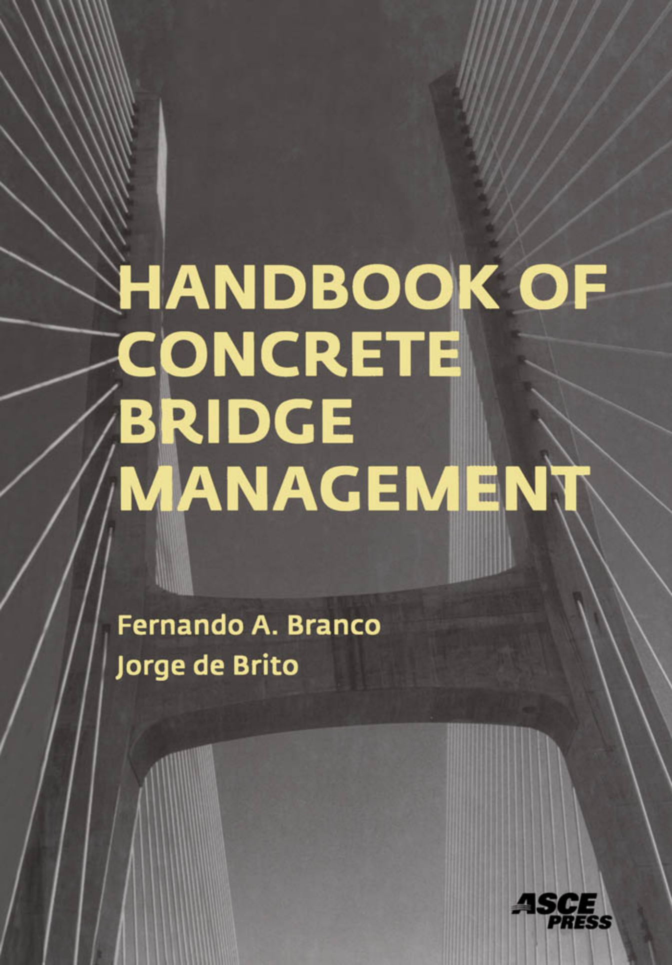

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Fernando A. Branco
Jorge de Brito

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PREFACE

The high levels of deterioration in bridges and the associated repair costs led to important advances in recent years, both in the technical and management fields. Nowadays bridge engineers care more about design and construction with durability, and bridge authorities optimize their investments, to keep their bridge systems functioning, through the use of bridge management systems (BMS).

This book presents the international present situation and the near-future evolutions, related to the design and implementation of bridge management systems and is customized to be used in concrete bridges, with a special economic emphasis on road bridges. The concept of service life both structural and functional is thoroughly discussed, as well as its implications in the design, construction and maintenance of bridges.

Testing and monitoring are essential parts of structures' safekeeping and are described in detail, according to the specific needs of the bridges' authorities. Every bridge designer and builder needs to introduce in his practice the concepts of "design for durability" and "construction for durability". All the aspects involved are discussed, aiming at conceiving bridges that not only are safe, but also easily inspected and maintained, i.e. durable.

Several bridge management systems throughout the world with special emphasis on North America and the European Union are described in order to introduce a standard architecture made of a database, an inspection module and a decision module. Each of these modules is then described in detail.

The computer-based database comprises a wide range of information, for whose organization guidelines are provided. This information is paramount in order to implement inspection strategies, based on knowledge-based, objective and rational criteria, in terms of inspection depth, data collected and report making.

Current maintenance prioritizing strategies are proposed, based on inspection data, defects classification and decision-making criteria.

Before decisions concerning structural repair, strengthening or replacement are made, a long-term economic analysis must be performed. A global costs model is presented and the corresponding decision-making criteria are described.

As a whole, this book presents important tools concerning the life cycle management of concrete bridges and due to its global overview is particularly important for engineers dealing with design, construction or management of bridges as well as for bridge authorities or road administrations.

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PART 1

Basic Concepts

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CHAPTER 1

INTRODUCTION

1.1. Historical Note

The problem of the durability of structures is as old as the history of construction itself. Looking at ancient buildings, we still can contemplate some made of stone, some made of brick, and some very rare wooden constructions. Nevertheless, it is well known that wood was the construction material of choice in the past because of its abundance, workability, light weight, and good tensile and compression strength. Nevertheless, ancient builders knew that the durability of wood was poor (fire, termites, decay, etc., shortened its service life), so when important monuments were built, those intended to last well into the future, stone was chosen as the best material, because of its durability. It is because of the durability of stone that it is still possible to appreciate Stonehenge, the Pyramids, and the Acropolis.

This knowledge of materials was also applied to bridges, specifically to the first road network that appeared within the Roman Empire. To rule over almost all of Europe and North Africa, and to allow their army quick access to anywhere in the Empire, the Romans built several thousand kilometers of roads, where bridges were important elements (Figure 1-1). Stone was chosen as the main construction material because of its strength and durability. New technologies were developed to build stone bridges, including the circular arch supported on a wooden scaffold, allowing easy repetition of the arch construction and the use of hydraulic mortar for connecting the stones. This technology, where such durability details as the drainage system were not overlooked, allowed several Roman bridges, 2,000 years old, to continue to be in use in Europe (Figure 1-2).

The importance of bridges in the Roman Empire led to a mythical aura associated with their builders, which was symbolized by the fact that the Jupiter Great Priest was called "Pontifex Maximus," or the Supreme Bridge Builder, a title that is still associated with the Pope.

A landmark text of this period is "De Architectura," which was written by Vitruvius more than 2,000 years ago (Vitruvius, I B.C.). Vitruvius Polio accompanied Julius Caesar in the Iberian and Gallic military campaigns and later became the bridge engineer of Emperor Augustus. His text, consisting of 10 books, presented the construction technologies of the Roman times in great detail, including measures for deterioration prevention and maintenance.

During the Middle Ages, road development almost came to a halt and it was the maintenance of the Roman network that supported communications in Europe. Bridges were

Figure 1-1. 2,000-year-old Roman bridge, still in service, in Chaves, Portugal

built and maintained based on simple rules as can be seen in the "Mappae of Claviculae" written in the ninth century, containing the "Dispositio Fabricae Pontibus," describing the existing orally transmitted rules at that time (Prade 1986). This document presents techniques for the design of foundations, columns, and arches based on human dimensions, thus making it a universal manual.

From the tenth century to the Renaissance, due to a new surge of development in Europe, the road and bridge network underwent a new improvement. Financing of the construction was frequently obtained from taxes, tolls, or even charity, the latter usually organized by religious communities—the bridge builders' friars—that became associated with bridge building activity. Among them, St. Benezet, the builder of the Avignon bridge in France (Figure 1-3), became one of the patron saints of bridge builders (Gordon 1980; Prade 1986).

The Renaissance was a period of great exchange of technical ideas. Bridge construction appeared associated with architects, who basically designed them using the rules of "nice building." The construction itself relied on the specialists of the construction equipment (the "engines"), from which arose later (in the seventeenth century) the term "engineer" associated with the construction of military infrastructures. Bridges became the meeting point of those who conceived the structure and the construction methods (then known as architects) and those who built them (the building masters).

In this period of new ideas, it is interesting to see the approach to rehabilitation of structures described by Leonardo da Vinci in a letter to the builders of the Duomo di Milano (Milan Cathedral): ". . . you know, in medicine, you can't begin a treatment or even a surgery without knowing well the patient, . . . the same occurs to the ill Duomo that, before the rehabilitation, needs a diagnosis by a doctor architect. . . ." (Giuffrè 1988).

Figure 1-2. Detail of the drainage system of a Roman bridge in Chaves, Portugal

In the transition between the seventeenth and eighteenth centuries, road design and maintenance first became a state issue in France, and a technical department consisting of king architects and military engineers was implemented to build and maintain roads, bridges, channels, and harbors. Texts related to bridge building and maintenance were written, and technical information began to be disseminated in a consistent way (Prade 1986).

In England, the use of steam energy led to the development of railways, and the horse was finally replaced by a new transportation system, leading to the birth of the railway network. In order to cope with the large spans and important loads of the railway bridges, a new material also appeared. Wrought iron, and later steel, finally replaced stone as a construction material (Figure 1-4).

Steel is a very strong material (as compared with stone), but unfortunately corrosion problems led to the development of maintenance strategies for railways bridges from the beginning of their service life. In fact, the first bridge management methodologies appeared associated with railway bridges, as steel became the main material used for bridge construction until the end of the nineteenth century.

With the beginning of the twentieth century, the automobile industry and associated road networks increased exponentially. Thousands of kilometers of roads and bridges were built throughout the world.

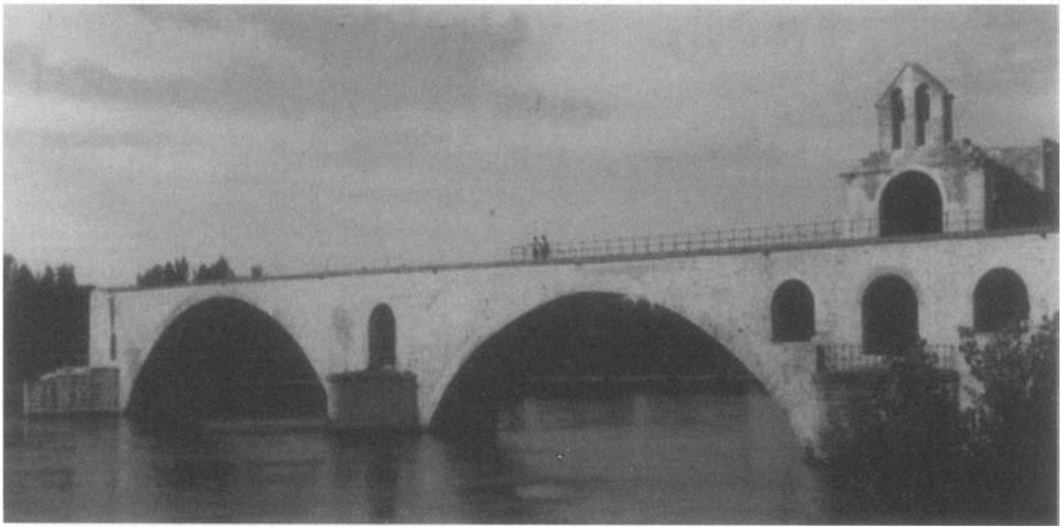

Figure 1-3. Avignon Bridge, France, partially destroyed during a flood in the 17th century

A new material for the bridge building appeared—reinforced (and later prestressed) concrete (Figure 1-5). The cost of the material was competitive with that of steel, the basic components existed almost everywhere, and because ". . . the reinforcing steel is insulated, inside concrete, no corrosion problem can occur," as quoted in the Bauschinger Memorandum published in 1887 (Tuuti 1982).

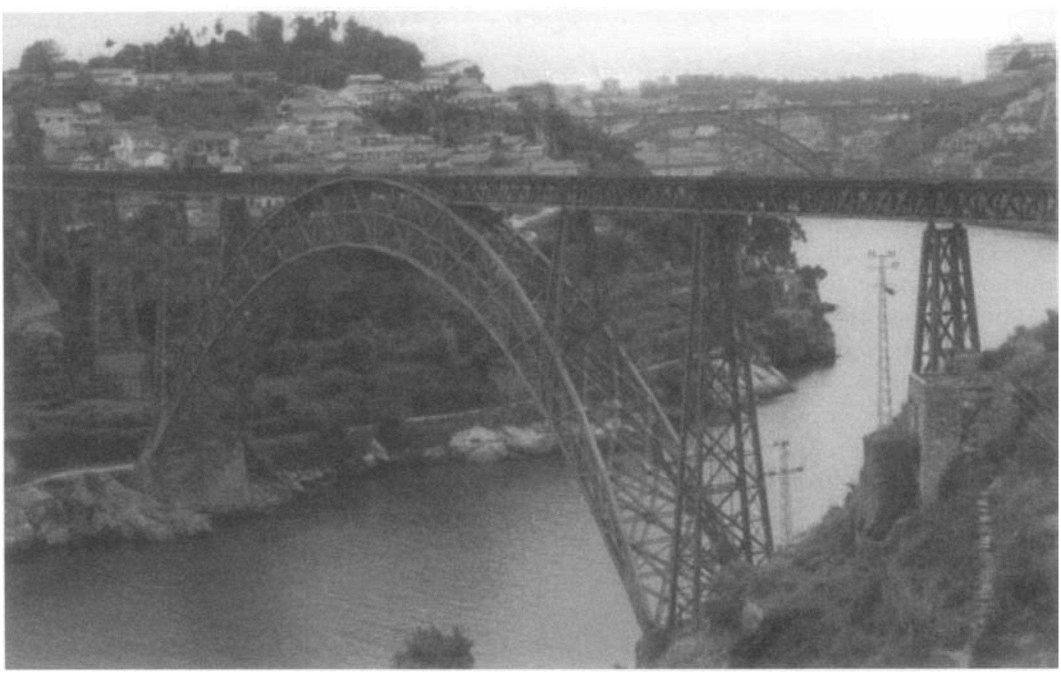

Figure 1-4. Maria Pia Railway Bridge (1877) in Porto, Portugal, built by Gustave Eiffel in wrought iron

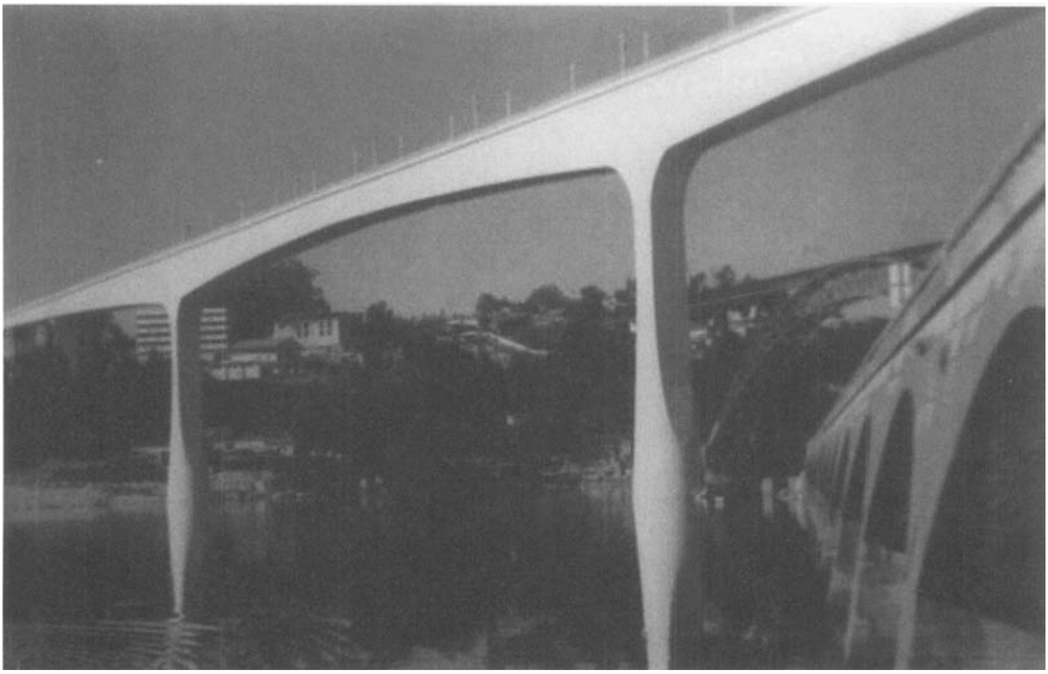

Figure 1-5. S. João Bridge in Porto, Portugal, the longest concrete box girder span for railway bridges (250 m)

Unfortunately, concrete insulation was not as good as it was initially thought to be, and environmental aggressiveness leads to corrosion of the reinforcement after some decades (Figure 1-6). A new problem arose at the end of the twentieth century, because an enormous number of concrete bridges throughout the world experienced significant durability problems and required maintenance or rehabilitation to keep the road networks functional.

The amount of money involved in bridge rehabilitation is enormous leading to a great need to implement design of new bridges in a more durability focused way and to manage economically the maintenance of existing bridges. These are the main challenges to bridge engineers in the twenty-first century.

1.2. Bridge Management Today

Bridge networks have undergone a significant change since the 1960s. In fact, it was from that period on that there was a significant increase in bridge construction in most developed countries. In those countries, more than 60% of the bridges are less than 40 years old and, among them, usually 60% to 70% of the bridges are made of concrete (Chase 1999; Baker 1999).

The construction boom in the 1960s was also associated with a move from steel to concrete as the dominant structural material, and presently there are more concrete bridges than steel, timber, and masonry bridges combined.

The importance of bridge maintenance and/or rehabilitation versus new bridge construction became a great concern of bridge authorities in the last three decades of the twentieth century. The situation, in developed countries, indicates that 20% to 30% of the total number of bridges in existence should be considered inadequate (Chase 1999; Baker 1999).

Figure 1-6. 50-year-old deteriorated quay (in foreground) and the 500-year-old well-preserved Belém Tower (in background) in Lisbon, Portugal

This is mainly due to the high structural deterioration rates that have been observed in some structures or to the lack of functionality that sometimes has occurred as a result of increasing traffic volumes or loads.

Causes of structural degradation include poor design with respect to durability, lack of quality control during construction, increasing levels of pollution, and the absence of regular inspection and maintenance actions.

As a result of the lack of long-term planning, many bridges became functionally obsolete (with significant traffic jams or reduced traffic weight limits), many years before the end of their structural life. This led to very high functional failure costs and/or extensive upgrading works. In the United States, among the bridges considered inadequate, 70% are considered so as a result of functionality and 30% for structural reasons (Chase 1999).

If a bridge failure occurs, a traffic disruption arises, but rehabilitation works also lead to traffic disruptions. The disruption of any particular bridge results in very high costs for society. It stops traffic passing over it and forces thousands of users to resort to alternative routes at extra cost and time.

Nevertheless, bridges are still frequently built according to the criteria of structural safety and minimum initial cost to achieve the proposed level of functionality. This often results in building bridges that are difficult and expensive to inspect and maintain or that are repaired quite frequently because the design was not geared toward durability.

The costs of rehabilitation of a disrupted bridge can be prohibitive, much higher than the costs of building a more durable, stronger, and larger structure from the outset. Average values between $200,000 and $300,000 dollars per bridge rehabilitated are current figures presented by bridge authorities. Budgets for keeping or upgrading existing bridges are always limited. This means that the authorities usually can deal only with a select number of the problems detected. To increase awareness of the existing problems and to help with rational maintenance decisions, bridge management systems have been developed and are being implemented throughout the world (de Brito 1998; Pickett 1999).

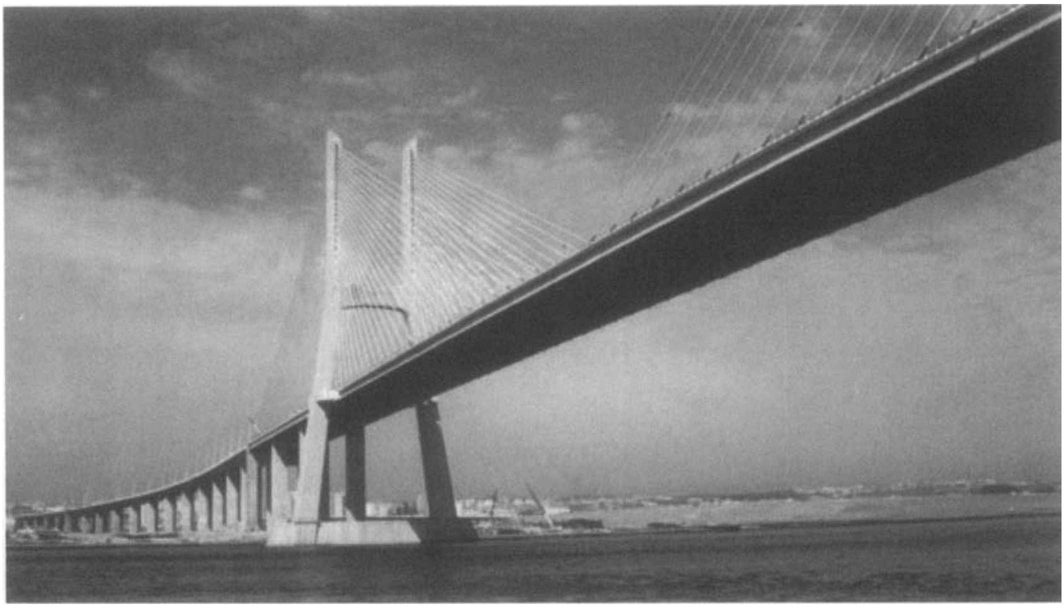

Figure 1-7. Vasco da Gama Bridge, in Lisbon, Portugal, was designed for a 120-year service life

The volume of construction to be inspected and the increasing maintenance and repair budgets led, as a first step, to a standardization of the procedures, namely, the development of inspection manuals and the implementation of databases. The experience acquired with these strategies is being used to develop new management systems in which human criteria are progressively being replaced by expert knowledge fed into computers. In these systems, the main module contains the decision criteria that will lead to the best repair decision, bearing in mind structural safety, durability, functionality, and economy (de Brito 1998).

Today, major bridges are being built for service lives of more than 100 years (Figure 1-7). For these structures, bridge authorities are beginning to understand that to obtain long service lives and reduce maintenance costs, correct actions must be implemented from the initial design and construction phases, in addition to the implementation of bridge management systems for the service stage.

This book presents the main steps to analyze bridges from design to service in terms of durability, with the goal of establishing a new integrated approach to increasing the life span of bridges.

First, a design stage is considered where a durability-oriented design must be developed, presenting the main issues to be assessed by the bridge design engineers.

Second, the main aspects that need to be considered during construction are presented, to achieve the specifications defined in the design and to avoid frequent anomalies originating in this stage, namely, considering proper quality control and monitoring plans.

Finally, a management system for the service life of the bridge is presented with a knowledge-based support that allows: (1) storage of all design information; (2) construction and in-field data; (3) standardization of reports and procedures related to inspection; and (4) assistance in maintenance and repair decision making, taking into account both structural safety and economy.

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CHAPTER 2

Service Life

2.1. Basic Concepts

The service life of a structure or an element is considered to be "the period of time, after being in use, during which all the performance properties are above accepted minimum levels, with a routine maintenance" (ASTM E632-81 1981; prENV 206-1 1999). Nevertheless, some difficulties arise when this concept is applied to the definition of a bridge life.

Bridges are built to provide a service to society consisting of the implementation of easier access, to people and goods, between two locations separated by a natural or artificial obstacle, such as a valley, a river, a road, and so forth.

The benefits of this service can be estimated by taking into account the reduction of time that users enjoy, crossing the bridge, instead of using the alternate route, which is usually longer and more time-consuming. The economic value of this service can be estimated considering the traffic on the bridge, multiplied by the average number of people transported in each vehicle, by the time saved by using the crossing and by the value of the country's gross national product (GNP) per labor hour. It is this economic value that justifies the imposition of bridge tolls by private companies, making it a sustainable business.

Bridges are designed to provide a service in terms of the amount of crossing traffic and the maximum allowable crossing load, for a certain period, which is called the bridge service life. At the end of this period, the traffic benefits of the bridge should be higher than the costs associated with its stages of design, construction, and service life.

Both its physical deterioration and its functional obsolescence control the duration of the life of a bridge. If no important structural degradation problem occurs, service life ends when the benefits obtained from the operation are exceeded by the functional, construction, and maintenance costs.

2.2. Bridge Functional Life

The bridge functional life is mainly associated with the traffic volume at an average speed that it can withstand, which is mainly associated with the existing number of lanes or with the width of the deck. For most bridges, this problem is integrated with the road where the bridge belongs (if both have the same width), as the functional life of the road is the same as that of the bridge.

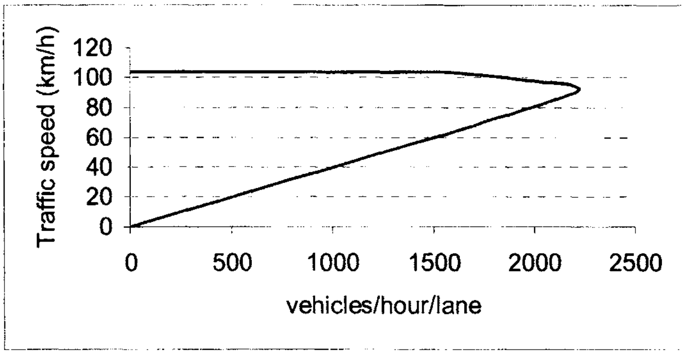


Figure 2-1. Evolution of speed versus traffic volume for an expressway (2 × 2 lanes)

The estimation of the end of the functional life requires specific prediction studies that should be developed during the planning stage. With bridges, functional obsolescence is mainly related to restrictions on traffic volume (restrictions on traffic weight are more related to its structural life), whose evolution can be estimated from statistical analysis based on traffic measurements. Using relations similar to those shown in Figure 2-1 (TRB 1995), between traffic and average speed, the number of lanes can be defined for a certain period, which is known as the target functional life.

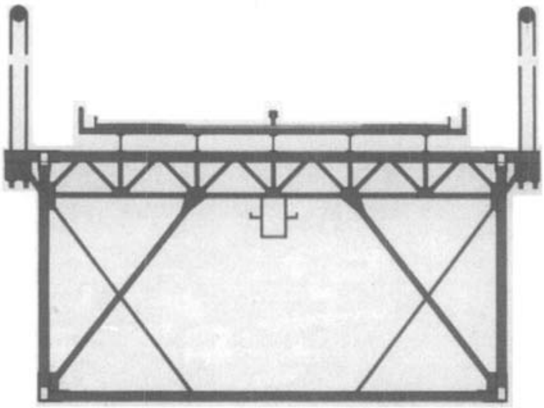
When traffic volume increases, the average speed decreases until it reaches a maximum saturation value (typically around 2,200 vehicles/hour/per lane for an expressway with 2 × 2 lanes, with a barrier wall between both directions; this volume is about 10% of the daily average traffic volume). After that saturation value, the speed and the volume of traffic decrease abruptly.

This reduction in the design traffic speed and volume is a decrease in the bridge's benefits. When the yearly benefits are negative (the bridge crossing takes more time than the alternate route) or are lower than the maintenance plus the functional failure costs, the functional life theoretically approaches its end, as small functional or economical profits are obtained.

Some measures can be considered at the design stage to delay this functional obsolescence of the bridge. Such measures have been adopted mainly for major urban bridges, where traffic increase has been quite high:

1. The deck can be designed with a total width that allows the introduction of an additional central lane where the traffic direction changes according to rush hours. The new lane is obtained with a reduction of each existing lane width. This solution implies the use of traffic lights in the lanes, the removal of the central separation barrier-wall, and a reduction of the average speed. This situation is illustrated in Figure 2-2 for the Lisbon 25th of April Bridge, a suspension bridge, where it was adopted in an intermediate stage before the deck was widened (REFER 1999).

2. A similar measure, but one that allows for the introduction of an additional lane in each direction. This usually leads to a reduction of traffic speed. This situation is illustrated in Figure 2-3 for the Vasco da Gama Bridge, where the initial deck for 2 × 3 lanes has a width (30 m) allowing for 2 × 4 lanes, when traffic volume demands.

a.


b.


