of a passivating corrosion inhibitor on the reinforcing steel. Finally, cracks that remained after the concrete removal process were to be repaired by epoxy injection. The recommended approach was implemented and all associated work was performed in 1997 and 1998.

Escambia Bay Bridges[6]

The Escambia Bay bridges are parallel structures that were built in 1966 to span the Escambia Bay between Santa Rosa and Escambia Counties near Pensacola, Florida. Each bridge is 4.1-km (2.6 miles) long and carries two lanes of traffic and one shoulder. The substructure of each bridge consists of a total of 223 bents. The bents in the higher elevations of the bridges are comprised of 1.37-m (54-in.) diameter Raymond piles that are connected by struts above the high tide mark. Smaller diameter 0.91-m (36-in.) Raymond piles support the lower elevations of the bridges. There are no struts in the lower elevation sections of the bridges. The bents at the channel spans consist of crash walls and square columns.

In 1996, the Florida Department of Transportation (FDOT) began reviewing alternatives for upgrading these bridges to accommodate future traffic projections and to bring the physical characteristics of the bridges in conformance with current safety standards. The alternatives included widening or replacing the bridges. To aid in the decision process, the Materials Office of FDOT conducted a preliminary corrosion condition assessment of the bridge substructure components in 1996. This investigation revealed that the chloride concentration at the depth of the reinforcement in the Raymond piles had reached the level normally associated with the onset of corrosion. In addition, the struts exhibited advanced corrosion induced damage and the crash walls showed extensive cracking. Previous repairs on the struts included gunite patching and epoxy injection of cracks. The cracks in the crash walls had also been epoxy injected.

Based on these findings, FDOT determined that a more extensive evaluation was needed and that this should include a comprehensive condition survey, development of a deterioration model, selection of potentially viable rehabilitation alternatives, and life cycle cost analysis. To facilitate selection of the most appropriate future course of action, FDOT also determined that the rehabilitation strategies and life cycle cost analysis should be based on maintaining the substructure components of the bridges for 20, 40, and 60 years.

The detailed investigation commenced in 1997. The corrosion condition was assessed by visual observation, direct examination of reinforcement, and electrochemical corrosion measurements. Chloride penetration was determined by obtaining chloride penetration profiles from extracted concrete cores. Reinforcement cover was measured by direct observation. The chloride profile data were analyzed to obtain apparent chloride ion diffusivities, surface concentrations, and bulk concentrations. The deterioration model used the statistical distributions of concrete cover, chloride diffusion coefficient, and surface concentration to estimate the distribution of time for corrosion initiation and appearance of external damage over the bridge substructure. The output of the model was a damage function indicating the amount and location of repairs needed as a function of bridge age. Figure 3 shows an example output for the piles[12]. At the time of the study, the bridge was 31 years old. Therefore, the 20, 40, and 60 year future projections correspond to 51, 71, and 91 years in Figure 3. A sensitivity analysis was also conducted to assess the effect of assumed chloride ion concentration thresholds, presence of a minimum cover, and variation in the time for corrosion propagation on the deterioration model output.

Due to the quality of the concrete used to fabricate the piles, the construction methods employed, and environmental exposure conditions, the 31 year old piles were found to be in excellent condition. However, future corrosion damage was considered inevitable based on the corrosion condition survey results and projections provided by the deterioration model. Fortunately, the timing of this project was such that the ensuing corrosion problems were deemed to be controllable in a cost-effective manner using select rehabilitation strategies. The deterioration model indicated that the severity of future corrosion damage on the piles varied with elevation. Thus, three separate zones were identified on the piles: the tidal zone, lower splash zone, and upper and above splash zone.

For the piles, the rehabilitation alternatives selected for life cycle cost (LCC) analyses included conventional patching at various time periods and installation of several different types of cathodic protection systems at specific time intervals. Two approaches were formulated for the struts. These were complete replacement with similar struts at the same elevation versus installation of new struts at a higher elevation. The cost of maintaining the crash walls was considered negligible in comparison to the other substructure components. Consequently, selection of rehabilitation alternatives and LCC analyses for the struts and crash walls were not performed.