Figure 2-2. Reverse central lane at 25th of April Bridge (Bridge over the Tagus) in Lisbon, Portugal, 1990: **a.**, initial cross-section; **b.**, view of the deck after 5 lanes have been installed

The reduction of the lane's width or the placement of barrier walls, lead to a reduction in traffic volume (R) quantified by the factors in Table 2-1 (TRB 1995, Branco 1995).

If no preventive measures were considered at the design stage, the typical solutions to increase the functional life are:

1. Design a new bridge, possibly using the existing bridge for one of the traffic directions (a solution depending on the existing bridge's structural conditions).

2. Increase the width of the bridge deck, specifically if the columns and foundations bear the new situation with an economically reasonable strengthening. This situation is also exemplified in Figure 2-4 for the Lisbon 25th of April Bridge, where it was adopted in the second stage (REFER 1999). An additional cable was placed to strengthen the bridge in order to support the deck widening and the railway inside the deck truss.

Figure 2-3. Vasco da Gama Bridge in Lisbon, Portugal, with 2 × 3 lanes, able to support 2 × 4 lanes

Functional life can also be affected by the degradation of the circulation conditions, which can be associated, for example, with the deterioration of the pavement, or to road deformations next to the abutments. These anomalies also must be repaired within the maintenance program of the bridge.

2.3. Bridge Structural Life

2.3.1. General

The structural life of a bridge is associated with its safety and serviceability conditions, namely, it is related to situations such as collapse, deformation, cracking, and so forth. These conditions depend mainly on the evolution of actions and materials during the bridge life.

Design codes usually define, through the actions and material's resistance design values, the safety levels to be considered (associated with probabilities of failure) (ENV 1991-1 1999). In order to have safety (S), the material's resistance (R) must be higher than the action effects (A), or, considering probabilistic distributions (Figure 2-5):

$$p\ (S) = p\ (R - A) > 0 \tag{2-1}$$

Table 2-1. Reduction in traffic volume with lane width and lateral barriers

Lane width (m)	Reduction (R)	Lateral space (m)	Reduction (R)
3.6	1	1.8	1
3.3	0.97	1.2	0.99
3	0.91	0.6	0.97
2.7	0.81	0	0.9

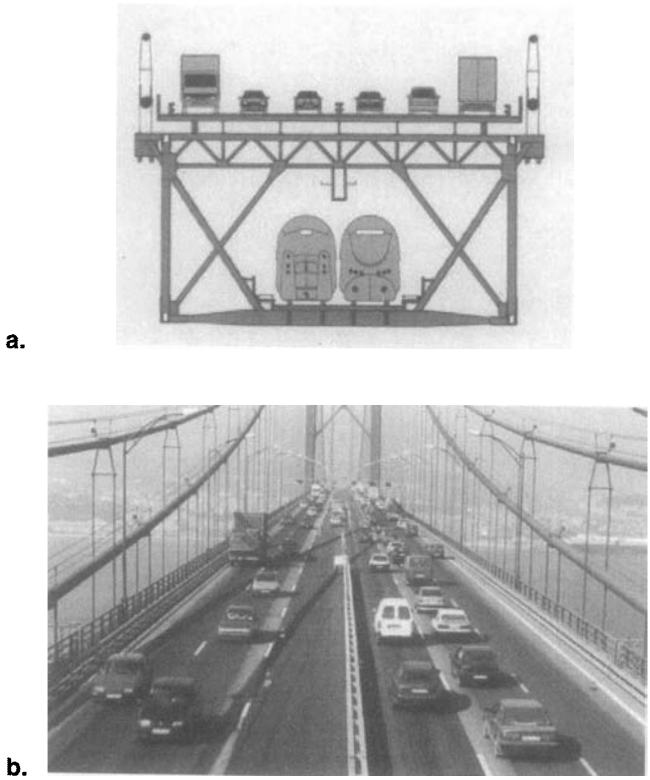


Figure 2-4. Deck widening at 25th of April Bridge (Bridge over the Tagus) in Lisbon, Portugal: **a.**, widened cross section; **b.**, view of the deck after 6 lanes were installed, 1998

The safety analysis is usually based on characteristic values for the actions (A_k) and for the construction materials properties (R_k).

The action characteristic values are defined in codes considering a statistical distribution and a reference period (usually associated with a structure life period of about 50 to 60 years), or reference values within that period, A_k being a value with a probability (usually 95%) of not being exceeded in that period.

As for the material's resistance properties, their characteristic values, R_k are obtained from statistical distributions measured in situ, R_k being a value with a probability (usually 95%) of being exceeded.

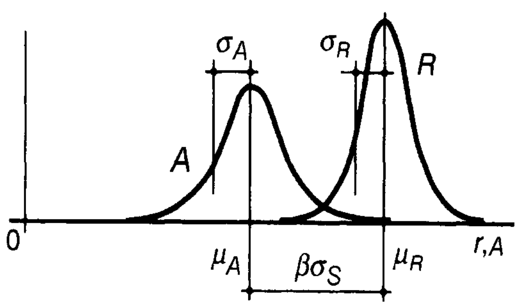


Figure 2-5. Statistical distributions for actions A and material strengths R

At the design level, these characteristic values are changed to design values considering additional safety coefficients, representing the imprecision of the models of analysis or the uncertainties connected to the construction procedures.

The design actions, A_d are then defined by their characteristic values, A_k multiplied by a safety coefficient, γ_a related to the quantification of the action effects (γ_a usually varies between 1.20 and 1.35 for dead loads and 1.5 for live loads). The design strength, R_d, is defined by the characteristic properties of the materials, R_k, divided by a safety coefficient, γ_m, related to the in situ variability of the properties (γ_m is usually considered to be 1.5 for concrete and 1.15 for steel).

Safety analysis is then based on the comparison between the design actions effects, A_d, and the design strength of the structure, R_d, to obtain during the bridge life the safety relation (Figure 2-5).

$$A_d < R_d \tag{2-2}$$

This allows the designers to define the safety level of the bridge, based on the adopted actions, material characteristics, and structure geometry. For a 50-year service life, the probability of failure, P_f, is frequently smaller than 7×10^{-5} (reliability index $\beta = 3.8$) (ENV 1991-1 1999; Litzner 1999).

$$P_f = P\,(S < 0) \simeq \Phi\,(-\,\beta\,(t)) \tag{2-3}$$

In equation 2-3, Φ is the normal distribution with an average value $\mu_S = \mu_R - \mu_A$ and a standard deviation $\sigma_S = (\sigma_R^{\,2} + \sigma_A^{\,2})^{0.5}$.

Nevertheless, that safety level is based on the design hypotheses and is referred to the bridge opening day, because as soon as the bridge life begins, all those parameters start changing.

2.3.2. Evolution of Loads

The characteristic value of the actions, A_k are defined in codes based on a statistical analysis usually considering the values associated with a reference period of around 50 years, associated with the bridge target life.

During the bridge life, the characteristic value of the actions or of the action effects may vary, leading to a decrease of the bridge safety. Examples of these situations can frequently be found in:

- successive repavement or insulation operations that lead to an increase in permanent loads;
- axle load increase due to changes in vehicles characteristics;
- foundation settlement caused by a change in geological conditions.

Besides these changes, if, during the bridge life, a rehabilitation design is performed, the quantification of the characteristic actions varies and this must be considered when bridge safety is reanalyzed.

As time goes by, the remaining bridge life is shorter, and for the same probability of occurrence, the characteristic values of the actions decrease, specifically for time-variable actions. This is important for rehabilitation design where the definition of a shorter life for

Table 2-2. Changes in characteristic values versus return period for wind

Reference period (years)	Correction factor
100	1.04
50	1
20	0.95
10	0.9

the rehabilitated structure leads to a reduction of the characteristic values of the loads. Corrective values are referred in codes for actions such as wind (Table 2-2) and seismic loads (ENV 1991-2-4 1999; prENV 1998-1-4 1995).

2.3.3. Evolution of Materials

The evolution of the material properties in concrete bridges during the bridge life has two types of components; the first is associated with the evolution of the strength of the materials, and the second is related to the changes in the geometry of the structural elements or materials deterioration.

Considering the evolution of the strength of the basic materials, the steel strength is practically uniform throughout the bridge life unless an accident such as a fire occurs. Nevertheless, problems may also arise with the fatigue strength of steel, specifically with prestressed steel, leading to a design with a conservative safety analysis and quality control during construction.

The strength of concrete increases with time from the design reference value (at 28 days after concreting) until obtaining higher values of more than 35% after 1 year and 45% at the end of the service life. This evolution increases the design safety considered at design level.

Besides this effect, the in situ concrete strength is frequently higher than the specified design value. This leads to an additional bridge safety that must be considered in a rehabilitation design.

These evolutionary aspects can be considered in bridge rehabilitation, through the use of a more realistic safety factor, γ_m, to quantify the strength design values. In fact, if a detailed inspection is performed and reliable characteristic values of the materials properties are obtained from in situ measurements, the coefficient γ_m can be reduced (codes suggest a reduction of γ_m that can go from 1.5 to 1.2 in terms of concrete properties (prENV 1998-1-4 1995).

Considering the durability problems associated with environmental actions, concrete deterioration and reinforcement corrosion may occur, with cross section reduction, leading to a decrease in the geometric dimensions and materials properties, and thereby to a decrease in the structure safety throughout the bridge service life.

Recent codes present design specifications for materials and construction procedures theoretically leading to a structural life of around 50 years, considering the concrete degradation effects (prNV206-1 1999).

Nevertheless, these recommendations are still a bit far from the structural design procedures. In fact, in trying to make a parallel analysis for durability analysis, the "actions" are a set of typical environments and the "material resistance" is a set of construction procedures and material properties (cover, concrete components and characteristics). These are defined to ensure that the main properties of the materials are kept along the bridge life so structural safety is not reduced by durability.

In the present situation, a safety analysis for durability, based on the environmental conditions and materials properties, can hardly be implemented, and the current design procedures consist solely of performing a structural safety analysis and adopting the code specifications for durability.

2.4. Service Life of Bridge Components

In addition to the concrete structural elements, bridges are made of several other components. Among these, some are related to structural behavior such as:

- bearings (Figure 2-6)
- seismic dampers
- expansion joints
- cables

Other components are nonstructural and are related mainly to the functionality of the bridge, such as:

- pavement
- earth fill protection
- handrails and sidewalks (Figure 2-7)
- barrier walls or railings
- drainage equipment
- electrical equipment (light, monitoring)

Figure 2-6. Deterioration of a bearing

Figure 2-7. Deterioration of handrails in a Lisbon viaduct, Portugal

Most of this equipment has a shorter life than the service life defined for the bridge. This implies that it will have to be replaced or repaired several times during the bridge life. Reference values are indicated in codes (CHBDC 1995):

• asphalt concrete pavement	15–20 years
• rubberized waterproofing membrane	25–30 years
• waterproofing concrete overlay	30 years
• steel coating systems	10–20 years
• expansion joints	15–30 years
• expansion joint seals	5–15 years
• bearings	25–40 years

The criteria for the definition of the service life for each of these equipments are variable and should be defined by the maker, indicating the pathologies that will lead to repair and/or maintenance measures or to their substitution (end of service life).

Frequently the estimation of the service life of the equipment is based on past experience with its use. To evaluate its service life, it is always convenient to ask the producer about the older installed equipment and preferably try to inspect it in situ.

2.5. Service Life, Safety, and Durability

There are several service lives in a single bridge structure. Nevertheless, when bridge owners want to build a bridge, they need to specify a value for the bridge service life, to be considered in design.

Codes, following the international tendencies on this matter, usually present reference values for structures service lives (ENV 1991-1 1999):

- temporary structures—1 to 5 years;
- structural components to be replaced—25 years;
- current bridges—50 years;
- major bridges—100 years.

For functionality reasons, this means that the lanes of a current or major bridge should bear road traffic for 50 or 100 years, respectively. If the bridge is designed for that life and the traffic saturation value is reached before that limit, then another bridge could be built to maintain the traffic on the first bridge, because it is still safe from a structural point of view.

If the owner wants to build a major bridge with an anticipated life of 100 years life (today, with correct maintenance, even medium-sized concrete bridges may have that life expectancy), the situation typically faced by the designers is rather complex.

In fact, for the analysis of the structural safety of the bridge, the designers have the reference code actions, usually defined for a reference period of 50 years. They need to adopt durability measures for 100 years, but the code indications are usually referred for 50 years (cover reinforcement, diffusion, etc.). They need to consider for that bridge bearings and other equipment capable of lasting at most 25 years (designed for actions defined for 50 years). When service life is increased beyond the current 50 years, the study of major bridges requires that safety be reconsidered to incorporate coherence into the design.

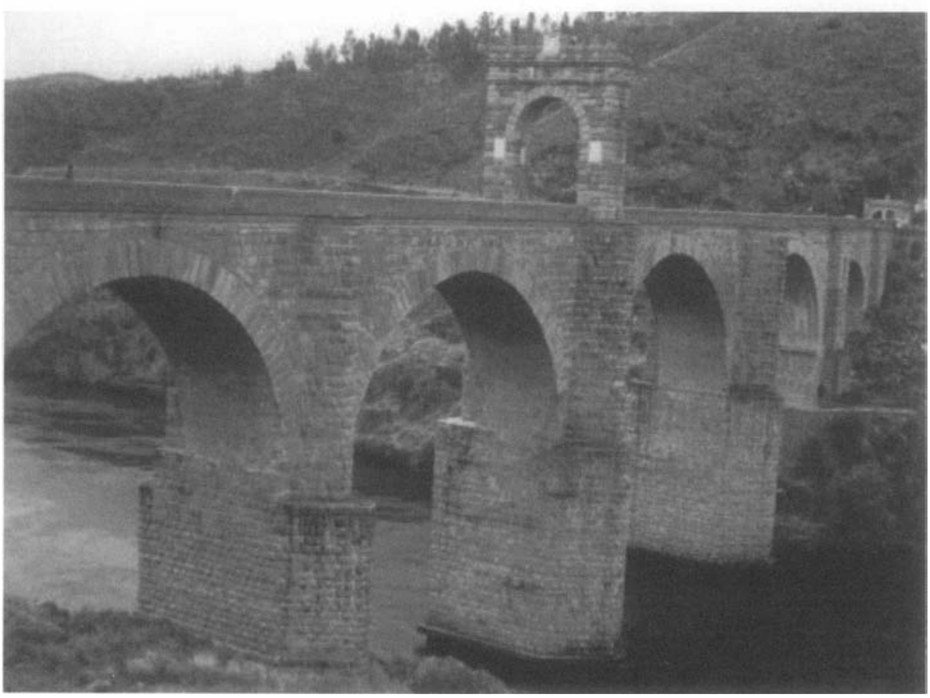


Figure 2-8. 2,000-year-old Roman Alcântara Bridge over the Tagus, Spain; preserved as a monument

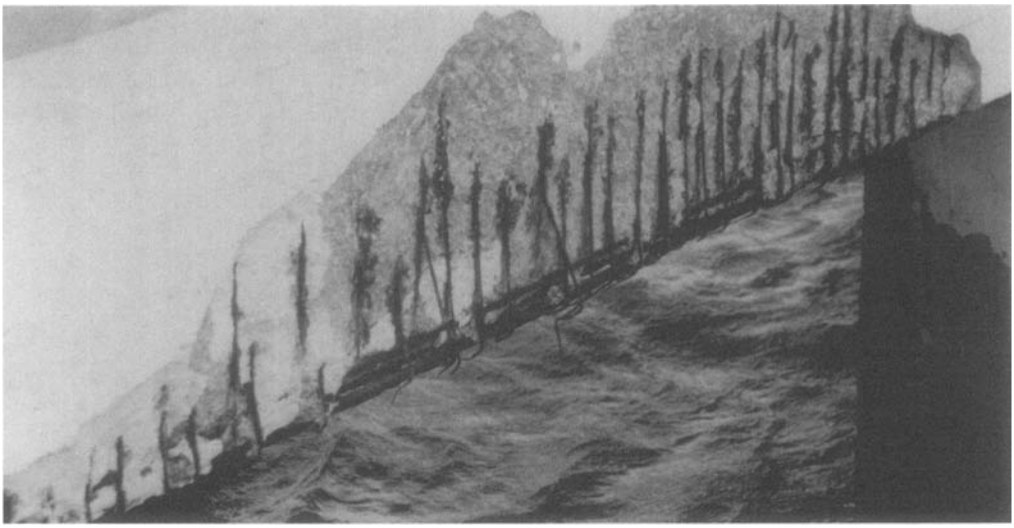


Figure 2-9. Bridge with reinforcement highly corroded, in which rehabilitation is not economically feasible, coastal area, Portugal

In fact, the service life concept must be integrated with design more efficiently. For a 100-year bridge, the durability analysis and structural safety must be updated from code values to that reference period. Bridge components such as bearings can be designed for code-reduced actions, related to their real life, assuming that they will be replaced.

The updating of action design values for structural analysis can be performed, based on their statistical distributions and considering, for time-variable actions, the new reference period (Borges 1985). When related to durability specifications, the updating is much more difficult and must take into consideration the degradation models and special procedures, such as those presented in Chapter 4.

2.6. End of Life and Decision Making

When the benefits of a bridge are lower than its costs and rehabilitation is not economically feasible, its life ends and traffic is stopped. In this situation, two case scenarios are possible:

1. Historical bridges can be preserved because of their landmark status, but the cost value must be assumed by society since keeping a bridge standing will require maintenance costs. In this case, the bridge loses its functional value as a crossing structure and acquires a new value as a monument (Figure 2-8).
2. The bridge is of no use and is abandoned or demolished (Figure 2-9). In this case, it is important that the concept of deconstruction be considered by the designers from the design stage, including opportunities for recycling materials.

Usually, the main problem of this process is associated with the in situ demolition of the bridge, because the concrete elements are very large and must be cut into smaller pieces to be carried to the recycling plant or to the dump. Frequently the deck must be supported by scaffolding during demolition, and mechanical, thermic, or chemical methods are then applied to cut the structure.

The recycling of bridges has great advantages because concrete bridges are built with only a fairly uniform composite material made of steel and concrete. The bridge elements can be cut into pieces in recycling plants. Steel is separated from concrete with strong magnets and can then be melted to make new steel. The concrete is cut into small aggregates that can be reused for several objectives, such as:

- aggregates for roads sub-base
- aggregates for new recycled concrete
- filling material for earth works

CHAPTER 3

Testing and Monitoring

At the design stage, bridge engineers estimate the behavior of a bridge based on theoretical models that analyze its structural safety and durability.

During the construction stage, the design specifications are implemented, namely in terms of geometry and materials properties. During this phase, several tests must be performed to check the compliance between the design and the effective properties of the materials.

When the bridge construction is finished, acceptance load tests may be performed, within the conformity control, to check agreement between the theoretical models at the design stage and the actual global structural response of the bridge.

During a bridge's service life, mainly in major bridges, several structural parameters are monitored to check the real behavior of the structure and to allow a quick reliability analysis if an accident occurs. Durability parameters are also monitored in this phase to obtain their effective evolution, by comparing them with design theoretical models, to allow for the evaluation of the real degradation of the bridge.

Bridge experimental testing and monitoring (Figure 3-1) is an increasingly popular activity, since bridge engineers and authorities became aware of their importance for obtaining a good quality construction control and for extending the bridge life, along with the concurrent cost saving associated with repairs necessitated by unexpected problems.

In this chapter, the main procedures for material testing and bridge monitoring are presented, with emphasis on the evaluation of its mechanical and durability properties. These tests, whose applications will be described in subsequent chapters, are related to the bridge's construction phase and its service life inspection and were selected from the tests that are more frequently used by bridge authorities and contractors internationally.

Besides their international applications, most of the tests described have particular technical specifications in North America (ASTM, AASHTO) and in Europe (RILEM, ENV, BS, DIN), where their details can be found. In this chapter the tests are presented only in a simplified way, so that bridge engineers can be aware of their general procedures and can select the most effective tests for a specific target, and understand their results.

3.1. Materials Lab Testing

Materials lab testing considers the tests performed in samples of the materials that are applied in the structure to check or to quantify their properties. These tests are first

Figure 3-1. Bridge load testing

performed at an initial stage of the work to define the properties of the materials. Afterwards, during construction, the tests are repeated within the production and conformity control of the work.

The lab tests should be complemented with in situ tests, these performed on the structure or on samples taken from it. This allows a check of changes in the material's properties, from the lab situation, when they are applied in the structure.

3.1.1. Mechanical Properties

Concrete Strength

Compression strength (f_{ck}) is obtained from the compression testing of samples (cylinders or cubes) at the age of 28 days, molded during concreting operations and tested according to technical specifications (BS 1881: Part 116; prENV206-1 1999).

The characteristic compression strength of concrete f_{ck} is obtained from the test results of several specimens, using the statistical equation (average f_{cm} and standard deviation σ):

$$f_{ck} = f_{cm} - 1.64\ \sigma \tag{3-1}$$

Within a production control the number of tests per batch is reduced and usually, in each group of three specimens, it is only checked so that the average results (f_m) and minimum value (f_{min}) verify the following relationships:

$$f_m > f_{ck} + 5\ \text{(MPa)} \qquad \text{and} \qquad f_{min} > f_{ck} - 1\ \text{(MPa)} \tag{3-2}$$

In addition to testing at age 28 days, the compression test is usually performed at earlier ages (3, 7, and 15 days) in order to obtain an evolution path, to allow earlier construc-

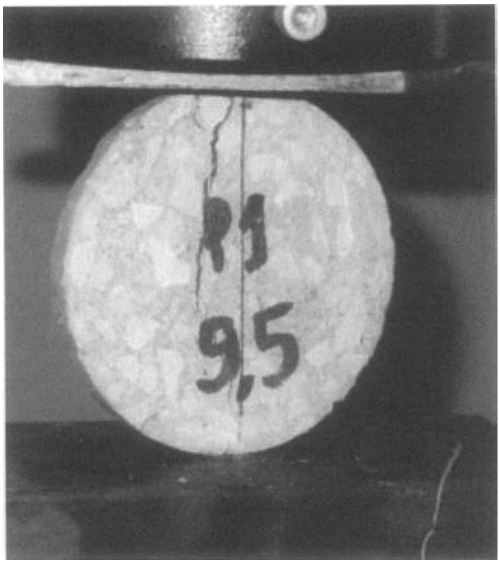


Figure 3-2. Split test

tion operations like formwork removal or prestressing. The relationship between hydration temperatures and strength evolution has also been used to estimate the concrete strength based on temperature measurements (*see* Maturity).

Tensile Strength (f_{ctk}) is usually obtained from the tensile split test (Brazilian test; see Figure 3-2) that can be performed according to technical specifications (prENV206-1 1999). The characteristic tensile strength is obtained from the test results in a way similar to the compression strength.

Frequently, the tensile strength is also estimated from the correlation between compression and tensile strength, presented in concrete Codes.

Other Properties of Concrete

Young modulus (E) is obtained in the lab by testing concrete samples (cylindrical or prismatic) molded during concreting operations. The specimens are tested, according to technical specifications, by increasing elastic compression and simultaneously measuring the axial strains ε (with strain gauges) and the compression stress f. The Young modulus is obtained from the relationship $E = f/\varepsilon$ (Santos 1997).

The characteristics of the Young modulus are particularly important to the behavior of the structure during the early days, specifically in terms of deformations associated with self-weight or prestressing operations. That is the reason why the determination of the Young modulus should also be performed using concrete samples of several ages to obtain its evolution with time.

Creep ($\varphi(t)$). The definition of creep evolution is important for confirmation of the models adopted at the design stage, when usually only the code recommendations are considered. The experimental results allow the adoption of more reliable camber geometry, namely preventing excessive deformations in central spans during construction.

Creep strains are measured in prismatic specimens that are tested under controlled moisture and temperature conditions and are subjected to a constant compression stress for several months. The stress level should be similar to that expected in the bridge main sections.

In order to keep the stress level constant, a system must be considered in the testing apparatus, to compensate for axial deformations of the specimen. The creep effect $\varphi(t)$ at a certain age is defined as the ratio between the additional deformation at that age and the initial elastic deformation minus one (Santos 1997).

These tests are time-consuming, and therefore they should begin at the planning phase of the construction, to obtain preliminary information about concrete creep values. Due to the characteristics of the creep phenomenon, these tests should also be performed in situ.

Shrinkage (ε_s). Similar to creep, the evolution of shrinkage is important for the confirmation of the models adopted at the design stage at which only the code recommendations are usually considered. These results allow for the adoption of a more reliable analysis of the longitudinal deck deformations during construction, which is important for bearings and joints placement.

Shrinkage strains are measured in prismatic samples that are tested under free deformation in an environment of controlled moisture and temperature (Santos et al. 1997). The deformation level defines the shrinkage effect at a certain age.

As these tests also take a long time, they should begin at the planning phase of the construction, in order to have information about shrinkage values as soon as possible. Due to the characteristics of shrinkage, these tests should also be performed in situ.

Heat of Hydration (q_v). In order to study the heat of hydration characteristics of concrete, a set of procedures has been developed, using experimental and numerical analysis (Branco 1992). Two concrete prismatic specimens (with a depth similar to structural element thickness) are cast in forms, insulated with thick internal sheets of expanded polystyrene in a quasi-adiabatic condition and placed in a shaded area. Internal temperature and ambient temperatures are measured hourly for 10 days. In one of the specimens, the lateral insulation is removed after the time expected for formwork removal, simulating this work phase.

Parallel to the experiments, a numerical analysis of the test situation is performed using the Fourier equation of flow, relating the temperature T in each point (x, y, z) to time t:

$$\rho \cdot c \cdot dT/dt = k\,(\delta^2 T/\delta x^2 + \delta^2 T/\delta y^2 + \delta^2 T/\delta z^2) + q_v \qquad (3\text{-}3)$$

where

ρ = concrete density (kg/m^3);

c = specific heat (J/(kg °C));

k = thermal conductivity (W/(m °C)).

q_v is the rate of heat of hydration, generated per unit of volume, obtained by:

$$q_v = (C\,E\,b\,n) \cdot ((t_e)^{-n-1}) \cdot \exp\,(-b \cdot (t_e)^{-n}) \cdot 2^{0.10\,(T(\tau)\,-\,Tr)} \qquad (3\text{-}4)$$

C = cement mass per unit of concrete volume (kg/m^3);

E, b, n = constants for heat of hydration simulation, depending on concrete mix.

t_e is an equivalent time, function of the temperature of the process T, related in each time interval to the difference to a reference temperature T_r:

$$t_e\,(t) = 1/3600 \cdot \Sigma_0^{\,t} \cdot 2^{0.10\,(T(\tau)\,-\,Tr)} \cdot \Delta\,\tau \qquad (3\text{-}5)$$

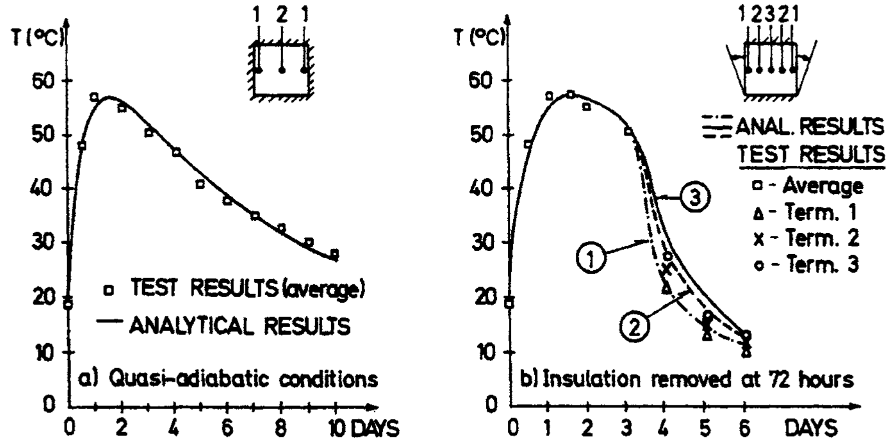


Figure 3-3. Heat of hydration tests for the S. João Bridge in Porto, Portugal

The Fourier equation for the tested cubes is solved by a finite element analysis and considers boundary conditions associated with surface heat exchanges within the environment.

The comparison of the numerical and experimental temperatures allows the characterization of the heat of hydration (Figure 3-3). Based on these values, simulations of the formwork release operations can be studied, thereby reducing the potential effects of differential temperatures, namely heat of hydration cracking (Branco 1992; Branco 1993).

Maturity. Concrete strength development is a function of time and temperature, and for predetermined mix and curing conditions that may be related to the maturity. Maturity is the product of time with the temperature above a reference value and is obtained from:

$$M(t) = \Sigma\ (T_a - T_o)\ \Delta t \qquad (3\text{-}6)$$

where

$M(t)$ = maturity (degree-hours)

Δt = time interval (hours)

T_a = average temperature during Δt

T_o = reference temperature (usually -10 °C)

To correlate the concrete maturity in a structure with the concrete strength obtained from cube (or cylinder) compression tests, the cubes must be subjected to the same temperature evolution (Bungey 1989).

This correlation is performed with the initial concreting of a reference structural element in which the temperatures are monitored and cubes are simultaneously placed inside a heated water tank with the same temperatures as the structure.

For other similar structural elements, concreted afterwards, the measurement of their maturity allows, with the computed correlation, the estimation of the concrete strength. This technique has been used to reduce the formwork time release in precast elements or to reduce the time needed before applying prestress.

Steel Properties

Reinforcement tests. The reinforcement steel strength tests are performed in the lab, with the classical tensile test applied to steel specimens. The tensile test should indicate the yield and the ultimate stress and the ductility of the steel. This test can also be used to obtain the steel's Young modulus, but due to the uniformity of this parameter, it is not usually considered.

Prestressing tests. The properties of prestressing steel and systems are checked considering several lab tests, namely (FIP 1974; FIP 1981):

1. Mechanical properties of prestress cables (strength, Young modulus, ductility, alternate bending, torsion, fatigue, etc.);
2. Static and dynamic resistance of cable-anchorage systems;
3. Fatigue behavior of cable-anchorage systems.

3.1.2. Durability Properties

Water Absorption

Water absorption is an important factor in characterizing the durability properties of concrete as it is related to the penetration of water in concrete from the surface, which is associated with several degradation problems. It can be measured with the following tests (Concrete Society 1987; BS1881-Part 122).

Capillary Absorption Test. Specimens are placed with one face inside up to 5 mm of water for 4 hours (Figure 3-4). With time, the wet zone increases and the correspondent absorption coefficient can be determined from:

$$I = a\,t^{0.5} \tag{3-7}$$

where

I = water absorption per unit area (mm^3/mm^2)

a = absorption coefficient (mm/min$^{0.5}$)

t = time (minutes)

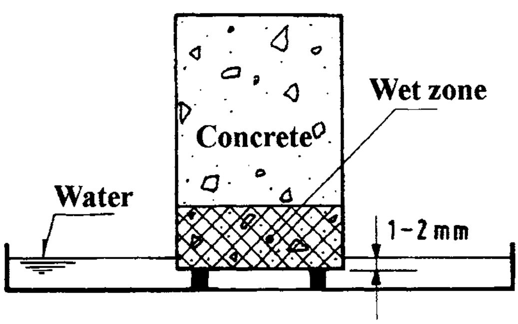


Figure 3-4. Capillary absorption test

Table 3-1. Concrete classification for capillary absorption test

Concrete quality	a (mm/min^{0,5})	Water height (mm)
High	<0.1	<10
Average	0.1–0.2	10–20
Low	>0.2	>20

The quality of concrete can be related with the absorption coefficient, or the water height obtained from the test, using the values of Table 3-1 (Browne 1991).

Immersion Test. In this test specimens (usually with h = Φ = 75 mm) are initially dried for a period of 72 h at 105 °C, after which they are cooled and weighed (M_1). They are then placed inside water for 30 min and weighted again (M_2) (Figure 3-5). The water absorption A is measured by:

$$A = (M_2 - M_1)/M_1 \tag{3-8}$$

For this test the following concrete classification is referred (Table 3-2) (Concrete Society 1987).

Water Permeability

The water permeability characteristics are obtained in a steady-state situation considering a specimen subjected to water pressure on one face and measuring the water volume that passes through the specimen (Figure 3-6). The water permeability coefficient k is obtained, using D'Arcy law, from the expression:

$$k = Q\mathrm{l}/\mathrm{Ah} \tag{3-9}$$

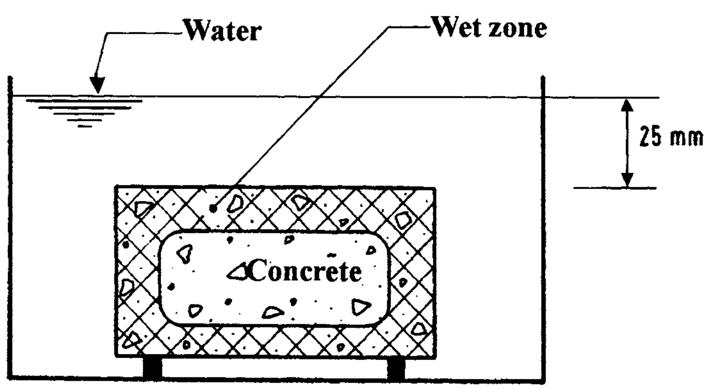


Figure 3-5. Immersion test

Table 3-2. Concrete classification for immersion test

Concrete quality	A (%)
High	<3.0
Average	3.0–4.0
Low	>4.0

where

k = permeability coefficient (m/s)

Q = water volume passing through (m^3/s)

A = specimen area (m^2)

l = thickness of specimen (m)

h = pressure, water height (m)

In concrete the water permeability coefficient usually varies between 10^{-16} and 10^{-10} m/s. Concrete quality can also be estimated based on its permeability according to Table 3-3 (CEB 1989).

Water permeability can also be obtained in a variable situation, using a similar test but measuring the depth of water penetration and under a pressure of 0.5 Mpa after 72 hours (DIN 1048 1978; prENV206-1 1999). Using this test, the recommended maximum water depth penetrations are:

- impermeable concrete <50 mm
- resistant to freeze-thaw <50 mm
- resistant to chemical attack <30 mm
- resistant to saline environment <50 mm

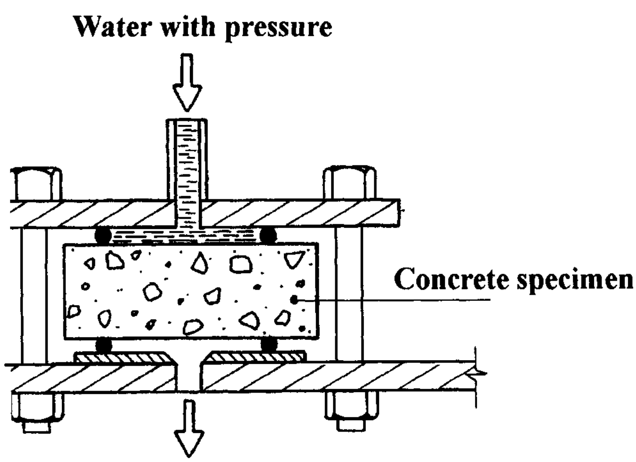


Figure 3-6. Water permeability test

Table 3-3. Concrete classification for water permeability

Concrete quality	Permeability	k (m/s)
High	Low	$<10^{-12}$
Average	Medium	10^{-12}–10^{-10}
Low	High	$>10^{-10}$

Gas Permeability

Gas permeability tests are frequently used as a substitute for water permeability tests, because they allow a quicker determination of concrete permeability (RILEM 1999). Usually oxygen is used to measure permeability in a concrete specimen ($\Phi = 150$ mm; $h = 50$ mm) by subjecting one of the faces to gas pressure and measuring the amount of gas that passes through the specimen. The oxygen permeability K_o is measured by

$$K_o = 1.14 \times 10^{-4}\, Q p_a/(p^2 - p_a^2) \tag{3-10}$$

where

K_o = oxygen permeability (m^2)

Q = volume of gas passing through (m^3/s)

p = initial pressure (N/m^2)

p_a = outside pressure (N/m^2)

Permeability oxygen coefficients between 10^{-14} and 10^{-19} are typical values for concrete strengths between 15 and 55 MPa.

Chloride Diffusion

The quantification of chloride diffusion through the concrete depth is important to estimate the service life of the structure and to simultaneously analyze the quality of concrete. There are several tests to perform this analysis (Luping 1992; Costa 1997; Gonçalves 1999).

Chloride Diffusion Tests

Diffusion cell is a stationary test in which a concrete sample is placed between two chambers, one chamber is saturated with chlorides while the second chamber contains no chlorides (Figure 3-7). By measuring the chloride concentration in both chambers after a period of time, the diffusion coefficient D_c can be approximated by using Fick's first law and considering $C_1 > C_2$, by:

$$D_c = V e\, C_2/(A\, C_1\, (t - t_0)) \tag{3-11}$$

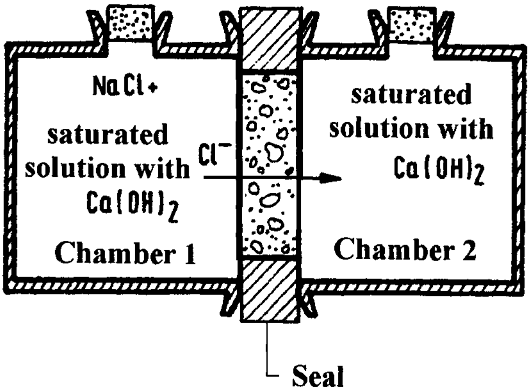


Figure 3-7. Chloride diffusion cell

where

D_c = diffusion coefficient (m^2/s)

V = solution volume of chamber 2 (m^3)

e = thickness of specimen (m)

A = area of specimen (m^2)

C_1 = chloride concentration in chamber 1 (mole)

C_2 = chloride concentration in chamber 2 (mole)

Immersion method. A similar test can be performed by immersing the samples in chloride-saturated water and measuring the chloride content at different depths after a period of time. The diffusion coefficient D_c is obtained using Fick's second law and the existing chloride profile of the specimen.

$$C(x, t) = C_s\,(1 - \text{erf}\,(x/(2 \cdot (D_c\, t)^{0.5}))) \tag{3-12}$$

or using a parabolic approximation

$$C(x, t) = C_s\,(1 - x/(2 \cdot (3\, D_c\, t)^{0.5}))^2 \tag{3-13}$$

where

D_c = diffusion coefficient (cm^2/s)

$C(x, t)$ = chloride concentration at distance x (kg/m^3)

C_s = chloride concentration at surface (kg/m^3)

x = distance from surface (cm)

t = duration of immersion (s)

Both of these tests take a long time to give significant results. Specimens usually need to be tested over a span of several months.

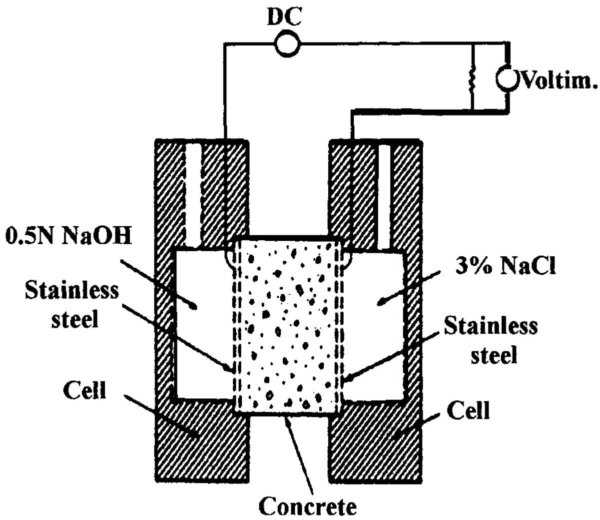


Figure 3-8. AASHTO T277 test

Chloride Migration Tests

In order to obtain quicker results, several migration tests have been developed in which chloride diffusion is accelerated by applying an electrical potential between the two chambers of the diffusion cell (Figure 3-8).

Among these tests, the test most often used is one in which a potential of 60 volts is applied between both chambers and, after 6 hours, the intensity is measured in coulombs (AASHTO T277 1983).

The quality of concrete can than be classified according to Table 3-4.

The correlation between the migration method and the diffusion method is not reliable. The migration tests are more efficient to calibrate the quality of concrete than to quantify the diffusion coefficient. If they are both used, the uniformity of the production can be checked periodically by migration tests, associated with an initial diffusion coefficient determined with a diffusion test.

Gas Diffusion

The penetration of gas in concrete is done by diffusion, an aspect that is related to the carbonation problems. The gas diffusion coefficient is usually determined with oxygen and

Table 3-4. Interpretation of AASHTO T277 results

Chloride penetration	Coulombs
High	>4,000
Average	2,000–4,000
Low	1,000–2,000
Very Low	100–1,000
Negligible	<100

Table 3-5. Characteristics of concrete oxygen diffusion

Concrete quality	D_o ($\times 10^{-8}$ m^2/s)
High	<0.5
Average	0.5–5
Low	>5

consists of passing oxygen through a specimen of concrete and measuring the amount that passes through under stabilized conditions (Schwiete 1969). Using Fick's first law, the diffusion coefficient D_o is obtained from:

$$D_o = S\, l/A \tag{3-14}$$

where

D_o = diffusion coefficient (cm^2/s)

S = volume of oxygen passing through (cm^3/s)

l = thickness of the specimen (cm)

A = area of the specimen (cm^2)

For concrete samples prepared according to (Schwiete 1969), the characteristics of concrete can be estimated according to Table 3-5.

Accelerated Carbonation

Accelerated lab tests can be performed to obtain an estimation of the resistance to carbonation of concrete. Samples are placed in a controlled chamber with a humidity of approximately 60%, a temperature of about 23 °C, and a CO_2 content of around 5% (Costa 1997) (Figure 3-9). Periodically the carbonation depth is measured.

The test carbonation coefficient is obtained, considering the diffusion law, by:

$$K_t = x/t^{0.5} \tag{3-15}$$

where

K_t = test carbonation coefficient (mm/year$^{0.5}$)

x = carbonation depth (mm)

t = time (years)

The test coefficient K_t needs to be multiplied by a correction coefficient used as a first approach to estimating the service life of the concrete due to the carbonation effect. The correction coefficient must be obtained from a correlation of in situ measurements to consider the real environmental conditions.

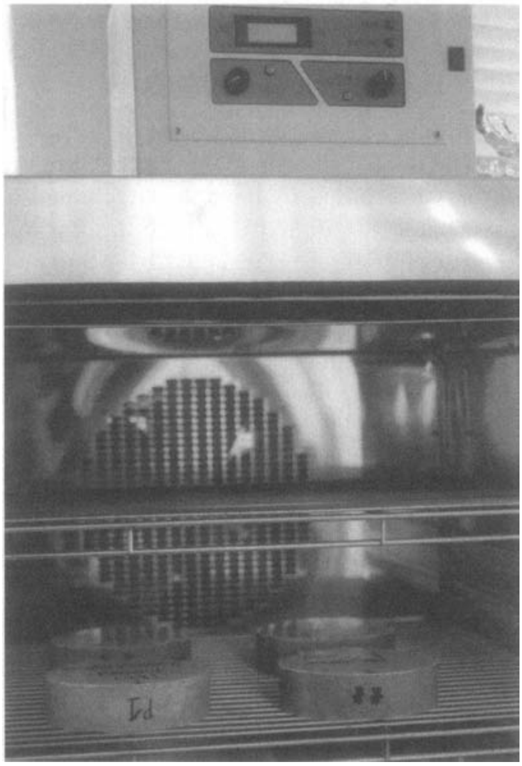


Figure 3-9. Specimens in a carbonation chamber

Other Tests

In addition to the aforementioned tests, which are more frequently used, several other lab tests can be used to characterize some aspects of the concrete durability characteristics.

Among these, the following can be used (Bungey 1989): pore analysis, petrographic or microscopic study of the concrete matrix, or specific tests for alkali-aggregate reaction, freeze-thaw resistance, abrasion resistance, etc.

3.2. In Situ Testing

In situ bridge tests are usually related to the mechanical or durability characterization of the materials already applied in the bridge or to the evaluation of the bridge structural behavior.

For the same concrete, some of the in situ concrete properties are different from those that are obtained with lab tests, mainly due to different concreting and curing procedures. That is why it is important not only to perform lab tests, but also in situ tests. Some of the in situ properties are in fact tested in the lab but with samples obtained directly from the bridge.

The in situ global evaluation of the bridge behavior is typically related to load tests performed at the end of the work or when some abnormal situation occurs leading to the necessity of the bridge evaluation.

Some of the tests more currently used and only related to mechanical and durability properties are now presented.

3.2.1. Mechanical and Geometrical Properties

Evaluation of Concrete Strength

Cores. The compression testing of cores drilled from the structure is one of the methods most frequently used to estimate the in situ concrete strength (Bungey 1989; BS 1881-Part 118).

After an inspection with a Covermeter that enables the detection of the location of bars, and therefore prevents their being cut, the core extraction location is marked. It should be stated that the in situ strength of concrete is usually higher at the bottom, than at the top parts of the same concrete element due to the compaction effect (Bungey 1989). The cores are then drilled with diameters ranging from 10 to 15 cm and with lengths around twice the diameter, with a minimum of one diameter (Figure 3-10). The top and bottom faces of the cores are then rectified and the cores are tested in compression, therefore obtaining the compression strength of each core f_{ci}.

To obtain the in situ concrete characteristic compression strength f_{ck}, from the core tests, there are two problems: the first problem is related to the number of cores available (the statistical value is small because only few cores can be extracted from a structure); the second problem is related to the correlation between f_{ck} in a standard lab test and the core test result f_{ci}.

The average concrete compression cube strength f_{cm} can be obtained from the core test results (for diameters above 10 cm), considering (Bungey 1989):

$$f_{cm} = (f_{cim} \pm \Delta) \cdot g \cdot s \cdot l \tag{3-16}$$

where

f_{cim} = average compression strength of tested cores

Δ = error (%) associated to the number of tested cores n ($\Delta = 12/n^{0.5}$)

g = correction factor for core geometry (length l; diameter d; $\lambda = l/d$):

horizontally drilled $-$ $g = 2.5\, f_{ci}/(1.5 + 1/\lambda)$

vertically drilled $-$ $g = 2.3\, f_{ci}/(1.5 + 1/\lambda)$

Figure 3-10. Drilling a core

s = correction factor for non-saturated cores (approx. = 0.9)

l = correction factor for relation between in situ and lab strength (around 1.25)

If the core contains bits of reinforcement bars, an additional correction factor should also be considered (Bungey 1989).

To obtain the characteristic strength of concrete based on the average value f_{cm}, it is necessary to assume a statistical distribution, as the cores results are but a few. The following equation is usually considered:

$$f_{ck} = f_{cm} (1 - 1.64\,\delta) \tag{3-17}$$

In this equation, δ ($\delta = \sigma / f_{cm}$) is the statistical variation coefficient of the sample, which in concrete structures has values around 10% for high quality controlled structures (precast), 15% for current construction, and higher values for low quality construction. So, to obtain an estimation of the characteristic strength, it is necessary to assume a value for δ, based on the inspection of the structure and eventually check against the results from the rebound hammer.

It is clear that the determination of the in situ compression strength of cores is a method with a certain margin of error that requires a good evaluation of some of the parameters by the technician in order to obtain reliable results.

Rebound hammer. This technique is a nondestructive test used to determine the superficial compression strength of concrete using very simple portable equipment. It is based on the relationship between the hardness of the concrete surface and its compression strength (Bungey 1989; BS 1881, Part 202). The equipment measures the rebound of a calibrated weight that is initially compressed by a spring against the surface (Figure 3-11).

The rebound of the weight indicates, in a scale, the rebound number N, which is correlated (for each piece of equipment) with the average compression strength f_{cm}, taking into account the position of the hammer during the test. This correlation is usually subject to an error margin Δ for each strength level.

The type of cement and aggregates, the environment humidity, the superficial carbonation, the surface irregularities, etc. affect the results. When testing an area, at least 10 points should be used and a variation coefficient δ of results less than 15% should be obtained for average concrete construction.

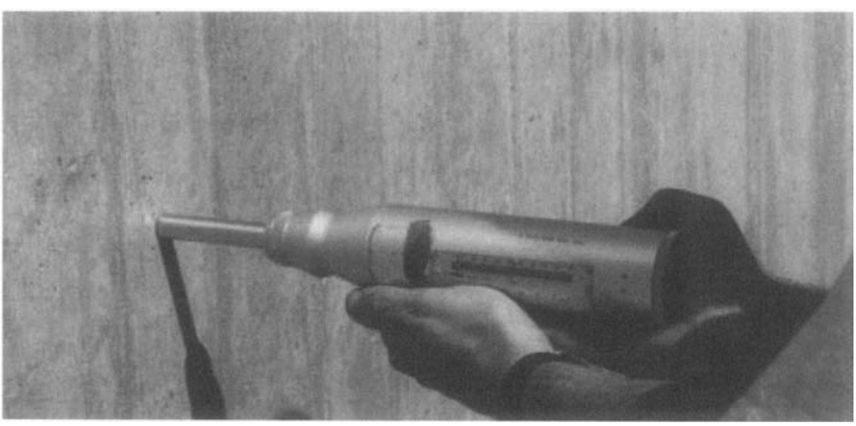


Figure 3-11. Use of the rebound hammer

The characteristic strength can be estimated from the rebound hammer results using the equation:

$$f_{ck} = (f_{cm} \pm \Delta)(1 - 1.64\,\delta) \tag{3-18}$$

Due to the lack of precision of this technique, the best results to estimate in situ concrete strength are obtained by testing the same area with the rebound hammer and cores, and after calibrating the hammer, using it to check concrete variations in other zones of the structure.

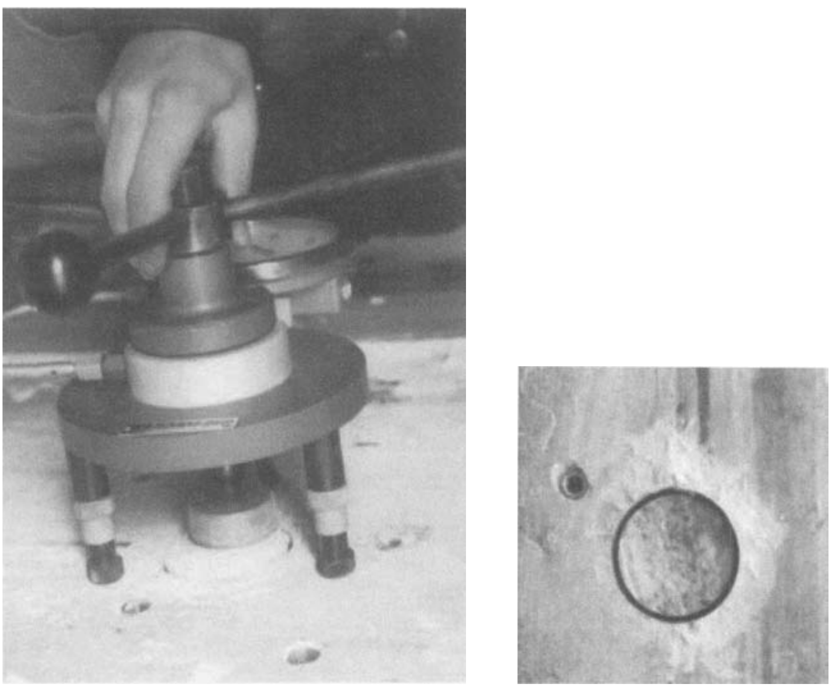
Pull Off. This test is specially used to obtain in situ the tensile strength of concrete or to check the adherence characteristics of new concrete (or steel plate) placed on old concrete, as a repair.

The test consists of partially drilling a small core entering in the old concrete and then applying a tensile force to the core top surface until it breaks, measuring the breaking force and thus the equivalent tensile stress f_{ct} (BS 1881: Part 207) (Figure 3-12).

Qualitatively, the result of this test immediately shows whether the adherence is good (i.e., if the breaking surface occurs inside the old concrete).

Evaluation of Concrete Young Modulus

Cores. The in situ evaluation of the Young modulus is performed as described for the lab test, but using as specimen a core drilled in situ.



a. b.

Figure 3-12. Use of the pull-off test; **a.**, pulling the concrete core; **b.**, broken core

Creep and Shrinkage

The in situ measurement of creep and shrinkage characteristics is done in a similar way as in laboratory conditions, even though special measures need to be taken, because the environmental parameters close to the bridge are always changing.

To obtain reliable results, two identical specimens are placed close to the bridge (or even on the bridge deck), in order to have similar temperature, humidity and rain conditions as the bridge deck (Figure 3-13) (Santos 1997; Fernandes 1994). These specimens should also have geometry similar to the equivalent thickness of the bridge deck, related to the time dependant effects.

One of the specimens (the creep specimen) is subjected to constant compression inside a steel frame, using a hydraulic jack with a uniform pressure control system. The strains of this specimen are designated as ε_1.