The detailed evaluation ultimately led to a decision to widen rather than replace the bridges. The evaluation results also provided a logical basis upon which FDOT could rely on to select an optimal rehabilitation alternative for each substructure component type.

CONCLUSIONS

The 'SALTS' methodology provides bridge owners with a useful tool to explore and compare potential future courses of action, from 'doing nothing' to 'repair' to 'rehabilitation' to 'complete replacement'. The resulting information allows one to make objective and well-founded decisions for allocating maintenance resources.

As the utilization of this comprehensive approach becomes more widespread, it is projected to have a significant positive impact on the use of cathodic protection technology. This is due to the process by which decisions are made in the methodology and the fact that life cycle costs over an extended time period are compared rather than initial construction costs only.

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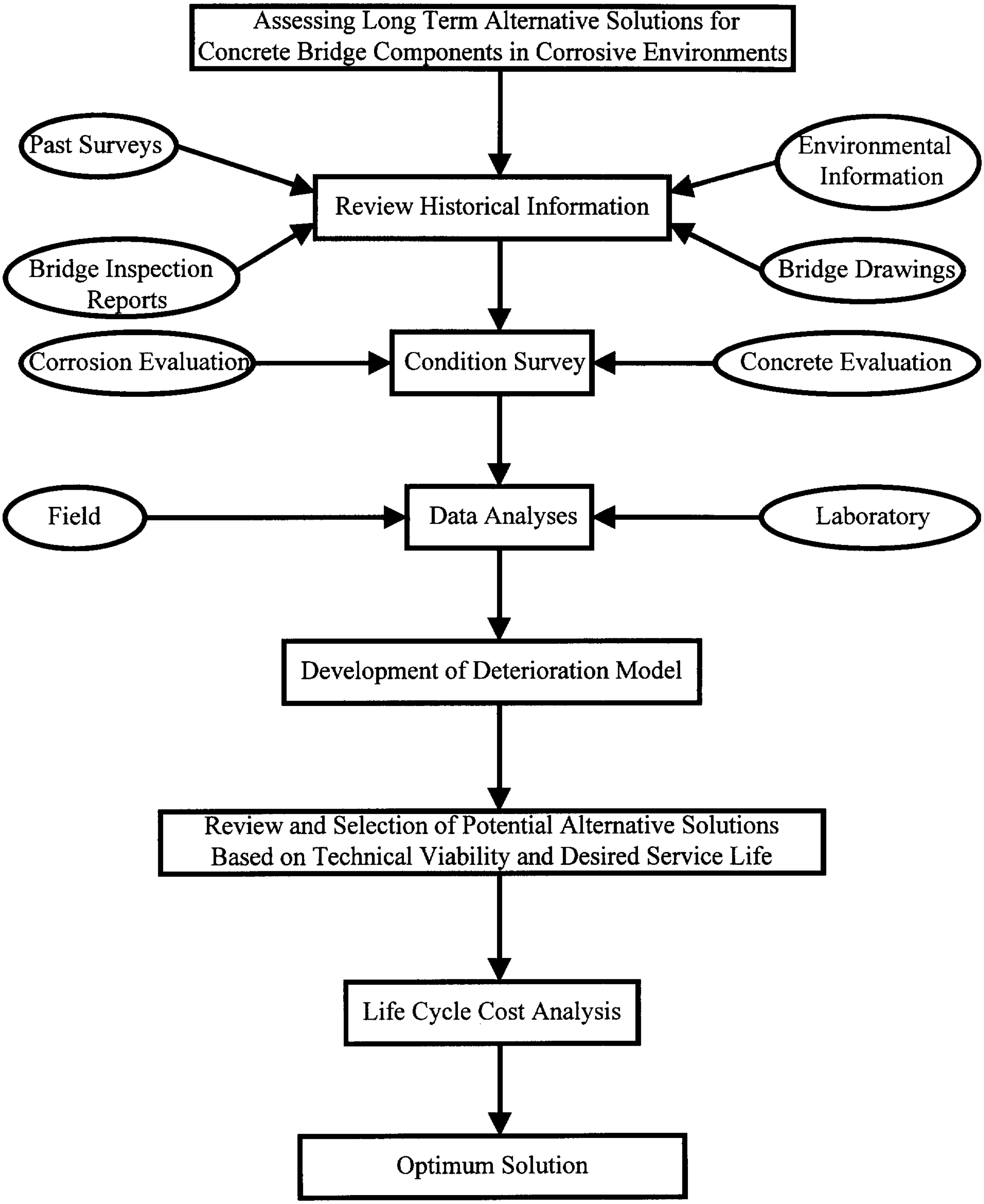