The other specimen (shrinkage specimen), geometrically identical, is under free movement conditions and their strains over time are measured and designated by ε_2. The temperature inside this specimen is also measured. The shrinkage strains ε_s are then obtained

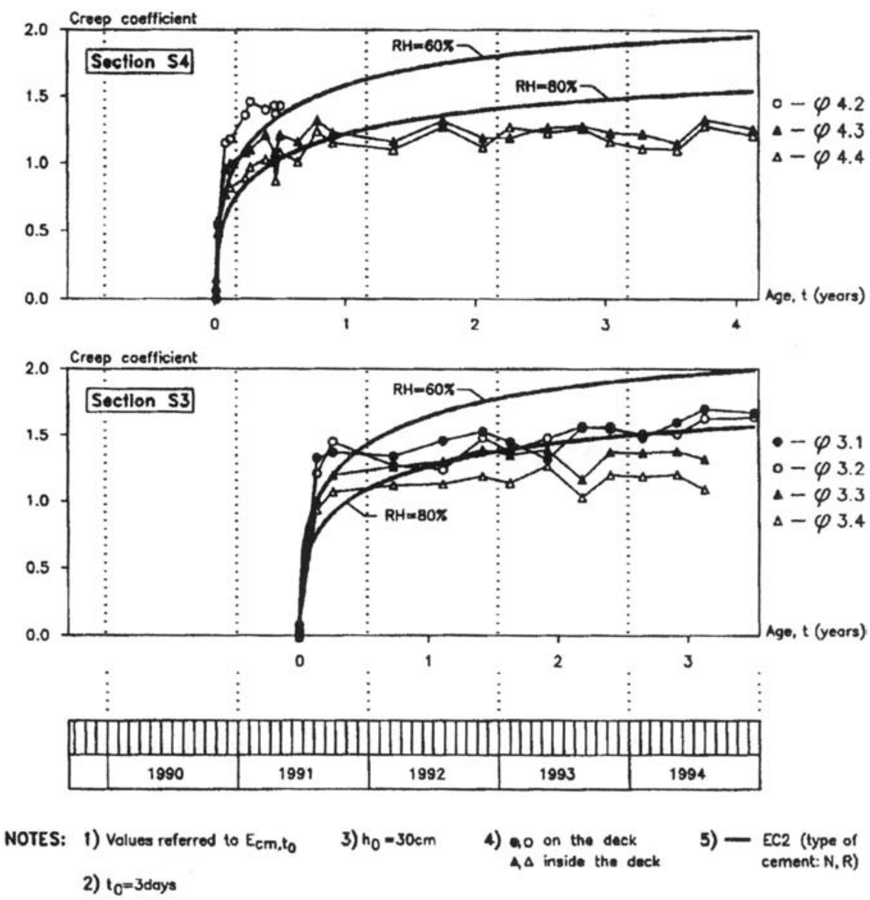


Figure 3-13. In situ measurement of creep coefficients evolution

from this measurement subtracting the temperature strains ε_t, computed from the temperature measurements.

$$\varepsilon_s = \varepsilon_2 - \varepsilon_t \tag{3-19}$$

The creep strains ε_c are then obtained from the strains from the first specimen by subtracting the strains from the second specimen

$$\varepsilon_c = \varepsilon_1 - \varepsilon_2 \tag{3-20}$$

Examples of results of in situ creep measurements are shown in Figure 3-13 (Fernandes 1994).

Evaluation of Concrete Uniformity

Ultrasonic test. This test basically consists of measuring the time that sound waves take to go from a transmitter to a receptor (Figure 3-14). The speed of sound is greater in solids than in open air, and also increases with concrete compaction. This property has been used mainly to check a concrete element's uniformity or the existence of cracks or cavities in the interior of elements (Bungey 1989; BS 1881: Part 203).

It is a practical test to be performed in situ and it gives reliable results, allowing the detection of internal anomalies in concrete structures, namely by comparison with measurements in zones without anomalies.

This test has also been used under laboratory conditions in order to evaluate concrete strength or Young modulus, but a very precise calibration with results from other tests needs to be performed to obtain reliable results.

Evaluation of Concrete Damage by Fire

Fire Behavior (FB) Test. The FB test was developed to evaluate the thickness of a concrete surface damaged by a fire (Neves et al. 1997; Branco 1999). The method first consists of

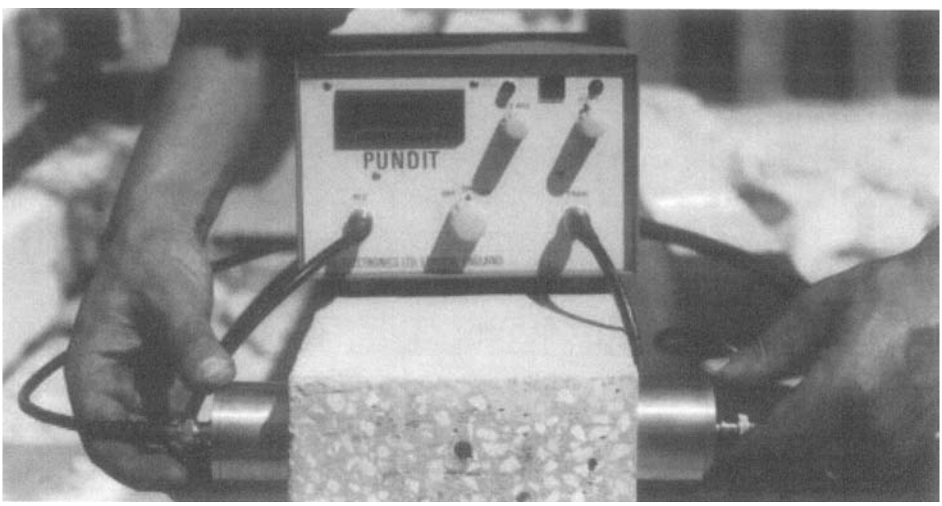


Figure 3-14. Use of ultrasonic measurements

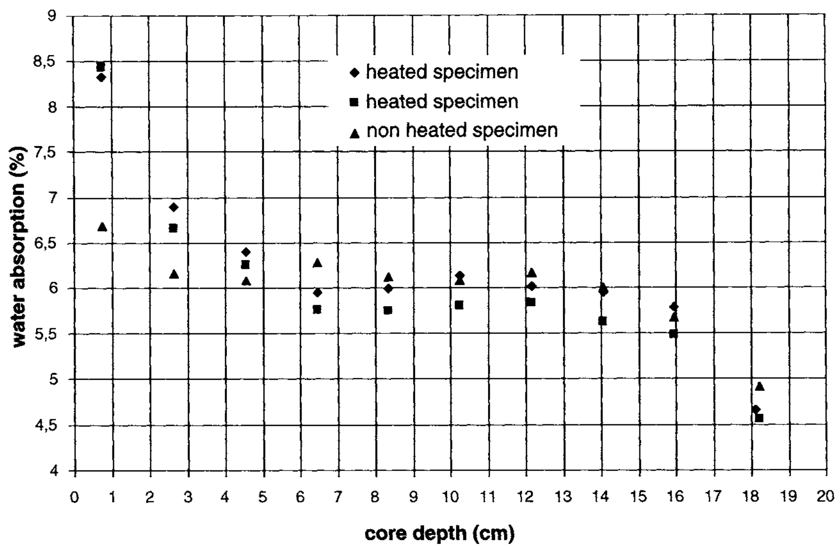


Figure 3-15. Water absorption evolution in core slices from a fire-damaged structure

drilling cores in the damaged zone. Then the cores are cut into slices (1.5 cm thick) and the water absorption of each slice is measured.

By plotting a graph of the absorption values with the slice position along the core, the absorption is seen to decrease from the surface to the interior until it stabilizes (corresponding to a temperature of the slice around 200 °C), indicating the limit of the damaged zone (Figure 3-15).

Evaluation of Steel Mechanical Properties

Reinforcement Tensile Test. The in situ evaluation of reinforcement strength is performed in the lab, just like in the lab tests, but with specimens cut from the structure. Care must be taken in selecting the steel specimens to prevent cutting the main reinforcement bars. If this is done, additional bars should be welded afterwards to replace the ones cut.

Prestress Tests. The tests to characterize the prestress steel after its application in the bridge are difficult to perform because cutting specimens in the bridge leads to a decrease in its structural safety. This is done only within a rehabilitation process, where temporary safety is ensured.

Evaluation of Reinforcement Cover and Position

Within a repair or rehabilitation process, especially when design drawings are not available, it is necessary to identify the existing reinforcement geometry and its cover. The cover depth is especially important for reasons of durability because it represents the reinforcement protection and the main barrier to aggressive elements. There are several techniques to obtain this information.

The classical method is to use *hand tools* to remove the concrete cover locally in some of the main structural sections. This is a semidestructive technique, but it allows the direct measurement of the bars diameter and spacing, of the concrete cover and the visualization of eventual corrosion.

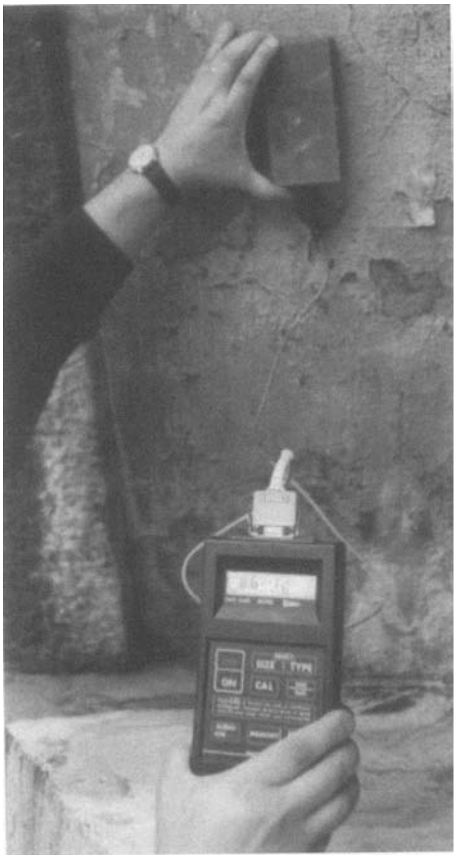


Figure 3-16. Use of covermeter

A *Covermeter* is a device that detects the bars, based on the alteration they introduce in a fixed magnetic field of the search unit (BS 1881: Part 204). It is a strictly nondestructive method whose results are affected simultaneously by the bar diameter and the cover thickness (Figure 3-16).

It gives reliable indication about the bars position but it needs to be calibrated in some sections where the cover is really measured (previous method), in order to be used as bar diameter and cover indicator.

X rays have also been used to detect reinforcement and prestress steel. A radiation source is placed on one side of the concrete element and film sheets are placed on the other side. The radiation passing through concrete shows a picture of the existing reinforcement, becoming clearer as the reinforcement level is closer to the film (Branco 1984; BS 1881: Part 205 1970). It is a method whose application has the drawback of being limited to concrete elements with a small thickness, which requires a long period of X-ray exposure, leading to the implementation of health protection measures.

Evaluation of In Situ Stresses

The best way to evaluate the existing stresses in a bridge due to self-weight is to monitor the structure from the beginning of construction and continuously measure the stresses

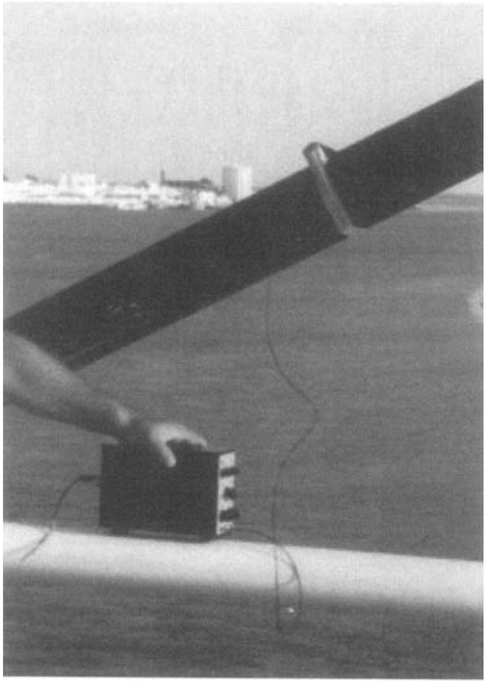


Figure 3-17. Measuring cable vibrations under wind excitation

in the main sections. There are only a few reliable techniques to evaluate these stresses after the bridge work is concluded without the implementation of a monitoring plan.

Dynamic Tests. Dynamic tests are applicable in evaluating the stresses in stay cables or in external prestress cables (Branco 1993). They are based on the relationship between the cable force F and the cable vibration frequency f, L being the cable length, and m its mass per length unit.

$$f^2 = F/(4\ m\ L^2) \tag{3-21}$$

Forcing cable vibration, by manual excitation or wind action (Figure 3-17), and measuring the cable first mode frequency f with an accelerometer, the axial force F can be easily determined, obtaining reliable results (Figure 3-18).

Anchorages Pulling. If cable or tendon anchorages are designed to be pulled, the existing cable forces can also be controlled by pulling the anchorages until the existing compression on the anchorage zone is cancelled.

Stress release. In order to obtain the in situ stresses in concrete structural elements, the best methods are associated with the stress release techniques.

The first method, developed for rock mechanics, consisted of the placement of strain gauges in the concrete surface. The surface is then cut with a circular saw and a plan hydraulic jack is introduced in the cut. Pressure is increased until the strains in the strain gauges return to the initial position, the measured pressure being the estimated initial in situ pressure.

An adaptation to this method has been recently developed, in which the cover of a reinforcement bar is removed, strain gauges are glued to the bar and the bar is then cut

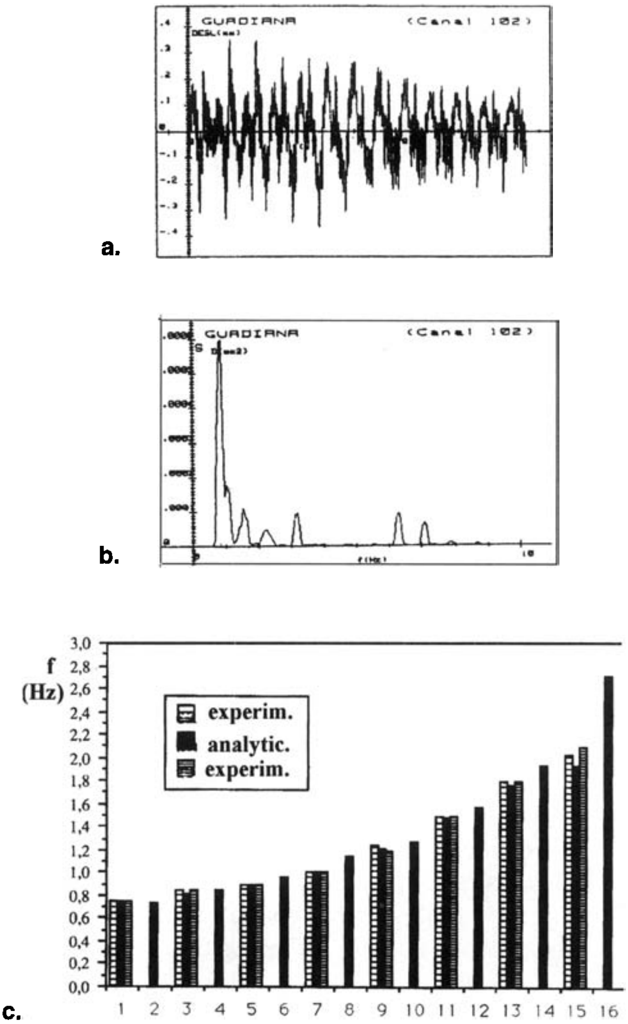


Figure 3-18. Test of cables in Guadiana Bridge, Algarve, Portugal: **a.**, cable vibrations; **b.**, frequencies; **c.**, comparison between analytical and experimental frequencies

(Figure 3-19). The change in strains measured in the bar indicates the stress level of the bar. The technique is simpler than the previous one and gives a conservative value of the existing stress level (Santos 1999).

3.2.2. Durability Properties

Depth of Carbonation

The concrete carbonation arises mainly from CO_2 penetration in the concrete surface that leads to a change from the initial pH value of around 12 to lower values. Corrosion is initiated when the concrete pH, close to reinforcement, reaches values around 9 to 10.

In situ depth of carbonation is obtained by making a hole in the concrete surface and, after carefully removing the powder, applying a phenolphthalein spray (1% of phe-

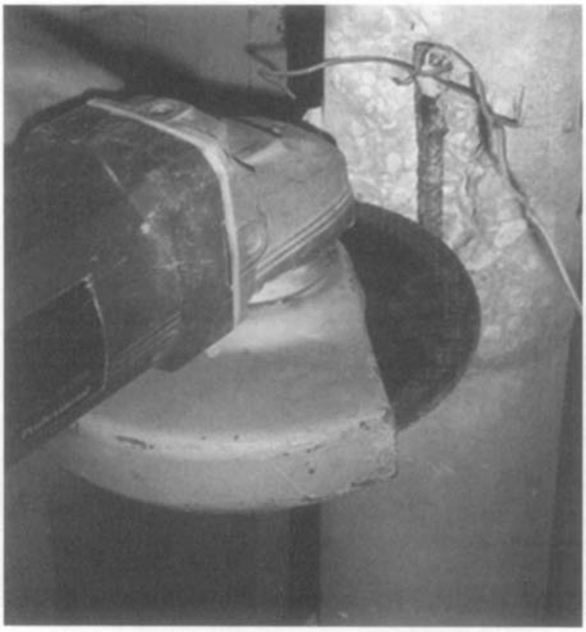


Figure 3-19. In situ evaluation of existing stresses

nolphthalein in alcohol at 70%) (RILEM CPC-18 1984) calibrated for an approximate pH level of 9.5 as an indicator of acidity/alkalinity. The change in color to pink, corresponding to a pH above 9.5, shows the approximate depth of the carbonation front (Figure 3-20).

If cores are taken from the structure, they can also be sprayed with phenolphthalein to measure the carbonation depth.

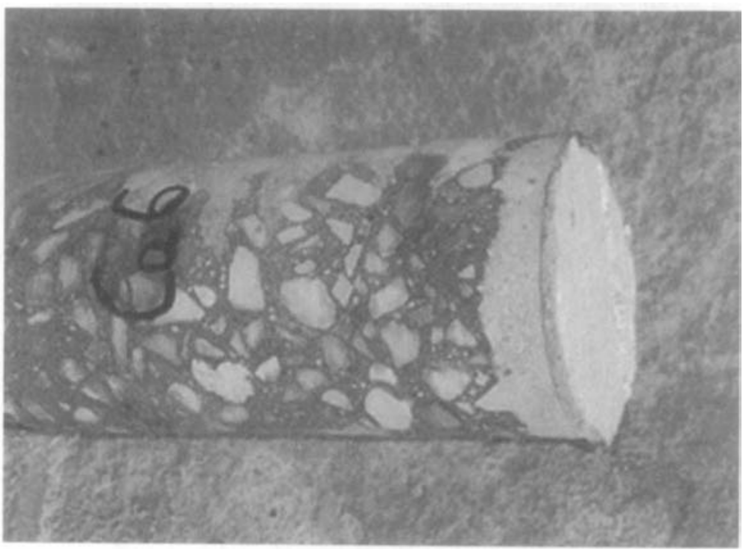


Figure 3-20. Use of phenolphthalein for carbonation depth evaluation in a core

Knowing the carbonation depth x and the age of the structure t, an average carbonation coefficient K can be obtained from:

$$K = x/t^{0.5} \tag{3-22}$$

This coefficient, representing the effective carbonation of the structure under actual environmental conditions, allows an estimation of its service life, as it is considered that corrosion begins when the carbonation front reaches the reinforcement bars depth.

Chloride Penetration

In situ chloride penetration can be obtained by making a hole in the concrete surface and collecting the concrete powder at different depths (every 5 mm) (Figure 3-21). The hole must be cleaned of any powder between each measurement depth.

The chloride percentage is then measured for each level in terms of total chlorides (acid attack) or free chloride (water extraction) and the chloride profile with depth is then obtained (Costa 1997; Salta 1999).

Using this profile and the age t of the structure, the diffusion coefficient D_c can be obtained using Fick's second law, as well the chloride percentage $C(x, t)$ at depth x and at the surface, C_s.

$$C(x, t) = C_s\,(1 - x/(2 \cdot (3\,D_c\,t)^{0.5}))^2 \tag{3-23}$$

Corrosion Potential

The half-cell potential is a parameter indicating the corrosion situation of a metal in a specific environment. The type of concrete, resistivity, humidity, carbonation, etc., affect the

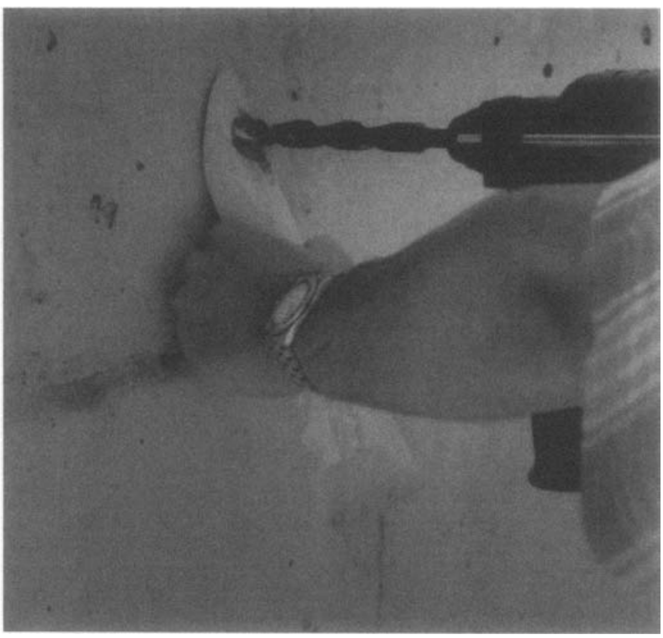


Figure 3-21. Sampling concrete powder for chloride level measurement

Table 3-6. Relationship between half-cell measurements and active corrosion probability

Half cell (mV)	Chance of corrosion
<–350	90%
−250 to −350	50%
>–200	10%

potential value, so this test is mainly used to obtain maps of equal potential levels and to determine the zones where active corrosion has a high probability of occurring (Table 3-6).

The technique uses a reference electrode, as external gauge, that is connected to a reinforcement bar (Figure 3-22). The potential between the electrode and the bar is measured at several points, from which a potential map of corrosion can be produced (RILEM TC-154 1998).

The method is of practical application and is useful, mainly if used together with other corrosion detection methods. Using copper/copper sulfate reference electrodes, the following reference values are indicated (Bungey 1989) (Table 3-6):

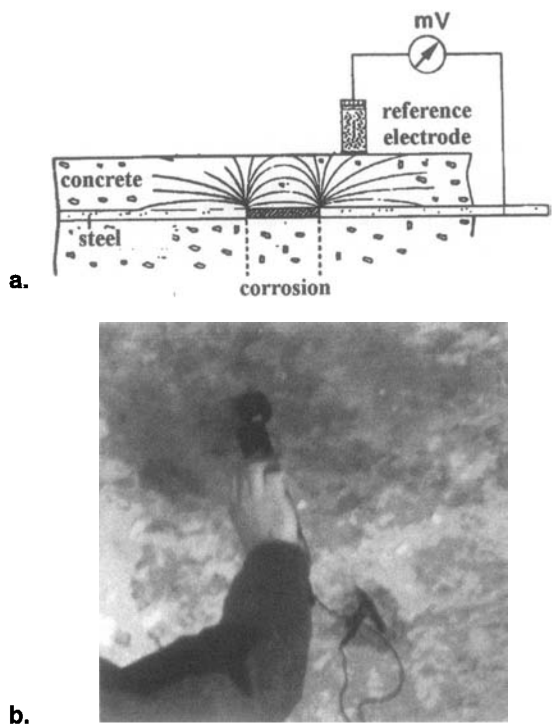


Figure 3-22. Half-cell potential corrosion measurement: **a.**, schematic drawing; **b.**, measurement

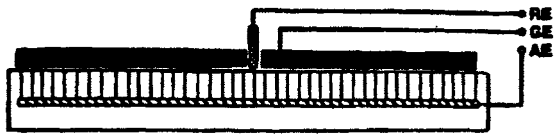


Figure 3-23. Measurement of corrosion rate

Corrosion Rate

The best technique to evaluate corrosion rate is through the periodic measurement of the mass loss. Unfortunately, this can only be applied to steel structures exposed to open air where the ultrasonic method is the most effective way.

The corrosion rate I_c in concrete reinforcement can also be obtained from polarization resistance R_p. In fact, close to the great probability of corrosion potential (Salta 1999), the following equation is valid:

$$I_c = B/R_p \cdot A \tag{3-24}$$

where

I_c = corrosion density current ($\mu A/\text{cm}^2$)

R_p = polarization resistance ($k\Omega$)

A = area of polarized reinforcement (cm^2)

B = constant with average values of 26 mV under active corrosion and 52 mV under passive situation

To measure polarization resistance, a technique with a reference and an opposition electrode is used, with reasonable results for uniform and localized corrosion (Figure 3-23). Frequently a protection ring must also be used (Salta 1999).

The corrosion rate is also approximately correlated with the concrete resistivity ρ by the formula:

$$I_c \cdot \rho = 10^4 \ \Omega \text{ cm/mA/cm}^2 \tag{3-25}$$

The level of corrosion can be evaluated from the corrosion rates of Table 3-7 (Chess 1996).

Table 3-7. Relationship between corrosion rate and corrosion level

Corrosion level	I_c (mA/cm²)
High	>1
Average	0.5–1
Low	0.1–0.5
Negligible	<0.1

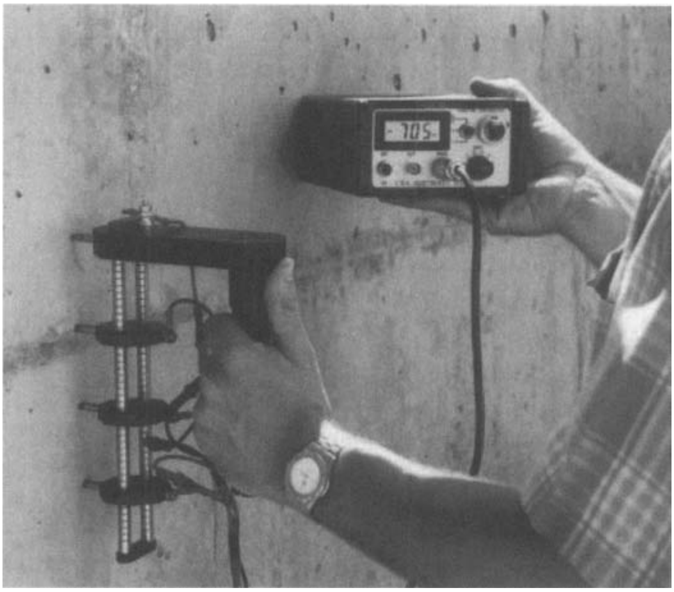


Figure 3-24. Measurement of concrete resistivity

Concrete Resistivity

The circulation of ions in concrete can be analyzed through its electrical resistivity. This analysis gives information on concrete quality, namely related to chloride diffusion.

The technique consists of the placement of four electrodes in line on the concrete surface. An electrical current passes between the two extremities and the associated potential is measured in the middle electrodes (Figure 3-24). The concrete resistivity is given by:

$$\rho = 2\,\pi\,s\,V/I \tag{3-26}$$

where

ρ = resistivity ($k\Omega$ cm)

s = electrode spacing (m)

V = voltage drop (Ω)

I = current intensity (mA)

Based on the resistivity results, the corrosion risks of the reinforcement bars can be estimated from Table 3-8 (Bungey 1989).

Humidity in Concrete

Humidity in concrete is a factor of acceleration of the durability problems. For humidity levels below 50%, corrosion rarely takes place. This fact is being used to protect steel box girders or even steel cables through the use of ventilated solutions to keep the humidity below that critical level.

Table 3-8. Relationship between corrosion potential and resistivity

Corrosion level	Resistivity (kΩ-cm)
Very High	<5
High	5–10
Moderate	10–20
Low	>20

Humidity in concrete can be measured from samples taken from the structure, placed in an insulated container and measuring the water content in the laboratory. Alternatively, electrical and chemical gauges can be placed inside an insulated hole in the concrete surface to measure its humidity (Figure 3-25).

Water Absorption

Besides the unidirectional lab tests that can also be performed with cores drilled in situ, other tests have also been developed for in situ direct measurement of the water absorption

- The INSAT (initial surface absorption test) is normalized in (Bs 1881: Part 5). A superficial area of concrete is subjected to a water pressure of 200 mm Hg and the absorption is measured in a capillary tube. The absorption ($cm^3m^{-2}s^{-1}$) for 10 minutes indicates low absorption (<0.25) up to high absorption (>0.5) in concrete.
- The Figg method (Bungey 1989) consists on drilling a hole in concrete that is then subjected to an internal water pressure and periodically measuring the decrease in pressure. Based on the time (seconds) necessary to absorb 0.01 ml of water, the absorption concrete characteristics may change from low absorption (>200 s) up to high absorption (<50 s).

Air Permeability

The Figg method (Bungey 1989; Gonçalves 1999) can also be used for in situ air permeability measurement, with the time to decrease the pressure from 55 kPa to 50 kPa being

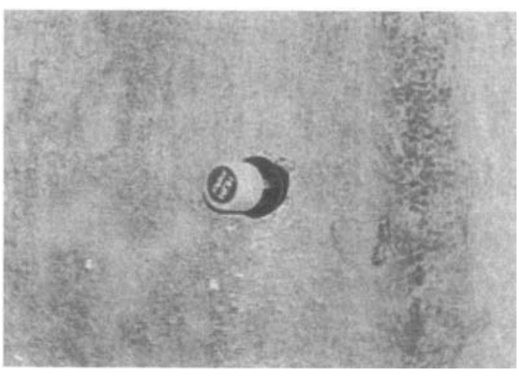


Figure 3-25. Chemical probe to measure in situ concrete humidity

considered the reference value. The absorption concrete characteristics may change from low absorption (>300 s) to high absorption (<100 s).

3.2.3. *Load Testing*

Load tests are performed to check the global behavior of the bridge and should always be performed in association with a parallel numerical analysis. There are two main categories of load tests, which are performed at different stages of the bridge life:

- The Reception Load Test is performed at the end of construction. Its main objective is to compare the experimental behavior of the bridge with the design numerical analysis (Figure 3-26).
- The Evaluation Load Test is currently used to define the bridge load capacity. It enables a decision to be made relating to eventual bridge rehabilitation. A preliminary numerical analysis should also be performed before the test.

In terms of the type of the tests performed, load tests can be divided into static and dynamic tests:

a) Static Load Tests. In these tests, deflections and strains in the main cross sections are usually measured, under the weight of truckloads, to compare the experimental values with those from numerical models.

In the Reception test, the designer should define loads not susceptible of causing cracking in the bridge. Values corresponding to 70% to 80% of the maximum bending moments (or other action forces) associated with design loads are usually used.

In the Evaluation test, increasing loads are applied until a predefined serviceability limit (deflection or crack width) is reached. The allowable loads are then rated based on a defined margin of safety.

Figure 3-26. Reception load test in a railway bridge

Figure 3-27. Measurement of displacements under bridge deck

In a static test, measurements are typically performed with the following sequence: before the trucks are positioned; with trucks in position; and after the trucks leave the bridge.

Based on the measured deflections and strains the recovery of the bridge behavior is evaluated. Furthermore, influence lines are also performed to obtain the continuity behavior of several spans with the movement of the trucks (this allows to check the behavior of joints and bearings), which should move slowly along the bridge.

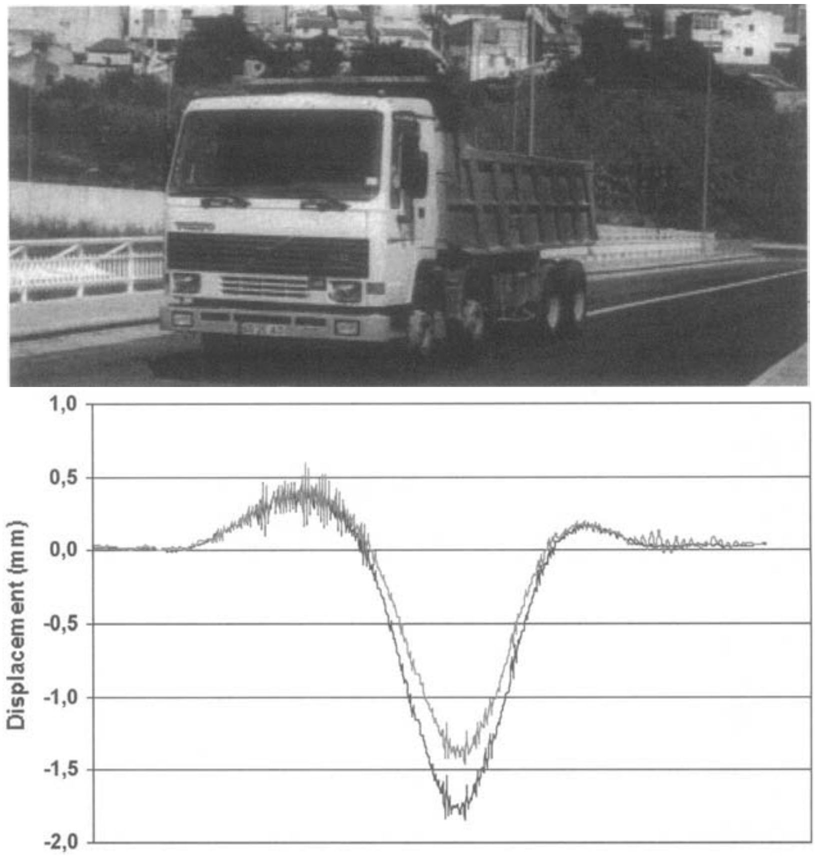
A passage with the trucks, moving quickly, frequently allows the definition, from the influence lines, of the first vertical frequency and the dynamic effect, due to the oscillations obtained with the truck (Figure 3-28).

Measurements of vertical deflections are usually performed with displacement transducers placed under the bridge in contact with a weight suspended by a wire from the deck (Figure 3-27, allowing for high precision (≅0.001 mm). Tests should be performed practically without wind to prevent the vibration of the wire. If the bridge is over deep water, measurements should be performed by surveying, which leads to lower precision (≅0.5 mm). Alternatively, the vertical deflection can be computed from the rotations along the deck, measured with inclinometers.

Measurements of strains are made with strain gauges placed in appropriate fibers of the main cross sections. Load tests are usually quick, so no corrections need to be performed due to variations associated with temperature, creep, or shrinkage effects.

b) Dynamic Load Tests. In these tests, measurements are performed to obtain the dynamic characteristics of the bridge, namely, their main frequencies and damping.

Measurements are performed using accelerometers (with results in terms of accelerations, velocities, or displacements of vibrations) placed in the most relevant cross sections and in the direction of the vibration to be measured (Figure 3-29). If accelerometers are placed in both lateral extremities of the deck, deflection and torsion frequencies can be obtained. The measured accelerogram is then numerically analyzed using a Fourier analysis to



a.

b.

Figure 3-28. Load test influence lines, urban viaduct, Lisbon: **a.**, moving truck; **b.**, displacement influence lines for central midspan

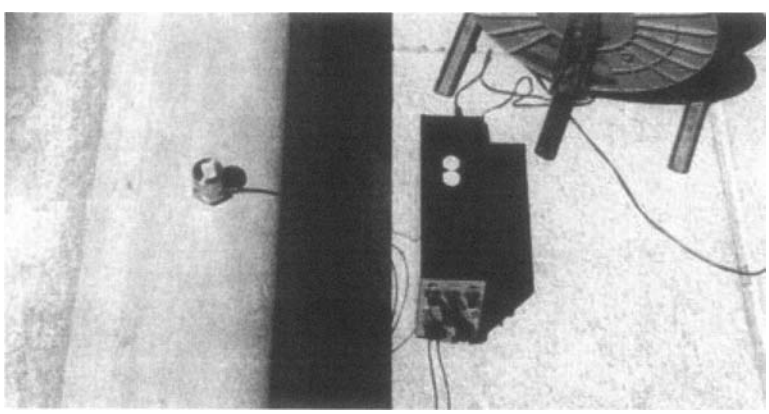


Figure 3-29. Measurement with accelerometer under way

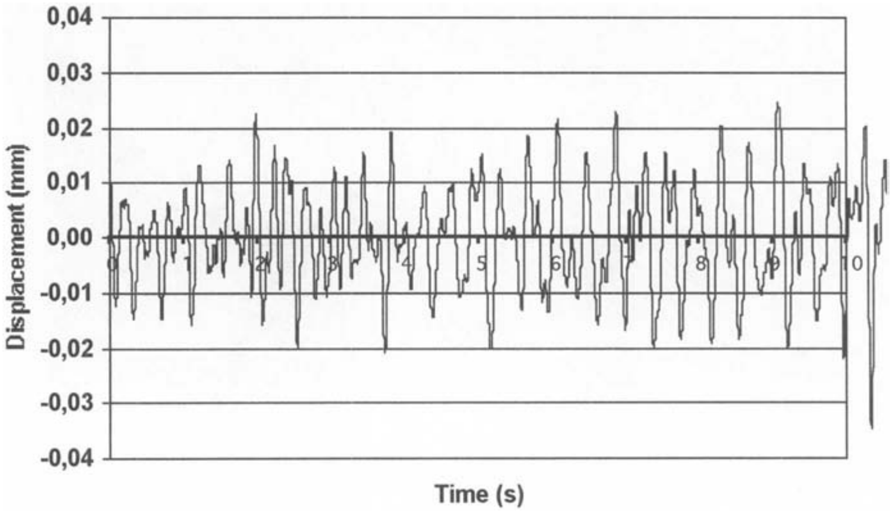


Figure 3-30. Accelerogram

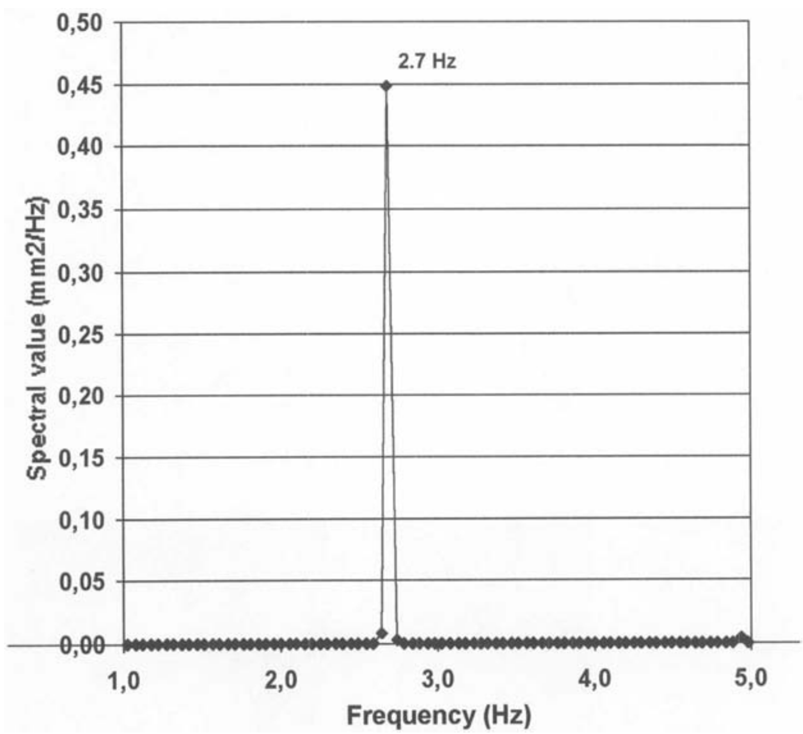


Figure 3-31. Spectral analysis

Figure 3-32. Truck with step

obtain a spectral analysis indicating the main frequencies (Figures 3-30 and 3-31). The accelerogram itself also shows the highest vibration levels occurred during the test.

To characterize vertical vibrations, a truck usually traveling at a speed above 30 km/h, passes over a wood step placed in the bridge and then stops, the vibration recording beginning after the truck stops (Figure 3-32). For horizontal vibrations, the wind effect is usually enough. Longitudinal vibrations can be obtained for flexible bridges with trucks braking on the bridge.

The truck braking technique for measurement of the longitudinal frequencies allows also the checking of the bridge longitudinal stiffness k. This can be performed from the longitudinal frequency value f and using the bridge deck mass m ($k = 4 \cdot \pi^2 \cdot f^2 \cdot m$) or by measuring the acceleration α inside the braking truck with mass m_t and dividing the product by the longitudinally measured displacement of the bridge d ($k = m_t\alpha/d$).

When seismic shock bearings (usually placed longitudinally), or even some sliding bearings, exist in the bridge, frequencies similar to the numerical model are difficult to obtain as the above excitations are not enough to move the bearings and frequencies are obtained related to a bridge fixed in those points (Figure 3-33).

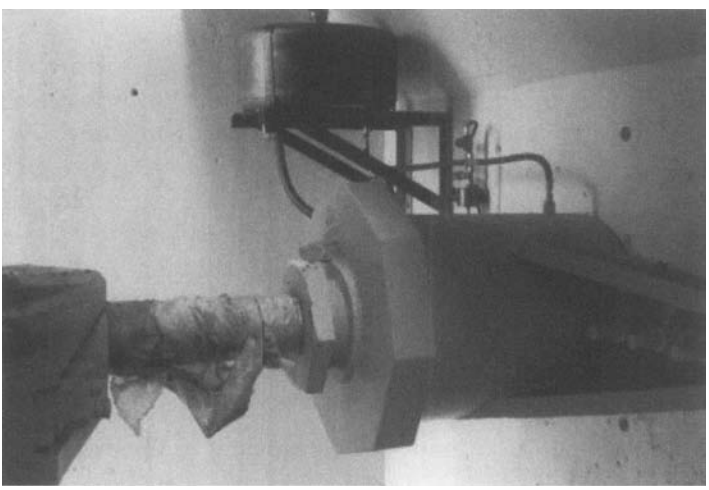


Figure 3-33. Seismic shock hydraulic bearings

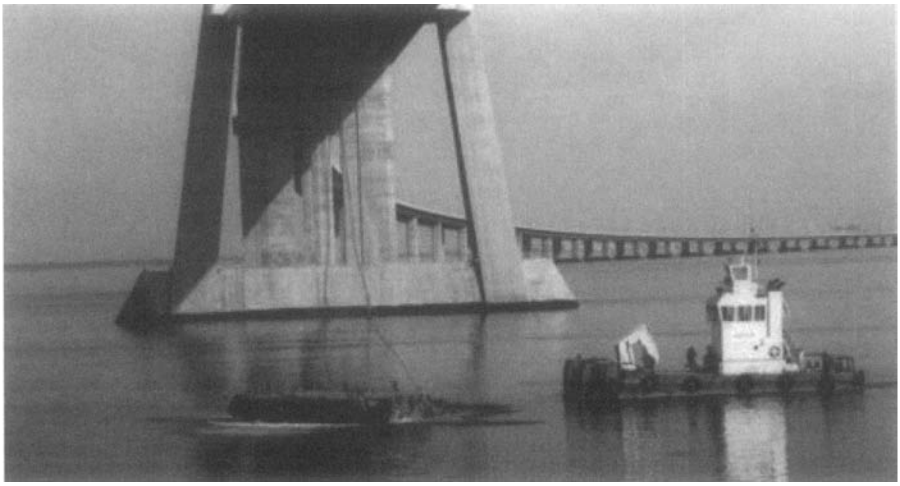


Figure 3-34. Suspended weight for measurement of vibrations, Vasco da Gama bridge, Lisbon, Portugal

Several other methods have been used to put the bridge vibrating with forced vibrations, namely, firing a cannon on the bridge, dropping a weight suspended from the bridge (Figure 3-34), etc. Recently, a technique consisting of a suspended pendulum with an oscillation frequency defined according to the bridge frequencies has proved quite satisfactory. The pendulum is made to oscillate in the desired direction and the main bridge vibrations are measured.

The vibration measurement technique, as already referred, can also be used to obtain the forces in stay cables. In fact, if their frequency f is obtained either from wind or by manually induced vibrations, the existing cable force F can be analytically quantified ($F = 4f^2\, m\, L^2$).

3.3. Bridge Monitoring

3.3.1. Monitoring Strategy

The monitoring of bridges is usually applied only to major bridges where the costs of a structural accident are enormous, leading to a procedure in which the bridge is under permanent observation, allowing the prevention of eventual anomalies, and in the case of an important accident (earthquake, strong wind, fire, etc.), quick decisions about its safety are expected.

The objective of bridge monitoring is mainly to check the structural safety of the bridge throughout its life by controlling the evolution of several parameters. The experimental values of these parameters are compared with the design analytical results and lead to predefined activities. These activities should be considered at the design stage, defining levels of action for each structural measurement parameter, of the following type:

- Level 1 – Parameter A < x – No action required;
- Level 2 – Parameter A > x and < y – Reanalysis by the designer;
- Level 3 – Parameter A > y – Stop traffic until deep analysis of the bridge is performed.

Level 2 needs a reanalysis of bridge safety and the understanding of the anomaly by the designer. Typically, a decision is expected from the designer such as do nothing, implement maintenance or repair action.

Level 3 is defined for a situation of important accident (earthquake, strong wind, etc.). After the inspection of the bridge and the analysis of the anomalies, the designer will define the most convenient procedures.

The parameters that are monitored in major bridges are related to the environment, structural behavior, and durability. Associated with environmental effects, wind speed, air temperature, earthquake acceleration, and water level/speed are usually measured. Related to the structural behavior, the displacements, the internal temperatures, the strains and the vibrations at special points in the structure are usually measured. Associated with durability, the parameters controlled are usually associated with reinforcement corrosion and/or carbonation depths or chloride content profiles.

3.3.2. Measurement of Structural Parameters

The in situ measurement of structural parameters requires the installation of special equipment during construction, and the measurements can be performed either manually, close to the equipment, or in a centralized office where the equipment results are connected (by cables or by cellular phone and modem) (Pincent 1999). In both situations, the equipment needs easy access to perform the measurements and maintenance. The following parameters are usually considered for structural monitoring (Fernandes et al. 1994):

Measurement of Displacements

Displacements of main structural elements. Typically, vertical deflections of main spans and horizontal displacements of towers and high columns are controlled. These displacements are usually measured by surveying methods leading to a precision of about 0.5 mm (depending on how far from the elements the equipment is).

Displacements in joints can be measured on line, with displacement electrical transducers placed between the sides of the joint (these transducers allow a precision of 0.001 mm), or periodically by in situ mechanical measurement with displacement mechanical gauges (precision 0.01 mm) or even using a precision rule (precision 0.1 mm).

Displacements in movable bearings can be measured on line, with displacement electrical transducers placed between fixed and movable plates or periodically by identical in situ mechanical measurements with displacement mechanical gauges or a precision rule.

Measurement of Rotations

Rotations of main structural elements. Typically, rotations of towers and high columns are obtained from surveying measurements. The use of electrical inclinometer transducers allows on line measurements (precision of 0.01°). There are also mechanical air bubble inclinometers for in situ manual measurements (precision 0.1°) (Figure 3-35).

Measurement of Strains

Strains in main structural elements are measured with strain gauges installed in main structural sections like mid-span and supports and at the base of columns. Electrical strain gauges, usually glued to reinforcement bars, or vibrating wire gauges inside concrete (precision 1 ×

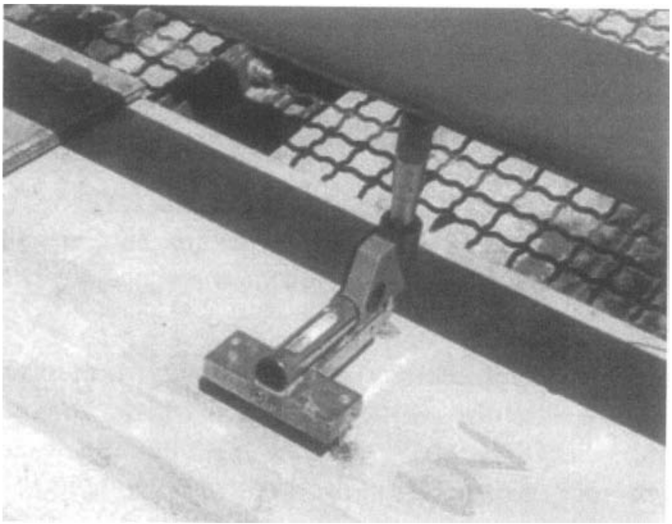


Figure 3-35. Air bubble inclinometer

10^{-6} strains) are usually used, which have a higher durability and precision and are installed during construction suspended from reinforcement and allow on line measurements (Figure 3-36).

Measurement of Temperatures

Temperatures in typical cross sections are measured with electrical thermal couples positioned at different depths in the thickness of the section allowing the acquisition of differential and average temperatures in the section (Figure 3-37). They allow online measurements and have a precision of around 0.1°.

Measurement of Forces

Forces in bearings can be measured with electrical load cells placed between the bearing and the structure, and allowing on line results.

Forces in steel bars can also be measured with electrical load cells, or, more easily, by gluing electrical strain gauges to the bar.

Forces in cables are measured using electrical (or oil pressure) load cells applied in the anchorage zone (also in prestress tendons) or by measurement of the cable vibrations, whose frequency is related to the applied tension force. Strain or displacement gauges can also be used to measure the forces in cables (Figure 3-38).

Measurement of Vibrations

Vibrations. Measurements are performed with electrical accelerometers (mostly of the piezoelectric type) with sensitivity as low as 0.1 Hz. The signal measured can be electrically derived or integrated, leading to results in accelerations, velocities or displacements of vibrations. A Fourier analysis leading to a spectral diagram with peaks representing the main frequencies can then be used to treat the output signal.

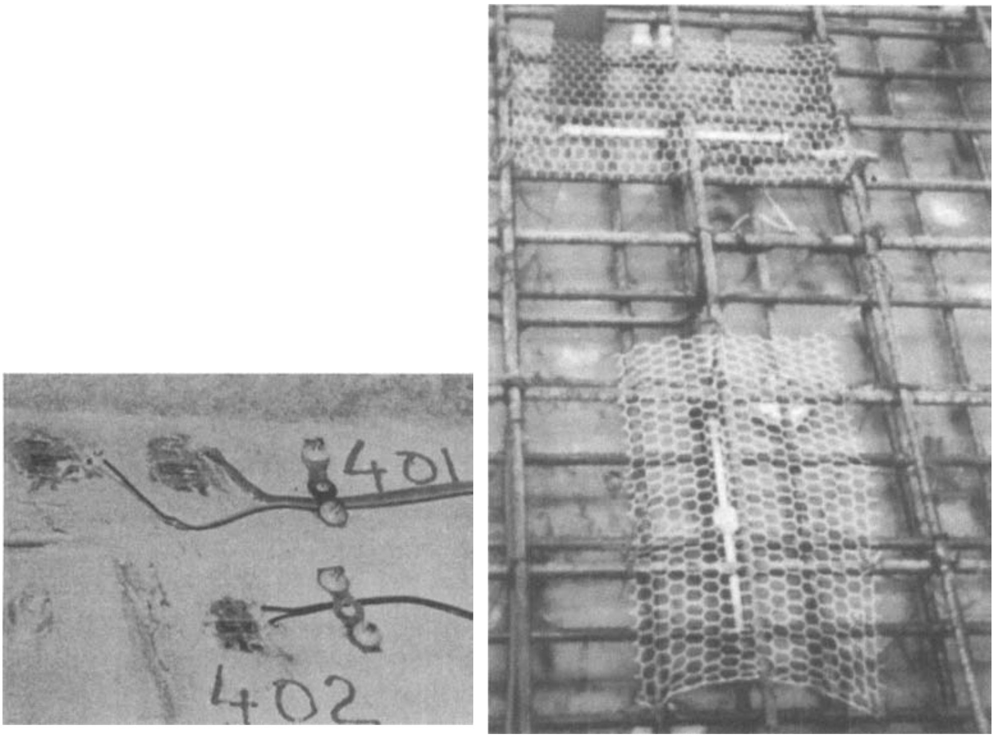
a.  b.

Figure 3-36. Measurement of strains: **a.**, electrical strain gauges; **b.**, placement of a vibrating wire transducer

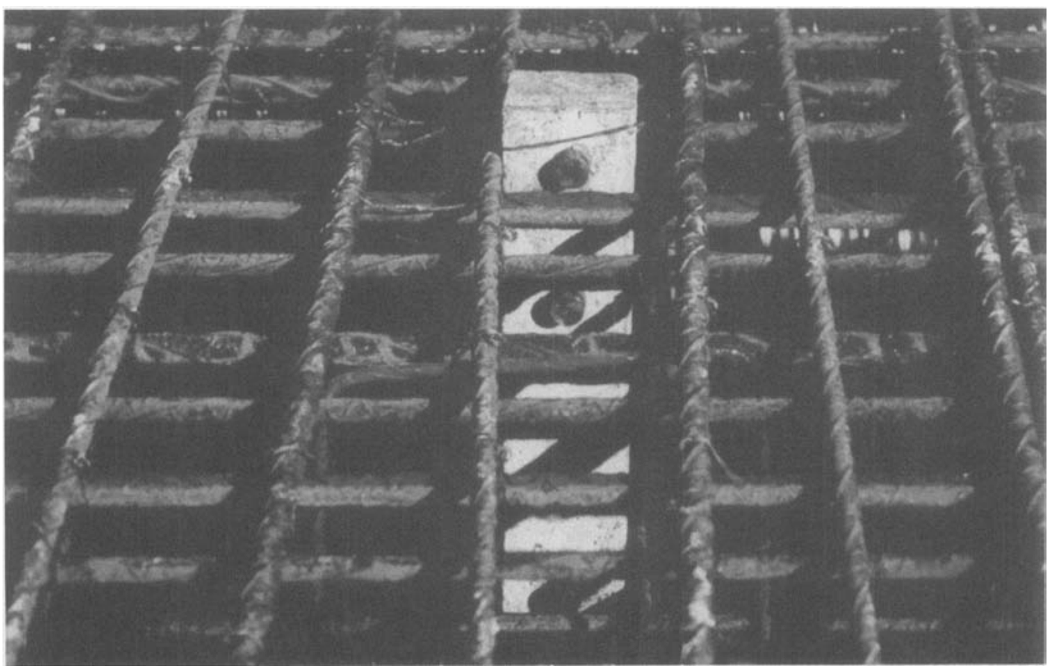


Figure 3-37. Placement of thermal couples

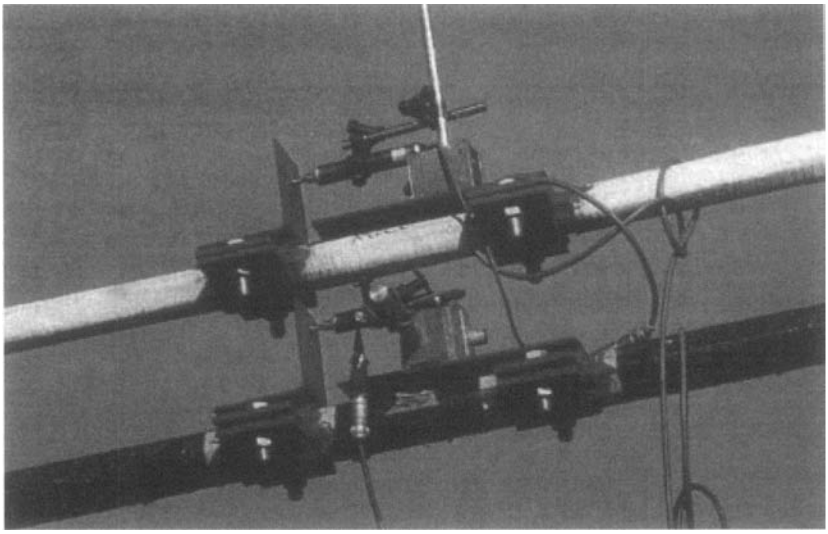


Figure 3-38. Measurement of strains in cables with displacement transducers

Measurement of Environmental Parameters

The structural measurements need to be correlated with some environmental characteristics such as:

- hourly environment air temperature (to correlate with displacement measurements);
- average daily environment humidity (to correlate with creep and shrinkage);
- average daily rainfall (to correlate with shrinkage);
- seismograph in the soil (to correlate with seismic vibrations);
- wind average speed and gusts (to correlate with wind displacements).