Figure 1. Methodology for Selecting Appropriate Long Term Solutions.

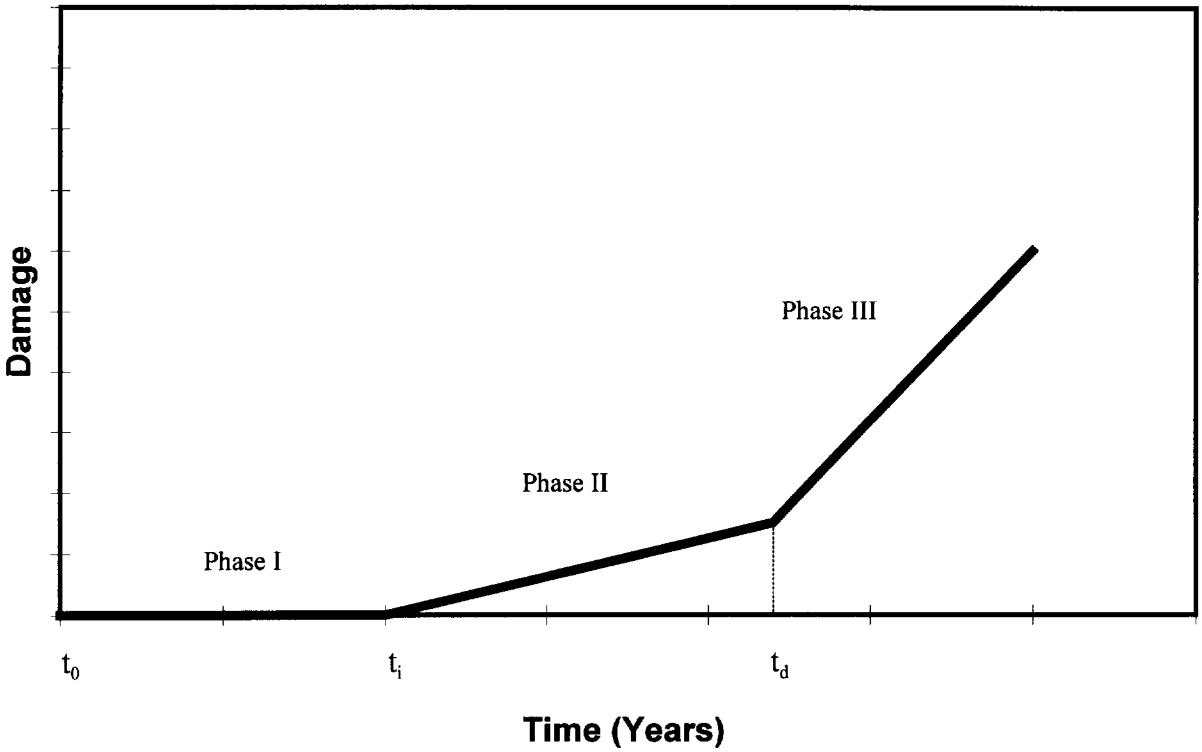

Figure 2. Simplified deterioration model.