3.3.3. Analysis of the Experimental Results

To understand the structural bridge behavior, the experimental results need to be compared with those from a numerical analysis simulating the experimental situation. Differences frequently arise and up to 10% they are reasonable. To overcome these differences, special attention need to be paid to the following points:

Numerical methods. The model should consider the real situation during testing. Sliding bearings frequently behave as fixed for the test conditions. Hand rails, barrier walls, pavement, and so forth all contribute to the bridge stiffness. Young modulus should be defined from experimental tests.

Experimental results. Various types of equipment often give wrong results (less than 10% of malfunction is usually a professional job), so redundancy should be used. In the main sections, more than one piece of equipment should read the same value. Additionally, results need to be understood considering the evolution of the environment parameters.

If monitoring does not begin with the bridge construction, it is very difficult to estimate the real stresses in the bridge due to self-weight.

Stresses must always be computed from the mechanical strains using experimental results of the Young modulus at different ages.

Strains directly obtained from monitoring need to be corrected with thermal, shrinkage, and creep effects to obtain the mechanical ones. To perform this compensation, specimens must be included in the bridge, thereby allowing the acquisition of the creep and shrinkage isolated effects. Temperature effects are obtained from thermocouples measurements.

3.3.4. Durability Parameters Monitoring

The evolution of the materials degradation is usually quite slow and unexpected sudden evolution is rare. Therefore, it is not necessary to have on line monitoring of these parameters.

In addition, it is frequently very difficult to define at the design stage the most critical points in the structure in terms of durability to expect, as these are a function of several material, geometric, and environmental parameters. If monitoring equipment is installed, there are high probabilities that the highest corrosion does not occur in the installed equipment zone.

Monitoring of the durability parameters is then usually obtained within periodic inspections, which means with a periodicity of one to two years. Nevertheless, equipment is available for online monitoring of corrosion (measuring potentials) that has been used mainly in repair situations, where the zone to be controlled is identified.

The parameters that usually are controlled in situ are the corrosion potential, the depth of carbonation, and the chloride concentration profile in the cover depth. These results are then used to update degradation evolution models and to decide on maintenance measures.

Because the models for corrosion evolution after initiation are not yet very reliable, some proof reinforcement specimens can be left in the cover, at smaller depths, and their corrosion evolution can be used as a reference for the bridge reinforcement corrosion.

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PART 2

DESIGN AND CONTRUCTION

CHAPTER 4

DESIGN FOR DURABILITY

4.1. Basis of a Durability Design

Concrete bridge design involves a set of procedures whereby a bridge is conceived to optimize the following aspects (Figure 4-1):

- *Functionality,* which is related to the traffic characteristics and pertains mainly to the definition of the deck width and eventually the height over a crossing road;
- *Safety,* which is associated with the structural safety of the bridge under traffic; the self-weight and environmental loads, which lead to the definition of the structural element geometry and its material properties;
- *Economy,* which is related to the most economic solution for the bridge to be built, in which the materials are reduced to a minimum that complies with safety conditions;
- *Aesthetics,* which is associated with the shape and color of some structural elements and bridge equipment (hand rails, cornices, etc.);
- *Environment,* which relates to a set of procedures that recently began being considered in order to analyze aspects such as protection against traffic noise and gas pollution, polluted drain water treatment, soil excavation and dragging treatment, and so forth;
- *Durability,* which until recently was considered to be related only to the adoption of minor details such as the drainage system or the deck waterproofing.

The large number of deterioration problems that occur in concrete bridges and the associated maintenance costs that authorities must expend to keep bridges safe and functional lead to the following new perceptions:

- Bridge costs are not only those incurred in building the bridge. The maintenance costs during the life of the bridge must also be included;
- If during its life the bridge needs an important repair or becomes functionally obsolete, the traffic disruption and the associated construction work are much more expensive than if the problem had been foreseen at the design stage or if it was corrected within current maintenance activities;

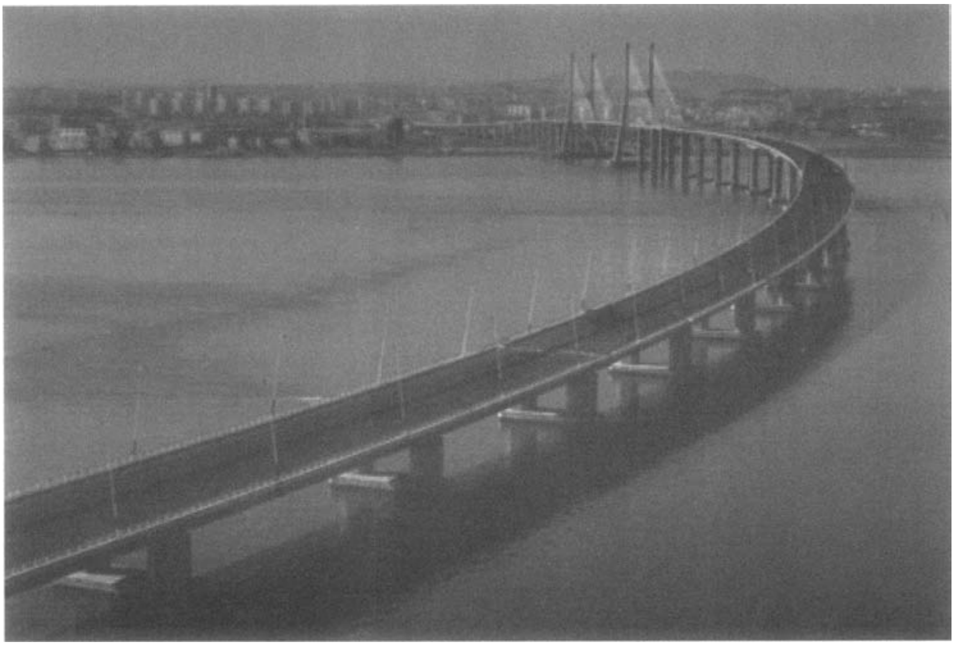

Figure 4-1. Bridge design must optimize several parameters: Vasco da Gama bridge, Lisbon, Portugal

- Designs must be developed that try to reduce all maintenance and repair work during the bridge life. The money invested in durability during design and construction leads to a reduction in the global cost of the bridge at the end of its life.

These concepts led to the development of a durability design for each bridge, in parallel with the classic structural design, and to the consideration of the whole life cost analysis in the decision-making processes (Rostam 1999; Head 1999).

Unfortunately, the durability design is not yet as straightforward as the structural design. In fact, due to the extensive research that has been developed in the past years, the main degradation mechanisms are identified, but the equations for their simulation still require more reliable calibrations.

This lack of reliability arises in part from the fact that aggressive environments are not yet well characterized, and in part because the degradation mechanisms are influenced by many factors, some of which are only empirically quantified. The degradation models have been tested with experiments that cover only one or two decades, but bridge degradation spans a much longer period of time, which frequently leads to low reliability extrapolations.

The degradation phenomena have been identified. The measures to improve bridge durability are also defined, but quantification of how much they improve durability remains imprecise. With this situation, there are two approaches to perform a durability design:

1. *By the codes*: Based on statistical analysis of existing bridges and on research on durability problems, codes (prENV 206 1999) present technical specifications. As a function of typical aggressive environments, a set of recommendations exists, related to geometry (cover, thickness of elements, etc.), material properties (strength, W/C

ratio, etc.) and construction procedures (curing, quality control, etc.), which, if implemented, will theoretically ensure a bridge service life of around 50 years.

2. *Using degradation models*: Considering a defined bridge service life and the most important degradation mechanism for the existing environment, material properties (diffusion coefficients, etc.) and geometry characteristics (cover, etc.) can be estimated to achieve a defined safety level at the end of the life. The advantage of this technique is that it can be applied to bridge lives of 100 years or more. The disadvantage is that the degradation models are not yet precise enough for 100 years previsions. Therefore, these design models must always be complemented by monitoring the de facto degradation and periodically updated, which allows timely maintenance procedures.

A durability design must be developed in parallel with the structural design, as they have interconnections that are associated with the material properties (strength and durability characteristics), geometry (cover, bar spacing, etc.), and general definition (easy inspection, drainage, etc.). The main steps to perform a durability design can be described as:

1. *Specification of a Target Service Life.* A bridge service life must be defined by the bridge owner at the beginning of the design;
2. *Identification of Environmental Aggressive Conditions.* Based on the local conditions, the main aggressive agents for concrete deterioration must be identified;
3. *Definition of Degradation Mechanisms and Simulation Models.* Based on the environmental aggressive agents, the designer identifies the possible reinforced concrete degradation mechanisms that may occur during the bridge life;
4. *Definition of Materials and Durability Parameters.* Using code recommendations or degradation models with the objective of reaching the target service life and/or degradation condition, the materials and other durability parameters are chosen;
5. *Interaction with Structural Design.* The materials and durability parameters obtained from the durability design are then used to analyze their compatibility with the structural design, eventually modifying the initial solutions;
6. *Monitoring Plan and Easy Inspection.* As the degradation models used in the durability design are still empirical, a durability monitoring plan must be defined to check whether the degradation occurs in accordance with the design models during the service life of the bridge. Furthermore, bridge design should enable an easy inspection of the main structural elements to detect eventual anomalies during the service life;
7. *Design against Water Related Problems.* Water is one of the most aggressive elements related to bridge life. A specific study related to drainage, insulation and moisture control should always be implemented;
8. *Design with Flexibility.* For those elements without degradation models (bridge equipment), in which only a replacement time can be estimated, a special design must be considered to make their replacement easier;
9. *Prevention of Animal Intrusion.* Deterioration from animal or human action must also be prevented with appropriate detailing;

10. *Technical Specifications.* They should specify the material properties related to durability, the tests to be performed, and the main construction procedures.

4.2. Bridge Owner and Service Life

One of the basic elements of a bridge design is the definition of its service life by the bridge owner. As described in Chapter 2, this service life is associated with bridge functionality, structural stability, and durability and should express the benefits that are expected from the bridge throughout its life.

Bridges are usually designed for service lives of approximately 50 years and, therefore, for this situation, codes present design actions, safety criteria, and durability requirements to be adopted at the design stage (Figure 4-2).

If the bridge owner wants a longer service life (major bridges are frequently designed to last 100 to 120 years), then design conditions related to structure and durability must be adapted (Branco 1999). For bridge equipment and other elements with shorter lives, flexibility in design must be adopted to allow for their periodic replacement.

Within structural analysis, service life value is associated with the probability of failure, leading to a quantification of the characteristic values of the actions that are considered in design.

Within a durability analysis, and considering the degradation models, the service life is frequently associated with more empirical limit states, specifically related to corrosion initi-

Figure 4-2. Degradation in a 60-year-old bridge

ation or to a certain level of corrosion in reinforcement, which can be evaluated with a deterministic approach. Nevertheless, a probability analysis of failure can also be developed using degradation models, and in this case the limit state is associated with a minimum limit value for the reliability index.

4.3. Durability Design Using Codes

4.3.1. Environmental Characteristics

The anomalies that arise in bridges during their lives are usually associated with abnormal physical actions or with the slow natural degradation of the materials.

The problems concerning physical action typically involve accidents (earthquake, flooding, foundation settlement, fire, etc.) and are usually considered within a repair structural analysis.

The normal degradation mechanisms are usually associated with the interaction between the environment and the materials and slowing down this interaction is the basis of durability design. Interactions with the environment must be identified at the beginning of the design stage and are usually related to:

- *climatic conditions*—temperature, moisture, rain, ice, solar radiation, etc.;
- *soil and air aggressive agents*—air pollution, contact with seawater, sulfates, or chlorides, etc.;
- *human actions*—deicing salts on roads, abrasion from traffic, fire, etc;
- *chemical reactions within concrete*—alkali-silica or sulfate reactions.

In reinforced or prestressed concrete structures, the most important degradation mechanisms occur in the following situations:

- carbonation of concrete;
- chloride attack in a saline environment;
- freeze/thaw cycles;
- chemically aggressive salts.

In these situations, the following environment classification can be adopted (prENV 206 1999):

- No risk of degradation for concrete without reinforcement and the nonexistence of freeze/thaw, abrasion, or chemical attack conditions for a very dry embedded reinforcement environment;
- Corrosion induced by carbonation (Table 4-1);
- Corrosion induced by chlorides other than in seawater (Table 4-2);
- Corrosion induced by chlorides from seawater (Table 4-3);
- Freeze/thaw cycles (Table 4-4);
- Chemical attack (Table 4-5).

Table 4-1. Classes of environments with corrosion due to carbonation

Class	Environment	Example
XC1	Dry or permanently wet	Inside water or dry
XC2	Wet	Long water contact, foundations
XC3	Moderate humidity	External concrete, sheltered
XC4	Cyclic wet and dry	Water contact

Table 4-2. Classes of environments with corrosion due to chlorides other than in seawater

Class	Environment	Example
XD1	Moderate humidity	Air chlorides exposition
XD2	Wet, rarely dry	Industrial waters, swimming pools
XD3	Cyclic wet and dry	Chlorides spray, pavements

Table 4-3. Classes of environments with corrosion due to chlorides from seawater

Class	Environment	Example
XS1	Airborne salts	Near coast
XS2	Permanently submerged	Marine structures
XS3	Tidal, splash zone	Marine structures

Table 4-4. Classes of environments with freeze/thaw cycles

Class	Environment	Example
XF1	Moderate saturation, no deicing	Vertical surface with rain, freezing
XF2	Moderate saturation, deicing	Vertical surface with airborne deicing
XF3	High saturation, no deicing	Horizontal surface with rain, freezing
XF4	High saturation, deicing	Bridge decks, marine splash zones

Table 4-5. Classes of environments with chemical attack

Class	Environment
XA1	Slightly aggressive
XA2	Moderately aggressive
XA3	Highly aggressive

Table 4-6. Characteristics of classes of environments with corrosion due to chemical attack

Chemical element	XA1	XA2	XA3
Groundwater			
SO_2 (mg/l)	20–600	600–3,000	3000–6000
pH	<6.5–>5.5	<5.5–>4.5	<4.5–>4.0
CO_2 (mg/l)	15–40	40–100	>100
NH_4 (mg/l)	15–30	30–60	60–100
Mg_2 (mg/l)	300–1,000	1,000–3,000	>3,000
Soil			
SO_2 (mg/kg)	2,000–3,000	3,000–12,000	12,000–24,000
Acidity (ml/kg)	>2,000		

In this case, the classes are characterized by the limit values of the chemically aggressive elements presented in Table 4-6.

4.3.2. Code Recommendations

Choice of material. Based on environmental condition classification, codes present a set of recommendations to be adopted in the durability design, mainly related to concrete composition (prENV 206 1999, LNEC E378 1993, CHBDC 1995). In the following tables, the variation of the recommended cover was obtained from those references and is only related to reinforcement steel.

- Corrosion induced by carbonation (Table 4-7)
- Corrosion induced by chlorides from seawater (Table 4-8)
- Corrosion induced by chlorides other than in seawater (Table 4-9)
- Freeze/thaw cycles (Table 4-10)
- Chemical attack (Table 4-11)

These recommendations show that for the same bridge different environmental conditions can be considered. In the case of precast bridges in cold countries, the deck slab can be associated with corrosion from chlorides other than in sea water, and the beams are associated with corrosion by carbonation.

Structural design. In addition to these recommendations concerning the materials, structural design codes also present durability recommendations mainly associated with crack width control (Litzner 1999, ENV1992-1 1999).

Cracks may be caused by the rheologic behavior of fresh concrete, by external loading, or by imposed deformations. To prevent cracks of the first type, correct construction procedures should be adopted, specifically those associated with curing and materials properties (prENV 206-1 1999). For cracks of the second type, two basic requirements are adopted:

- minimum steel reinforcement;
- limitation of crack width.

Table 4-7. Specifications for classes of environments with carbonation

Recommendations	XC1	XC2	XC3	XC4
Maximum w/c ratio	0.65	0.6	0.55	0.5
Minimum strength class (MPa)	C20/25	C25/30	C30/37	C30/37
Cement content (kg/m^3)	260	280	280	300
Minimum cover (mm)	20–40	25–60	25–60	25–60

Table 4-8. Specifications for classes of environments with chlorides from seawater

Recommendations	XS1	XS2	XS3
Maximum w/c ratio	0.5	0.45	0.45
Minimum strength class (MPa)	C30/37	C35/45	C35/45
Cement content (kg/m^3)	300	320	340
Minimum cover (mm)	40–60	40–60	45–70

Table 4-9. Specifications for classes of environments with chlorides other than in seawater

Recommendations	XD1	XD2	XD3
Maximum w/c ratio	0.55	0.55	0.45
Minimum strength class (MPa)	C30/37	C30/37	C35/45
Cement content (kg/m^3)	300	300	320
Minimum cover (mm)	40–60	40–60	45–70

Table 4-10. Specifications for classes of environments with freeze/thaw cycles

Recommendations	XF1	XF2	XF3	XF4
Maximum w/c ratio	0.55	0.55	0.5	0.45
Minimum strength class (MPa)	C30/37	C25/30	C30/37	C30/37
Cement content (kg/m^3)	300	300	320	340
Minimun air content (%)		4	4	4
Minimum cover (mm)	40–60	40–70	40–70	40–70

Table 4-11. Specifications for environments with chemical attack

Recommendations	XA1	XA2	XA3
Maximum w/c ratio	0.55	0.5	0.45
Minimum strength class (MPa)	C30/37	C30/37	C35/45
Cement content (kg/m^3)	300	320	360

Minimum steel reinforcement should ensure a balance between the tensile stresses and prevent cracks in young concrete resulting from hydration heat, shrinkage, etc.

Related to crack width, codes impose limit states of decompression (which is especially important for prestressed precast beams) or maximum crack width (w_k = 0.2 mm or w_k = 0.3 mm), as a function of load level, environmental actions, the risk of deterioration, and the type of the structural material (prestressed or reinforced concrete). The corresponding load levels are associated with the probability of being quasi-permanent (exceeded during 50% of the structure service life), frequent (exceeded during 5% of the same period), or rare (the characteristic load values).

4.4. Durability Design with Degradation Mechanisms

4.4.1. Degradation Models

There are no code recommendations for major bridges for which greater service lives are required (100–120 years). In such cases, the study of the service life must be performed in terms of physical deterioration, based on local environmental conditions and on the limit conditions adopted for design and by using mathematical models for deterioration and/or available local experience (Sarja 2000).

These models have been developed mainly for carbonation and chloride attack prediction, but there are simplified models to simulate other degradation mechanisms such as sulfate attack or leaching (Clifton 1993).

Carbonation simulation. Carbonation of concrete is caused by the reaction of CO_2 in the atmosphere with Ca $(OH)_2$ (contained in the cement hydration products) in the presence of water. The result is a loss in the alkalinity of the concrete cover, the pH of which decreases from concrete initial values (pH around 12.5), to values for which corrosion can be initiated (pH ca. 9). The penetration of CO_2 into concrete pores tends to move as a front that proceeds at a rate controlled mainly by the CO_2 diffusion coefficient (Clifton 1993, Mangat 1991). The evolution of the depth of carbonation d_c can be estimated by:

$$d_c = K\,t^{0.5} \tag{4-1}$$

where

K = carbonation coefficient (mm/year$^{0.5}$)

d_c = carbonation front depth (mm)

t = time (years)

The carbonation coefficient depends on the effective diffusion of CO_2 through the concrete and on the environment and in-depth concentrations of CO_2. Based on experimental data, the parameter K has values around 1.0 to −1.5 (mm/year$^{0.5}$) for good concrete and current situations, but can increase to 7.0 to 8.0 (mm/year$^{0.5}$) for poor concrete and industrial environment situations (Mangat 1991).

Chloride attack simulation. Fick's law of diffusion can reasonably predict chloride penetration into concrete. The solution of this differential equation was adapted to take into account the time dependence of the chloride diffusion coefficient $D_c(t)$ (Mangat 1994):

$$D_c(t) = D_{ci}\,t^{-m} \tag{4-2}$$

where

D_c = diffusion coefficient at time t (cm^2/s)

D_{ci} = initial diffusion coefficient (cm^2/s)

t = duration of exposition (s)

m = empirical constant

This variation with time leads to the following penetration equation, giving the chloride concentration $C(x, t)$ within concrete at any depth x and time t:

$$C(x, t) = C_s\,(1 - erf\,(x/(2\,(D_{ci}\,t^{(1-m)}/(1-m))^{0.5})) \tag{4-3}$$

where

D_{ci} = initial diffusion coefficient (cm^2/s)

$C(x, t)$ = chloride concentration at distance x as % of weight of cement (kg/m^3)

C_s = surface chloride concentration at as % of weight of cement (kg/m^3)

x = distance from surface (cm)

t = duration of exposition (s)

m = empirical constant

In this equation, *erf* is the error function and m is an empirical constant to take into consideration the variation of D_c with time (usually m has values between 0.4 and 0.6) (Mangat 1991, Mangat 1994).

The preceding equation can be used to estimate the time for the corrosion initiation for a reinforcement cover x, considering the surface chloride concentration C_s (Table 4-12) (Mangat 1991), using the concrete chloride diffusion coefficient (from experimental data) and considering that corrosion initiation in reinforcement usually occurs for critical values around C_{crit} = 0.4% (of weight of cement) (prENV206-1 1999). Although this is the value usually adopted, it can be shown that in situ measurements have obtained values for C_{crit} varying between 0.2 and 2.0 (Salta 1999).

Corrosion propagation. After initiation, the evolution of the bars diameter D_t (at time t) due to corrosion can be estimated by (Mangat 1991, Andrade 1993)

$$D_t = D_i - 0.023 \cdot t \cdot I_c \tag{4-4}$$

Table 4-12. Typical surface chloride concentrations

Structure	Environment	C_s (%)
Bridge deck	Air zone	1.6
Bridge column	Splash zone	2.5
Bridge column	Tidal zone	5
Bridge deck	Deicing salts	1.6
Bridge column	Deicing salts	5

where

D_t = bar diameter at time t (mm)

D_i = initial bar diameter (mm)

t = time after corrosion initiation (year)

I_c = corrosion rate ($\mu A/\text{cm}^2$)

The corrosion rates that have been measured are affected by several factors, from humidity to aggressive agents' concentration, leading to the variable values expressed in Table 4-13 (Gonzalez et al. 1996).

The great variation in these values makes the estimation of the corrosion evolution difficult without in situ measurements.

Cracking and cover spalling. Several studies have been developed to obtain models to simulate the cracking of concrete after the initiation of reinforcement corrosion (Noghabai 1999; Liu 1998) (Figures 4-3 and 4-4). These methods take into account the amount of corrosion produced around the bars and analyze, in accordance with the theory of elasticity, the internal pressure induced in concrete until cracking occurs. To estimate the length of time after initiation before cracking occurs, Liu (1998) proposes the following equation:

$$t_{cr} = W_c^2/2\,k_p \tag{4-5}$$

where

t_{cr} = time after initiation before cracking occurs

W_c = critical amount of corrosion products to induce cracking

k_p = parameter associated with the rate of metal loss

$$k_p = 0.098\ (1/\alpha)\ \pi\, D\, I_c \tag{4-6}$$

where

D = bar diameter (cm)

I_c = corrosion rate ($\mu A/\text{cm}^2$)

α = coefficient with values between 0.5 and 0.6, as a function of the rust composition

Table 4-13. Average corrosion rates

Situation	I_c (μA/cm^2)
Passive situation	<0.1
Low corrosion	0.1–0.5
Moderate corrosion	0.5–1.0
High corrosion	>1.0

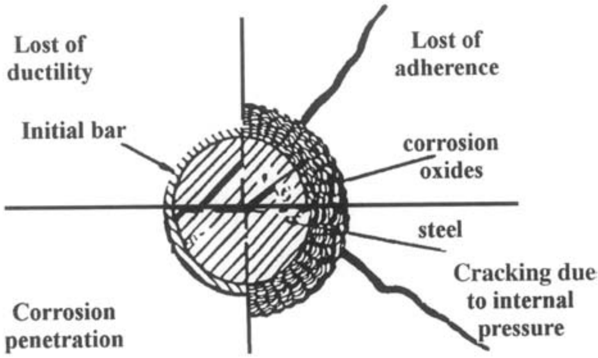

Figure 4-3. Crack around a bar

$$W_c = \rho_r \left(\pi \left(d_s + d_o\right) D + W_{st}/\rho_{st}\right) \tag{4-7a}$$

where

$$d_s = C f_{ct} \left(\left(\left(a^2 + b^2\right)/\left(b^2 - a^2\right)\right) + \nu_c\right)/E_{ef} \tag{4-7b}$$

$$a = \left(D + 2\ d_o\right)/2;\ b = C + \left(D + 2\ d_o\right)/2 \tag{4-7c}$$

where

W_{st} = amount of corroded steel ($\cong \alpha . W_c$)

ρ_r = density of corrosion products ($\cong$3600 kg/m^3)

ρ_{st} = density of steel ($\cong$7800 kg/m^3)

d_o = thickness of pore zone concrete-steel ($\cong$12 μm)

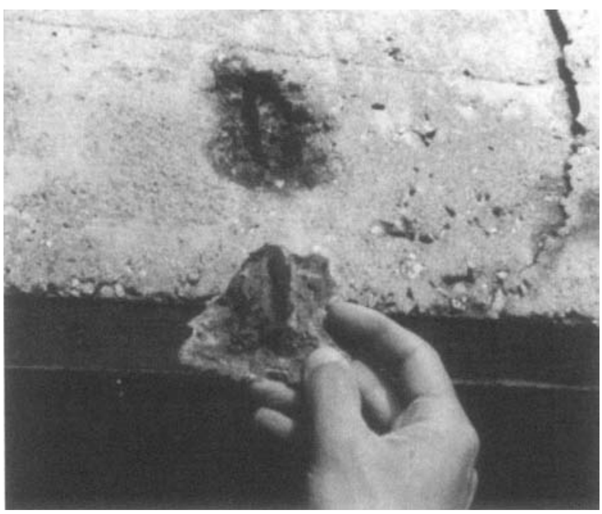

Figure 4-4. Corrosion and spalling of concrete

d = reinforcement cover (m)

f_{ct} = tensile strength of concrete (MPa)

ν_c = Poisson ratio of concrete ($\cong$0.2)

E_{ef} = effective Young modulus of concrete ($E_{ef} = E_c/(1 + \phi)$)

E_c = concrete elastic Young modulus (MPa)

ϕ = creep coefficient ($\cong$2.0)

4.4.2. Deterministic Analysis

Based on the preceding models, the duration of service life can be estimated by assuming materials properties and defining a limit state. With these models, three types of limit situations can be analyzed:

- corrosion initiation
- cracking initiation
- percentage of corrosion of a bar

If corrosion initiation is the limit state and carbonation is the main degradation mechanism, at the end of service life (t_u years) the carbonation depth should reach the reinforcement cover depth d_c, so concrete should have a carbonation coefficient K with the value

$$K \leq d_c / t_u^{0.5} \tag{4-8}$$

If chloride attack is the main degradation mechanism, however, at the end of service life (t_u years) the chloride concentration $C(d_c, t_u)$ should be 0.4% at the reinforcement cover depth ($x = d_c$). The solution of the diffusion equation will provide a value for the diffusion coefficient D_{ci}. This is the critical chloride concentration that the concrete should have to reach the end of the service life.

The preceding analyses, which consider the initiation of corrosion with regard to the end of service life, are in fact conservative hypotheses, because some time will still elapse from initiation until a significant loss of the bar's cross section occurs.

Alternatively, the analysis that considers the beginning of cracking as a limit for the service life may be a more realistic hypothesis, specifically for prestressed structures, because this means that corrosion in reinforcement bars has already occurred, and that it will soon begin in prestressed tendons with a deeper cover. Nevertheless, this may not apply to precast beams in which prestress cover is frequently quite reduced.

These analyses may be complemented by a limit state for reinforcement bar corrosion percentage (use of the propagation rate model), which can be defined by a structural safety analysis of the main cross sections. For example, a bar corrosion percentage limit can be defined, based on the required reinforcement for an assumed reduction of safety factors for materials γ_m or actions γ_a at the end of the service life (Chapter 2).

These types of analysis have led to the following examples of materials specifications for major bridges with expected service lives of about 100 years (Gonçalves 1999) (Tables 4-14 and 4-15).

Table 4-14. Concrete specifications for 100-year-old bridges

Bridge	Maximum w/c ratio	Cement (kg/m^3)	Fly ash (% cement)	Silica fume (% cement)	Strength (MPa)
Joigny	0.36	450			65
Pertuiset	0.38	400		8	80
Rance	0.37	400		8	60
Lacey Murrow	0.33	380	20	10	70
Helgeland	0.36	420		10	60
Boknasundet	0.35	400		8	60

Related to steel elements and equipment of a bridge, they should be galvanized to provide maintenance operations with a periodicity varying between 5 and 20 years.

For structural elements made of unprotected steel, the service life can be estimated based on corrosion rates. To achieve the expected lifespan in these elements, an additional thickness must be adopted to take into account the corrosion depth lost during the service life (Figure 4-5). Table 4-16 presents typical corrosion rates in a maritime environment for unprotected construction steel.

From the preceding estimations of the service life at the design stage, a few conclusions must be pointed out:

- The existing models are still imprecise but are nevertheless useful in developing sensitivity analyses, which help in making decisions concerning material properties;
- The concept of limit state due to durability is not yet well defined, so the authorities must decide what to adopt, or at least be aware of the assumptions that can be considered during design;
- The use of these models during service life and the updating of the adopted parameters with in situ measurements is a powerful tool in the reduction of maintenance costs and also yield a sufficiently precise estimate of the bridge degradation.

Table 4-15. Concrete characteristics for 100-year-old bridges in saline environment

Structure	Diffusion Coefficient ($\times 10 - 12\ m^2/s$ at 28 days)	AASHTO (Coulombs)	Cover (mm)
Great Belt link	0.6		
Aftroll platform	0.9		
Oresund		1,200	
Lacey Murrow		1,300	70
V. Gama bridge (immersion)	5.0	3,000	70
V. Gama bridge (splash)	2.0	1,500	70
V. Gama bridge (sea air)	5.0	3,000	50

Figure 4-5. Steel elements in a marine environment, South viaduct, Alcochete, Portugal

4.4.3. Reliability-Based Methods

To overcome the problem of the ultimate limit related to an analysis considering degradation models, reliability-based methods have been developed to compute the evolution of the reliability index of the bridge during its lifespan. The end of that lifespan occurs when the reliability index reaches a limit value β_{lim} (Thoft-Christensen 1998; de Brito, Branco et al. 1997; Gehlen 1999).

Reliability-based methods have been applied mainly to current bridges and can consider several limit states (shear failure, bending failure, crack width, deflection limit, etc.). For each failure mode, it is assumed that it is possible to formulate a failure function:

$$g\,(x,\,t) = g\,(x_1,\,x_2,\,\ldots\,x_n,\,t) \tag{4-9}$$

in which t is the time and x is a realization of the basic stochastic variables modeling the uncertain quantities (loads and strengths).

Table 4-16. Steel corrosion rates in marine environments (Branco 1996)

Location	μm/year
Buried zone	0.01
Submerged mud zone	0.1
Fully submerged zone	0.08–0.12
Intertidal zone	0.1–0.2
Splash zone	0.2–0.4
Air zone	0.1–0.2

Failure function is defined so that a positive value of g indicates a safe set of basic variables and a negative value of g indicates a failure set of basic variables. For time invariant reliability problems, the probability of failure P_f of one failure mode in the interval $(0, t)$ is then estimated by:

$$P_f(t) = P\,(g\,(x,\,t) < 0) \cong \Phi\,(-\beta\,(t)) \tag{4-10}$$

in which Φ is the distribution function for a standardized normal distributed stochastic variable and $\beta\,(t)$ is the reliability index.

The evolution of the failure modes is associated with a reduction of the reinforcement cross-section due to corrosion. The failures are modeled as elements in a series system such that the structure is assumed to fail if any of these failure modes is reached.

To estimate the deterioration of the reinforcement with time, the equations indicated in 4.4.2 are used and are related to the initiation time for chloride attack and carbonation, as well as to the rate of corrosion during the propagation period. In these equations, some of the parameters are considered as stochastic variables, and are expressed by assumed statistical distributions. Among these are the chloride diffusion coefficient, the carbonation coefficient, the external chloride content, the corrosion rate, the reinforcement cover, and so forth.

With this type of analysis, the evolution of the reliability index $\beta(t)$ with time can be estimated (Figure 4-6). The initial reliability β_0 remains constant during the initiation period and begins to decrease at time t_i. The service life is associated with reaching the minimum acceptable value β_{lim}.

The estimation of the bridge service life using reliability methods considers a reliability analysis with only the main failure situations, and also needs to assume statistical distributions for several variables from the degradation mechanisms. Its main advantage is that it allows sensitivity studies of the materials parameters, thus allowing decisions to be better grounded. Studies to estimate increased service life based on a reduction in the chloride diffusion coefficient of concrete, or the rate of corrosion, can easily be performed.

After the design stage, the reliability analysis can also be updated, during service life, with experimental measurements of the stochastic variables, leading to a more precise analysis of the evolution of the reliability index, and allowing for an optimized maintenance program.

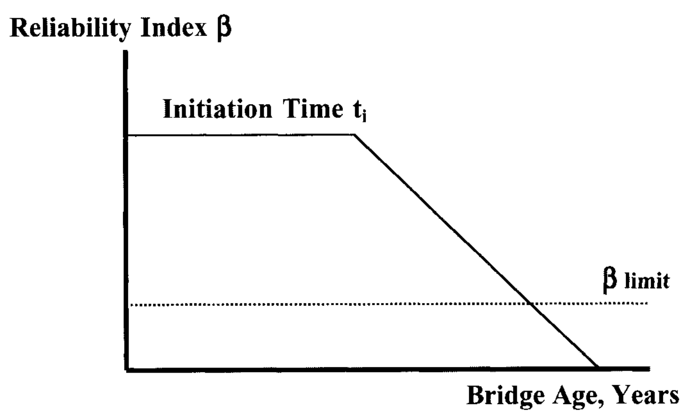

Figure 4-6. Typical evolution of the reliability index during bridge life

4.5. Interaction with Structural Design

Bridge design begins with a preliminary definition of the bridge geometry, which is usually determined by fulfilling functional and structural safety requirements. Even at this stage, the designer must be aware of the possible considerations that will arise from the durability design of bridge details to achieve the following:

- Easy periodic inspections (access to abutments or box girders, height of box girders, etc.);
- Placement of monitoring and other equipment (existence of ducts, space for the equipment, etc.);
- Flexible design, such as the periodic change of bridge equipment (bearings, joints, etc.) or additional strengthening (external prestress).

The detailed bridge structural analysis is performed afterwards and basically consists of assuming a structural numerical model with geometric and material characteristics and subjecting that model to the design actions estimated for the structure during its life. Based on the action effects obtained from the numerical model, the safety requirements are checked taking into account the materials strength and the geometry initially adopted.

When a durability design is performed, some of the durability parameters can interact with the structural design. These are:

Geometry parameters: reinforcement cover, bar diameter and spacing, thickness of elements;

Materials properties: concrete strength;

Safety analysis: crack width control.

This interaction leads to an iterative study in which the structural analysis parameters must be checked with the durability parameters. If they do not agree, a reanalysis must be performed. In practical terms, it is advisable to begin by defining the parameters for the durability analysis and then using them as reference limit values for the structural analysis.

4.6. Complementary Reinforcement Protection Methods

In aggressive environments, the initiation of corrosion is a matter of time and a function of the period needed for chlorides or CO_2 to reach the reinforcement. In addition to the adoption of all the design measures discussed previously for a durability design, other complementary reinforcement protections can be used (Gonzalez et al. 1996, Salta 1999).

Some of these measures can be applied directly to the reinforcement:

Cathodic protection. This is the only technique that effectively stops corrosion after it begins, despite the number of aggressive elements. Disadvantages include the need for a skilled staff, continuous monitoring, and energy (Daily 1999). Currents between 2 and 20 mA/m^2 must be applied as a function of the corrosion degree. It should be decided at the design stage whether to use cathodic protection during construction, since a good connection among reinforcement bars must be guaranteed.

Problems may arise when using this system, such as an increase in alkali-silica reactions or the loss of the adherence and/or the ductility of the reinforcement due to hydrogen production close to the steel.

Galvanization. Galvanization of the reinforcement is an old technique for protecting the steel bars. The zinc coating increases the corrosion resistance, but in practice it seems that it leads only to a small delay in the development of corrosion. It is easy to apply, especially in precast structures, at low costs and with no maintenance required.

Epoxy paints. Epoxy coating is used quite often because of its good adherence to steel and concrete and its low permeability to chloride ions. Its long-term performance has not yet been established with significant reliability.

This system does not require maintenance, but the implementation costs are high and it usually has been used for precast elements in which the coating is applied by immersion.

Use of noncorrosive reinforcement. Stainless steel has been used as an alternative for concrete reinforcement. It has a much longer lifetime, but some care must be taken with regard to concrete-steel adherence. The cost is much higher than for mild steel.

Glass (gold type) and carbon tendons have also been used for concrete reinforcement as a means of corrosion prevention (Sieble 1999, Ferreira 2000). These materials have also been used in reinforcement nets for controlling cracking in very deep reinforcement covers as an alternate solution for increasing durability.

Other complementary measures can also be applied to concrete. Among these, are:

Corrosion-inhibiting additives. The addition of a corrosion inhibitor, such as nitrite ion (calcium nitrite), has appeal as a mean of preventing corrosion in an aggressive environment. It is still unclear whether it is effective in an environment in which leaching of the inhibitor occurs.

The additive is easily applied at a reasonable cost, and no maintenance is necessary. Optimal percentage calibration requires further research. The toxic hazards of nitrites necessitate special attention during application.

External protection. A variety of paints and membranes can be applied externally to the concrete surface to reduce the penetration of aggressive agents. These products may act as water repellents (silicone type), which prevent saltwater but not CO_2 from penetrating, or as a coating with a thickness of between 0.1 and 5.0 mm, thus preventing the penetration of chlorides or CO_2 (EN 1504 1996).

None of these measures will stop corrosion after it has begun, but they will inhibit further penetration. Costs may be high and periodic maintenance may be needed to maintain the protective coating.

4.7. Protection Against Water

Water, whether from snow, rain, or high humidity, is the main enemy of bridges, because it is one of the major material degradation mechanisms. Some of the most common defects caused by water because of deficient design measures are indicated below (Godart 1999):

- lack of waterproofing membrane on the deck (Figure 4-7);
- lack of waterproofing membrane under the sidewalks;
- defective waterproofing in expansion joints;

Figure 4-7. Lack of waterproofing

- water drainage defects at the bridge ends;
- unsuitable gutters in cornices;
- absence of drips;
- water accumulation on the deck;
- defective waterproofing next to equipment.

The other defects caused by water are essentially related to deficient construction control:

- defects in the waterproofing membrane;
- abnormal rising of the membrane;
- leakage between precast cornice units;
- water drainage defects under the deck;
- defective connections of drainage devices;
- leakage in water and construction joints;
- treatment of anchorage zones.

The main measures for overcoming water defects are improved drainage and insulation with measures such as those that follow.

4.7.1. Drainage

Drainage is necessary to take water out of the bridge and can be achieved by positioning drains along the bridge deck so that the water runs into the soil below the deck or is moved to a sewer system using pipes.

The pavement slope and the openings must be correctly spaced to allow a good rate of water flow and thus prevent ponding. The following aspects should be considered, as they usually lead to maintenance problems.

1. Cantilever drainage. Side cantilevers and handrail supports should be designed with gutters and drips to prevent water from coming into contact with the structure.
2. Preventing water contact with structure. Systems that drop the water under the bridge should be extended for a sufficiently long length (>15 cm) to prevent water from coming into contact with structural elements, as it falls (Figure 4-8).
3. Adjacent road drainage. Bridges usually are in valleys and also receive water from adjoining roads. The water should be drained before reaching the bridge as well as in the abutments to ensure collection of the run-off from the joints and prevent contact with the bearings.
4. Pipes that allow an easy cleaning. Pipes should be designed so that they can be opened for cleaning, especially in curved or intersection zones (Figure 4-9).
5. Bridge drainage connected to road drainage. In viaducts, water should be drained to the nearest road drainage system or to an area where it does not provoke soil erosion, especially close to the bridge foundations.
6. Environmentally protected zones. The water from drainage contains polluted elements as it washes off the bridge deck. In environmentally sensitive zones, the water should be drained to settlement reservoirs.

Figure 4-8. Inexistence of pipe

Figure 4-9. Pipes allowing easy cleaning

4.7.2. *Waterproofing*

The first barrier to water reaching the deck is the bituminous pavement, which is good waterproofing under normal conditions.

As a result of pavement deformation or cracking over a period of time, water frequently reaches the structural deck. A second barrier may than be used to insulate the concrete slab of the deck. This is especially important in cold countries where deicing salts are frequently used to prevent the road from freezing. The penetration of these salts is highly corrosive to the deck, so waterproofing protections must be adopted. Several commercial products can be applied to the deck. The following properties are important in making a choice: insulation, adherence, elasticity and durability.

Waterproofing under sidewalks and close to handrails must also be considered, because frequently no bituminous protection is applied in these areas, which are close to the draining water, thus leading to easier corrosion. Insulation protection around bridge equipment should also be adopted.

4.7.3. *Moisture Control*

Most of the degradation problems that occur inside concrete depend on the existence of moisture, specifically chloride transportation and the development of corrosion in reinforcement.

Corrosion in steel requires moisture levels above 50%. This fact led to techniques of corrosion prevention associated with moisture control in steel box girders and in cables from suspension and cable-stayed bridges, with successful results. These techniques require the continuous piping of dry air into the area under control with dehumidification systems working permanently.

In concrete structures, the classic way to reduce moisture inside concrete is the use of paint, which provides good insulation.

4.8. Design with Flexibility

Flexible solutions in terms of traffic volume were described in Chapter 2, where solutions considering an increase in the number of traffic lanes for the same deck width, or the adoption of a reverse lane according to the rush hour, were discussed.

In terms of structural behaviour, the global service life defined for a bridge is expected to be accomplished in most structural elements with minor repair costs. However, each bridge nonstructural component will have a different service life, frequently less than the global service life, which will lead to the necessity of repairs during the structure's life. Bearings, joints, handrails, drainage systems, electrical systems, and so forth are elements with typically shorter lives. Analysis of the durability of these elements should also be performed at the design stage, defining replacement periods and degradation monitoring parameters.

All components that will require repair or replacement during the global service life should be designed with flexibility, which means that their replacement and/or repair should be conceived so that they can be performed with minor effects on bridge operation. Some examples of these design concepts are described.

Additional prestress. The creep and shrinkage effects in bridges or traffic increases are factors that sometimes make it necessary to strengthen the deck. This should be considered during the design phase, in which case empty ducts for additional prestress or cells for external prestressing are left, which will be applied during service life when necessary.

Replacement of bearings. The replacement of bearings is frequently performed with the help of temporary hydraulic jacks between the deck and the top of the columns. The bearings should be designed with geometry based on bearing area and free height, thus allowing for the introduction of the jacks at the tops of the columns (Figure 4-10). The placement of clamps in the deck, close to the bearings, to facilitate lifting the equipment is frequently useful.

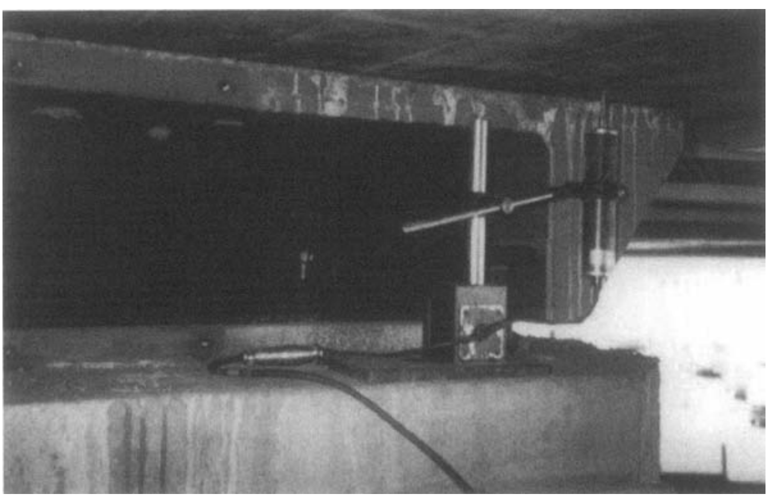

Figure 4-10. Bearing prepared for replacement

Replacement of joints. This operation always interferes with traffic. If joints are built with separate elements per lane, it will allow easier sequential replacement with less interference. The existence of an inspection corridor inside the abutments also facilitates the replacement operation.

4.9. Design for Easy Inspection

The access to the bridge elements during inspection is easily guaranteed in the upper part of the deck but frequently requires special equipment to reach the components below the deck. In the latter case, special trucks with mobile platforms are frequently used for decks that are high above the ground. Alternatively, trucks with elevator platforms may be used in decks with heights up to around 8 m.

Some bridge areas may have easy access during bridge service life, if special details are considered during design, such as those that follow (Santiago 1999).

Interior of box girders. Access to the insides of box girders should be foreseen, usually by placing a door in the abutments (Figure 4-11). Typically, box girders should have a minimum interior height of 15 m (preferably 1.8 m) to allow easy access. Electrical lighting should also be available. Steps are frequently made in the intermediate diaphragms to facilitate going through them. At intermediate supports, some visual access to the bearings should also be considered.

Abutments, bearings and joints. The design of a corridor with a width of at least 1.2 m between the end of the deck and the abutment wall is a solution that allows an easy visual inspection of joints, abutment drainage, and abutment bearings, and also provides access to the interior of box girders. The access to the corridor must be closed with a door in the lateral side of the abutment, preferably at a height that a portable ladder can reach. The access can also be placed in the bridge deck using a closed door.

Hollow columns, bearings and joints. Hollow columns should have a door close to the ground to provide access to their interiors and stairs that allow access to the top bearings. In long span bridges, towers should have internal elevators.

Under the deck. In long span bridges, permanent mobile scaffolds, suspended from the deck, can be used to perform inspections. This solution has the advantage over the mobile inspection trucks of not interfering with traffic and it is not limited by the width of the bridge.

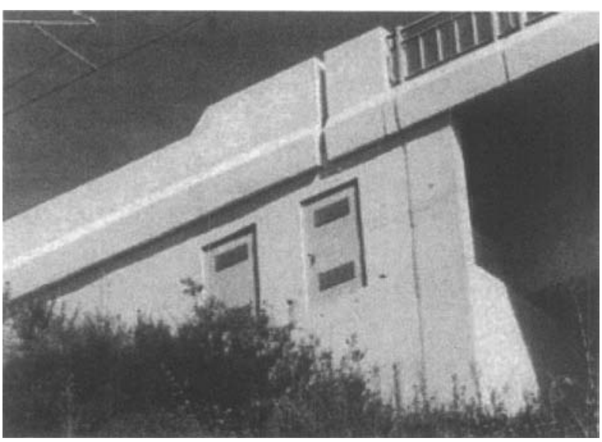

Figure 4-11. Access door to the girder

Figure 4-12. Mobile scaffold truck

Mobile scaffolds can also be adapted to inspect long viaducts. The mobile scaffold is put in place only during inspection. In this case, the definition of travelling suspension rails and a solution for allowing passage over (or between) intermediate columns must be taken into account during the design stage.

The alternative consists of using truck mobile scaffold, thus allowing for inspection from the bridge border (Figure 4-12). Attention must be paid to all existing equipment near the border, which can make access of the truck difficult (Figure 4-13).

Drainage pipes. Drainage pipes should be designed to allow interior inspection and cleaning, especially in curved stretches (Figure 4-9).

4.10. Design to Prevent Human and Animal Intrusion

4.10.1. Human Intrusion

"To sleep under the bridge" is a popular expression related to poor people who unfortunately are often found in urban areas. They try to sleep in the upper part of the abutment earth fill, under the bridge deck, or in a box beam deck.

The main problems associated with having people living in this manner are related to the garbage they leave, which frequently affects the behavior of the bearings and drainage systems. They also start bonfires that may lead to damage to structural elements.

The access to the interior of the box beams should be made difficult, by making a ladder necessary and using the protection of a closed door (Figure 4-14). To make the area under the bridge uncomfortable, aesthetically irregular devices should be placed there, and signs forbidding bonfires should be conspicuous.

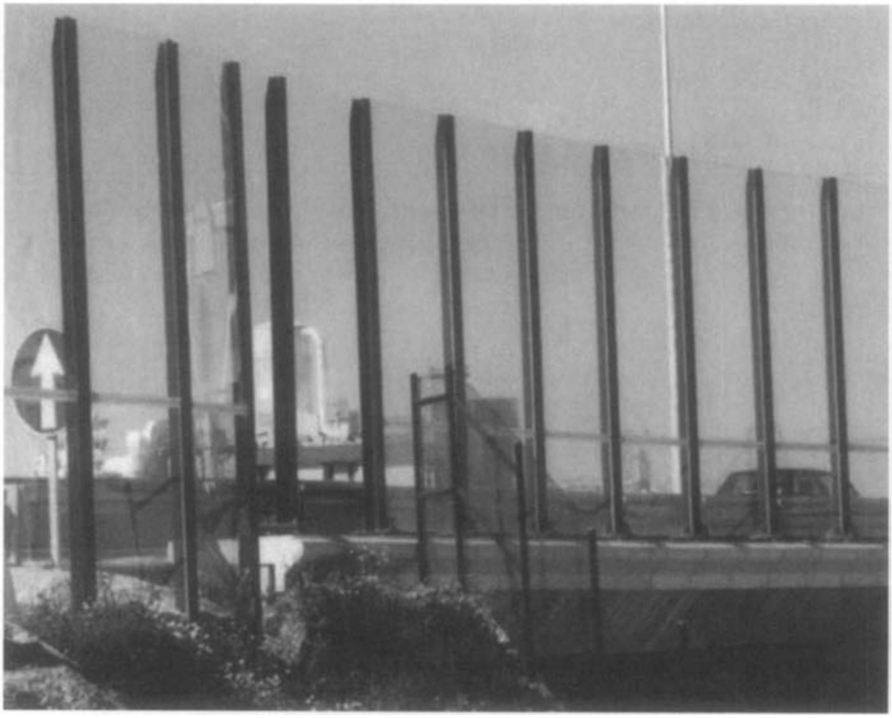

Figure 4-13. Obstructions for a mobile scaffold

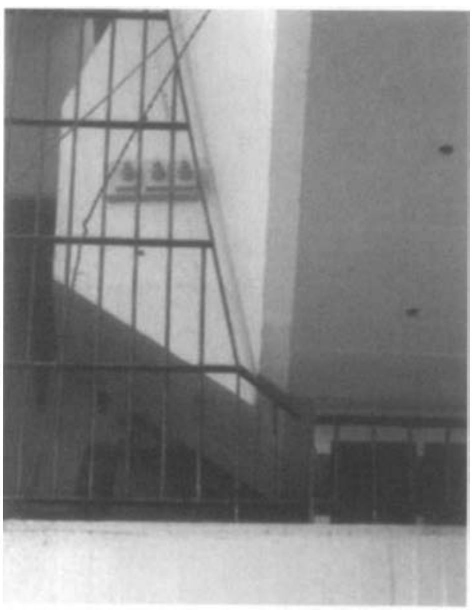

Figure 4-14. Protected access to an abutment

4.10.2. Animal Intrusion

The use of bridges to live in is also very frequent among birds and bats in both urban and rural areas. They become accustomed to the traffic noise and use either the interior of the box beams (especially bats, because it is dark) or the beams to build their nests.

The problems resulting from these intrusions also arise from accumulated debris, which leads to problems in the drainage system or even the bearings. Additionally, animal excrement chemically attacks construction materials (provoking corrosion in steel). Their actions may also break connection cables, leading to malfunction of the electrical and monitoring systems.

To prevent the problems caused by birds and bats, all the holes that allow access to the interior of box beams should be protected with nets. The nests that are built in the outer surfaces of the bridge must be eliminated during each inspection.

4.11. Monitoring and Maintenance Plan

As discussed in Chapter 3, the objective of bridge monitoring, which is implemented in major bridges, is to check the behavior of the bridge, throughout its life, by controlling the evolution of several parameters. The measured values of these parameters are compared with the design assumptions and results, based on which predefined activities will be implemented. These measurements allow, for instance, the permanent updating of the expected service life.

These monitoring activities should be considered during the design stage, defining levels of action for each parameter, as a function of the measured value. The parameters measured in major bridges are mainly related to the environment, structural behavior, and durability (Figure 4-15).

For environmental characteristics, the following variables can be measured:

- wind speed;
- air temperature;

Figure 4-15. Bridge monitoring central station

- humidity;
- rainfall;
- earthquake acceleration;
- wind speed;
- river water level/speed.

For structural behavior, the following variables associated with the global behavior of the bridge can be measured:

- displacements (in bearings, joints and some structural sections);
- rotations (in some columns);
- concrete temperatures (average and differential values in main cross sections);
- strains (maximum tensile and compression values in main cross-sections);
- vibrations (at main span, tops of columns and cables).

Related to structural behavior, but associated with the evolution of concrete properties, the following variables can be measured:

- creep evolution in test specimens;
- shrinkage evolution in test specimens;
- concrete compressive strength in cores.

The parameters associated with durability that can be controlled are:

- carbonation depth (in several points from the main structural elements);
- chloride concentration profiles (at several points from the main structural elements);
- diffusion coefficient from cores in some elements;
- humidity in concrete surface.