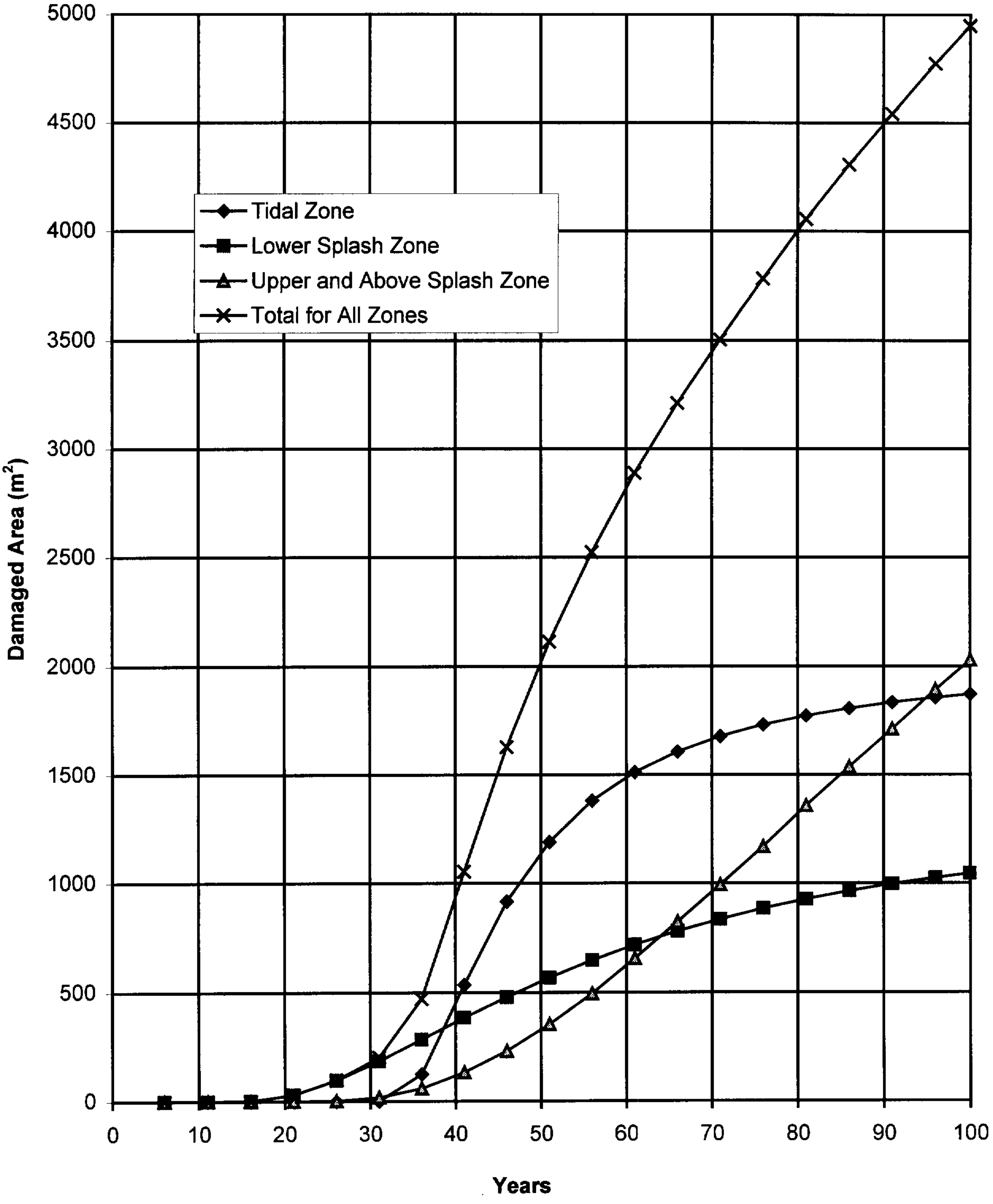

Figure 3. Deterioration Model Output[12].