The decision as to exactly what is to be measured, where the measurements should take place and which test and/or equipment to use to make the measurements, as well as what levels of action to take as a result of the measured values obtained, should be accomplished during the design phase, based on considerations such as bridge importance, the structural analysis of the bridge, and the possible degradation mechanisms.

Using these measurements, the timing for monitoring procedures can be implemented with online measurements, which are important mainly for environmental parameters.

For structural parameters, some online measurements can also be established, such as joint displacement or deck vibration. In most cases, however, it is sufficient to use continuous manual measurements during certain reference periods (such as 3 consecutive hot or cold days) each year (Figure 4-16).

Because variations are quite slow for the durability parameters and structural characteristics of concrete, manual in situ measurements can be performed every one or two years.

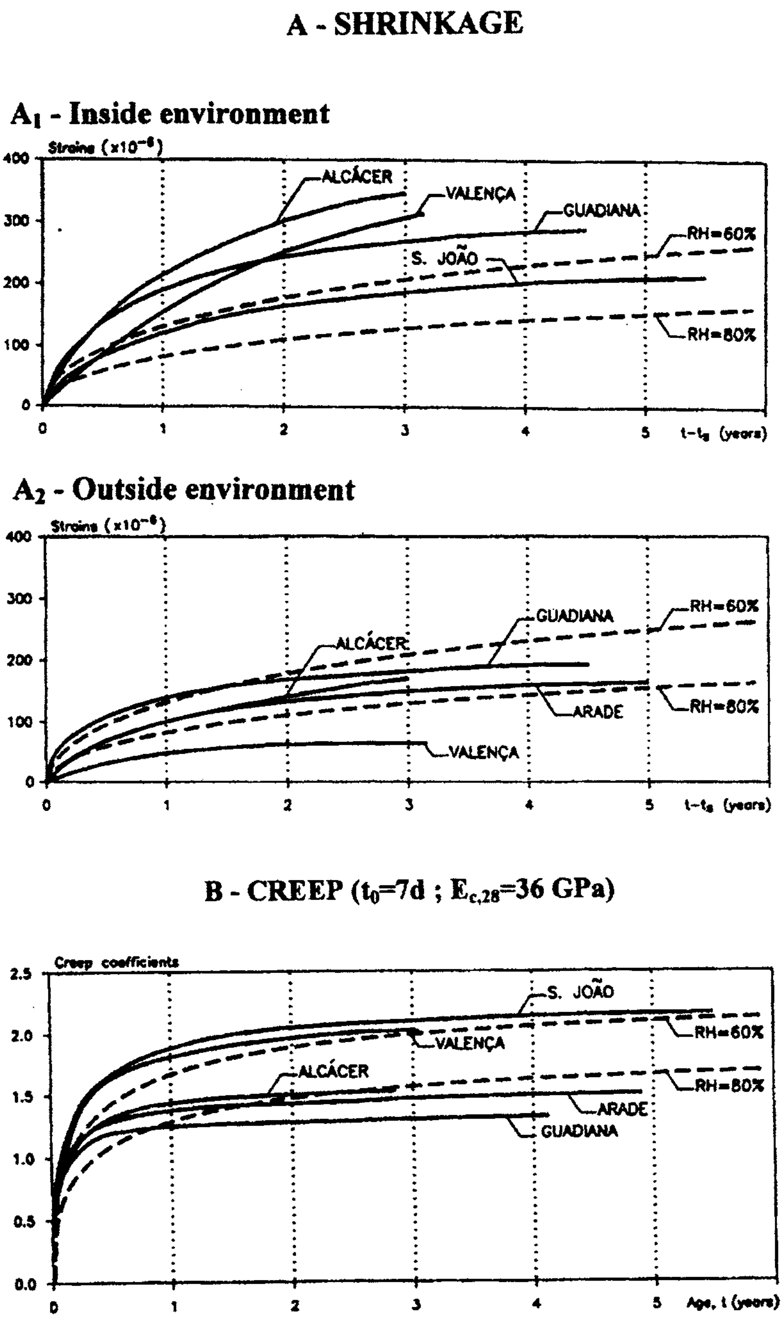

Figure 4-16. Monitoring creep and shrinkage during several years in several bridges

4.12. Technical Specifications

The design technical specifications should point out the correct procedures for building the bridge, specifically:

- materials characteristics;
- building procedures;
- production control;
- conformity control;
- monitoring implementation.

The main issues associated with these construction and control procedures are discussed in Chapter 5.

At the design stage, the specifications for the materials and components should include not only those characteristics expected for the materials but also definitions of the tests with which those characteristics are measured. In fact, the same characteristic can be measured by different test methods and the results are frequently difficult to correlate.

Based on design estimations of the service life of the components, a basic inspection and maintenance plan for the structure should also be presented in the technical specifications.

4.13. Data Information Storage

For bridge management from design to service life, all the bridge information must always be available in the bridge owner's files.

Some elements of the design should be highlighted in the files regarding any assumptions that were made to estimate the service life of the bridge or the time-dependent evolution of the concrete. This information should be placed in a file of "Parameters to be checked" and should be analyzed during construction and service life and compared with the results of the construction tests and/or the monitoring plan.

All the design elements, from drawings to technical specifications, should be placed in a computer database. If this database is at an Internet site, it will always be available for updating not only to the authorities, but also to the bridge builder and later to the bridge inspector.

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CHAPTER 5

Construction for Durability

5.1. General

Concrete structures are frequently built under conditions that are far from ideal, which may lead to some anomalies. Among these, the anomalies that occur at the concrete surface (cracks, honeycombing, etc.) are bound to have the most effect in reducing the durability of the structural elements.

The causes of these anomalies are essentially related to poor structural detailing, unsuitable concrete mixing, and negligent concreting operations. Detailing should prevent congestion of reinforcement and excessively narrow sections. Concrete mixing that achieves satisfactory strength may not be good in terms of durability or workability, so initial studies must be performed to select a suitable material. From a durability point of view, compaction and curing are the most important operations for which special rules should be adopted to prevent construction anomalies.

These anomalies occur less frequently in concrete precast elements because factory production allows better quality control and more standardized procedures than does in situ production. In fact, the durability of precast elements in terms of concrete quality is generally better than the in situ concrete elements for the same bridge.

In addition to the problems related to the execution of concrete structural elements, durability problems may also arise in other bridge components as a result of incorrect positioning of the equipment, deficient drainage or insulation solutions, bad compaction of earth fills, and so on.

Special actions must be taken during the construction phase to prevent these anomalies, which can include:

- preparation (including the construction planning and initial characterization of the materials);
- production (including all activities related to the construction of the bridge and the associated quality control).

In general terms, good quality control procedures are the basis for obtaining the target service life (Figure 5-1). These quality control procedures should include the production control performed by the contractor and the conformity control performed by the bridge

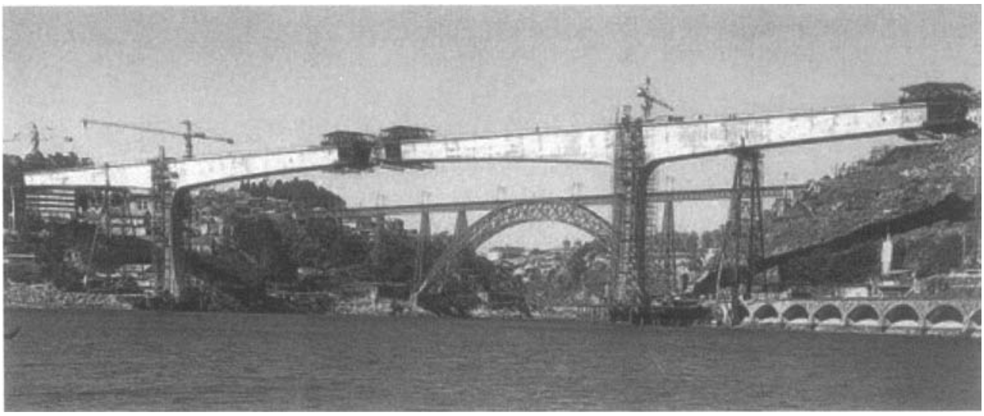

Figure 5-1. S. João Bridge in Porto, Portugal, during construction

owner. Close contact between the contractor and the designer is also very important to prevent anomalies and to deal with them when they occur.

In addition to the procedures developed during construction to build a durable bridge, it is also important for the future bridge management system that the relevant construction information be kept within the bridge database. This is frequently helpful in the analysis of problems that arise during the service life of the bridge.

5.2. The Preparation of the Construction

5.2.1. Construction Planning

The construction phase begins with a careful study of the bridge design, followed by planning sessions in which the methods and equipment to be used are analyzed so that construction procedures are implemented that are compatible with the fulfillment of the design specifications, especially those related to durability.

To prevent structural durability problems, the initial analysis of the design should identify details such as congestion of reinforcement or excessively narrow sections, in which compaction may be difficult (Figure 5-2). Alternatives should be discussed with the designer, such as reducing the number of bars (Figure 5-3) by increasing their section, or introducing slight changes in the geometry (eliminating sharp corners) to allow easier concreting.

The construction methods and equipment plan should also identify potentially difficult situations. These may arise from large masses of concrete being dropped from high above, locating construction joints, cold/hot weather, and wind conditions. In conclusion, all the situations that require special measures to prevent surface anomalies should be taken into account.

At the planning stage, the production quality control plan should be developed, indicating appropriate actions for the following activities:

- control of the construction procedures, before and during concreting and curing;
- inspection of anomalies after concreting;
- controls and procedures for bridge equipment installation;
- repair measures for anomalies.

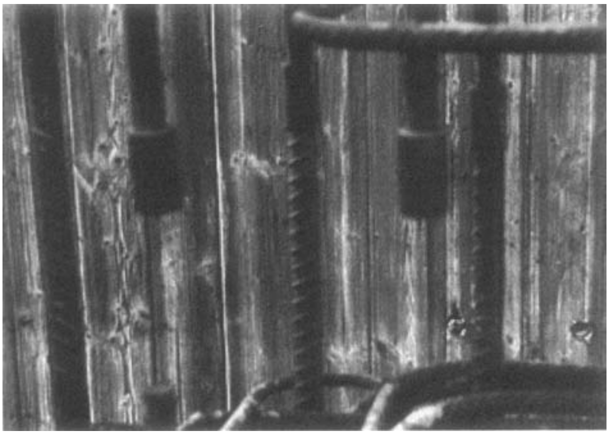

Figure 5-2. Use of splices to prevent reinforcement congestion

The construction plan should be developed with the inclusion of an initial characterization of the materials. If a monitoring plan exists, its interference with construction should also be considered.

5.2.2. *Initial Characterization of Material Properties*

Before construction begins, the materials must be selected and their conformity must be checked against design specifications. As far as structural materials are concerned, this means that the characteristics of steel (reinforcement and prestressing) and concrete must be initially analyzed.

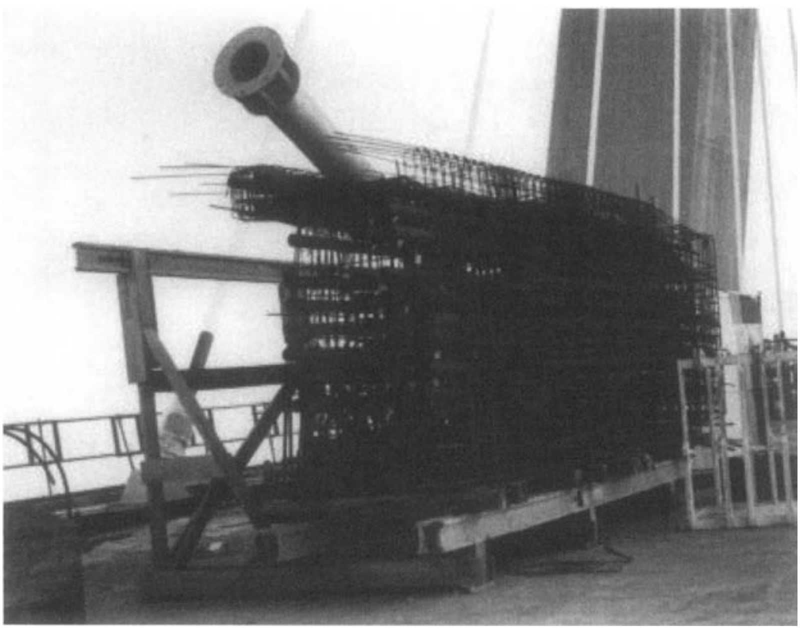

Figure 5-3. Use of premounted reinforcement to achieve better control

Steel strength is usually controlled in steel foundries and few initial tests are usually required. Fatigue (especially for railway bridges) and anchorage-tendon connections are important characteristics for prestressed steel and systems, which are less uniform because they may vary with production conditions. Therefore, tests should be performed before construction begins.

Concerning concrete characteristics, several tests must be performed before construction begins to achieve a concrete with the properties defined in the design specifications. These tests include the study of several concrete compositions and must consider the following aspects (pr ENV206 1999):

Requirements for Materials

The basic materials shall not contain ingredients that are detrimental to the durability of the concrete. The suitability of cement, aggregates, mixing water, admixtures, and additions should be checked according to technical specifications.

Requirements for Composition of Concrete

The concrete composition and basic materials must be chosen to fulfill the requirements specified for fresh and hardened concrete, especially as they relate to consistency, strength, and durability. This leads to the study of the following parameters:

- type and content of cement and water/cement ratio (which should be suitable in terms of environment, hydration heat, reactivity of aggregate to alkalis, etc.);
- aggregates (should be suitable considering maximum dimension aggregate, environment conditions, reactivity to the alkalis, etc.);
- additions and admixtures (their quantities should comply with technical specifications);
- chloride content (it should be limited to 0.2 % by mass of cement for prestressed concrete or 0.4 % for reinforced concrete).

Testing Fresh Concrete

The characteristics of fresh concrete are mainly related to its workability, but samples also must be used to check some of its durability characteristics as they relate to its composition. The following parameters are usually determined:

Consistency, which allows the classification of concrete in terms of workability. It is usually determined by the slump test, Vebe test, or flow table test;

Water/cement ratio and cement content, which are obtained from the weight of the concrete components;

Air content, which is obtained from ISO 4848;

Chloride content, which is the measurement of the amount of chloride ions in fresh concrete;

Hydration heat temperature, which is the measurement in adiabatic conditions of the evolution of the hydration heat temperature following the procedures presented in Chapter 3 (especially if problems are anticipated when concreting thick elements);

Density, which is the measurement of fresh concrete density.

Testing Hardened Concrete

The characteristics of hardened concrete are mainly related to its mechanical and durability properties. The tests associated with these properties can be divided into the following categories:

General Characteristics

Density—Measurement of density of hardened concrete;

Young Modulus—Obtained from the testing of prismatic specimens in which compression/deformation is measured. The evolution of the values during the first months should also be determined.

Strength

Compression strength at 28 days—Determined by using cubical or cylindrical specimens;

Tensile splitting strength at 28 days—Determined using the Brazilian test in cylindrical specimens;

Compression strength evolution—Determined several days after concreting to obtain the evolution of the concrete cure. The relationship between compression strength evolution and the hydration heat temperature evolution allows for the correlation of empirical values between strength and temperature.

Long-Term Behavior

Creep and shrinkage evolution—The evaluation, in the lab and under in situ conditions, of the evolution of creep and shrinkage effects during the first years, and its comparison with code path proposals (Figure 5-4).

Durability

Oxygen permeability test—Measurement of the permeability of oxygen in concrete;

Accelerated carbonation test—An evaluation of the evolution of carbonation;

Water absorption tests—Measurement of water penetration with time;

Water permeability test—Measurement of concrete permeability;

Diffusion tests—Measurements of the diffusion coefficient of chloride inside concrete;

AASHTO 277 migration test—Definition of the quality of concrete in terms of chloride diffusion.

Among these tests, some, such as the accelerated carbonation and the diffusion cell test, may take a long time to give rise to significant results. To overcome this situation, the best procedure is always to initiate the concrete composition study as soon as possible and perform the long-term duration tests.

At the same time, with the same concrete, less time-consuming tests can be performed (water absorption, oxygen permeability, and AASHTO 277), which will be periodically repeated during construction to check the uniformity of production within the quality control procedures.

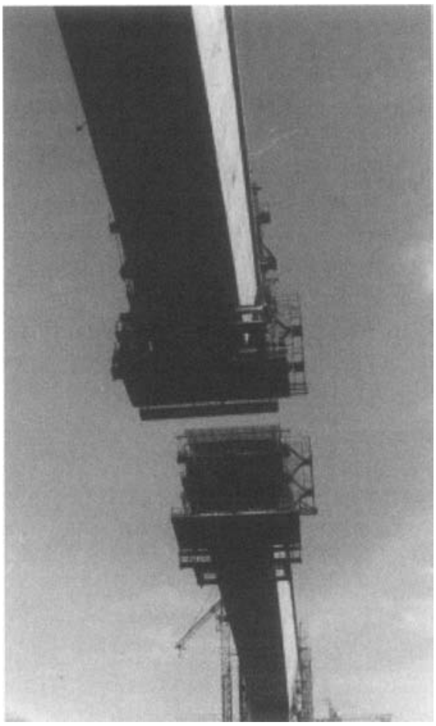

Figure 5-4. Cantilever construction needs accurate control of shrinkage and creep, S. João Bridge, Porto, Portugal

5.3. The Production

5.3.1. Inspection Before Concreting

The following actions and checks should be performed before concreting (prENV13670-1 1999):

- formwork geometry and reinforcement position;
- cleaning of formwork and previous hardened concrete surfaces;
- treatment of existing hardened concrete joints;
- wetting hardened concrete surfaces;
- formwork stability and fixing systems (Figure 5-5);
- placement of formwork windows;
- insulation of formwork joints to prevent spilling;
- appliance of release agents in the formwork;
- cleaning of reinforcement (oils, rust, paintings);
- availability of efficient means for transportation, compaction, and curing;
- availability of skilled workers.

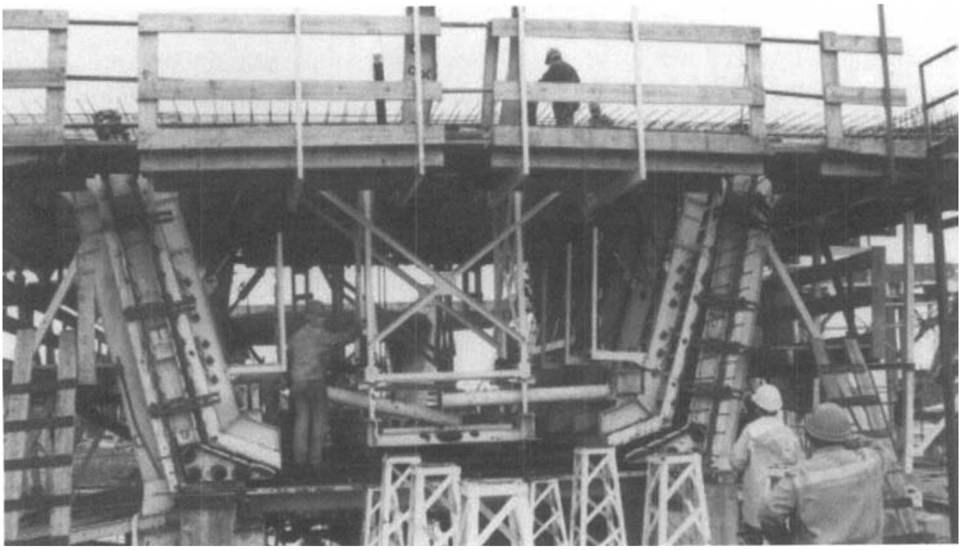

Figure 5-5. Initial inspection of formwork, central viaduct, Lisbon, Portugal

At this stage, a well-designed and well-built formwork is especially important. The design of the formwork should consider its stability and deformation control under construction loads (weight and lateral pressure of fresh concrete), as well as the foundation stability of the scaffolding.

The structures associated with the formwork are frequently complex systems (Figure 5-6), which must be moved step by step, during concreting; deficient connections may lead to accidents.

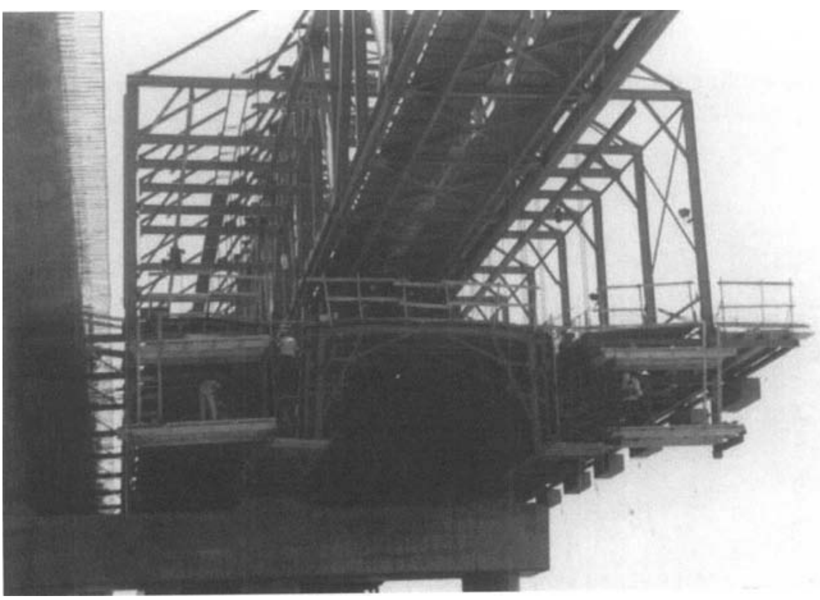

Figure 5-6. Mobile formwork, south viaduct, Alcochete, Portugal

5.3.2. Concrete Placing and Compacting

There are general procedures for the placement and compacting of concrete in order to achieve good quality. They can be listed as (prENV13670-1 1999; prENV206-1 1999):

- concrete must be placed in situ as soon as possible after mixing, to reduce the loss of workability;
- concrete must be placed within a temperature range of 5 °C to 30 °C;
- during concreting, segregation must be avoided, reducing the downfall height of the mixture;
- after placement, concrete must be carefully compacted close to reinforcement bars, in formwork corners, and especially in the reinforcement cover zones;
- vibration of concrete should be performed avoiding displacement of reinforcement bars;
- vibration must be performed continuously (Figure 5-7), thus preventing segregation and until air bubbles stop forming.

5.3.3. Concrete Protection and Cure

To obtain the properties defined for concrete, especially close to the surface, which is the most critical zone in terms of durability, it is necessary to implement good protection and curing during an adequate period (prENV206-1 1999; Gonçalves 1999).

Surface Protection

Protection of the surface should be implemented as soon as possible after concreting to prevent the following hazards:

- washing of fine aggregates resulting from rain or water;
- rapid cooling during the first days after concreting;

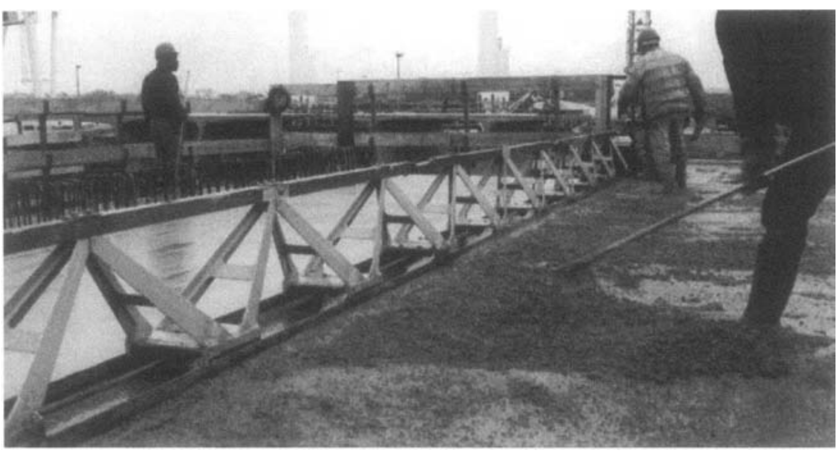

Figure 5-7. Surface vibration

- high thermal differences between surface and interior;
- low surface temperatures or ice;
- vibration and impact damage, which may fracture the concrete structure.

Curing

Curing should be implemented after concreting to prevent drying of the surface due to solar radiation or wind. The usual procedures for achieving good curing are:

- keep the formwork in place;
- covering concrete with plastic film;
- place wet covers on top of exposed surfaces;
- wet the surface;
- use curing products that develop protective membranes.

The curing period is related to the time necessary to obtain a certain degree of insulation to gas and liquids in superficial concrete, thus protecting the reinforcement. The curing time is usually defined by:

- maturity, based on hydration of concrete and environmental conditions;
- local specific demands;
- using reference time values (prENV206-1 1999) (Table 5-1).

The temperature during the curing period is used in the precast industry to reduce the curing period by heating the steel formwork with heated water at around 60 °C. This leads to a curing period of around 12h, at the end of which the precast beams can be prestressed and removed from the formwork.

Table 5-1. Reference curing periods

	Minimum curing period			
	Strength development ($r = f_{cm2}/f_{cm28}$)			
Concrete temperature °C	**Quick $r > 0.5$**	**Average $r = 0.3$**	**Slow $r = 0.15$**	**Very slow $r < 0.15$**
$T > 25$	1	1.5	2	3
$25 > T > 15$	1	2	3	5
$15 > T > 10$	2	4	7	10
$10 > T > 5$	3	6	10	15

f_{cmi}—mean compressive strength at i days

Formwork

Formwork should be removed after concrete gains enough resistance to withstand any action at the construction stage or when there is no necessity for the curing process. Research has also been developed to achieve better surface quality by acting on the formwork.

Solutions using the Controlled Permeability Formwork (CPF) have been studied to achieve better durability characteristics at the concrete surface (Coutinho 1999). CPF is a textile liner applied to the formwork, which acts as a filter/drain, allowing air bubbles and surplus of water to drain while retaining cement particles. This process leads to a very dense outer concrete layer with a very low w/c ratio.

A comparison of concrete in which CPF was used with one molded in a standard wood formwork showed that the concrete in which CPF was used had a decrease in water absorption and carbonation of around 60 % and that the diffusion coefficient decreased by about 40%.

The use of vegetable release oils (Figure 5-8), instead of classic mineral oils, has also proved to be a good solution because it leads to a better surface quality and has the important environmental advantage of being biodegradable (de Brito 1999).

Hydration Heat Control

Hydration heat is generated at the early ages of concrete and is associated with the exothermic chemical reactions of the cement hydration. As a result of the poor thermal conductivity of concrete, high temperature gradients may occur between the interior and the surface of the structural elements, or between parts of an element with sequential concreting phases. These nonlinear temperature distributions may lead to tensile stresses exceeding the tensile strength of the young concrete, and cracking will then occur. To prevent such damage, empirical curing methods are adopted, depending on the experience of the contractor. These methods usually include precooling the materials, using cooling pipes or low hydration heat cements.

Knowing the hydration heat temperatures in adiabatic conditions, a numerical analysis can be used (Figure 5-9) to study and optimize the concreting phases (see Chapter 3) and to avoid hydration temperature problems (Branco 1992).

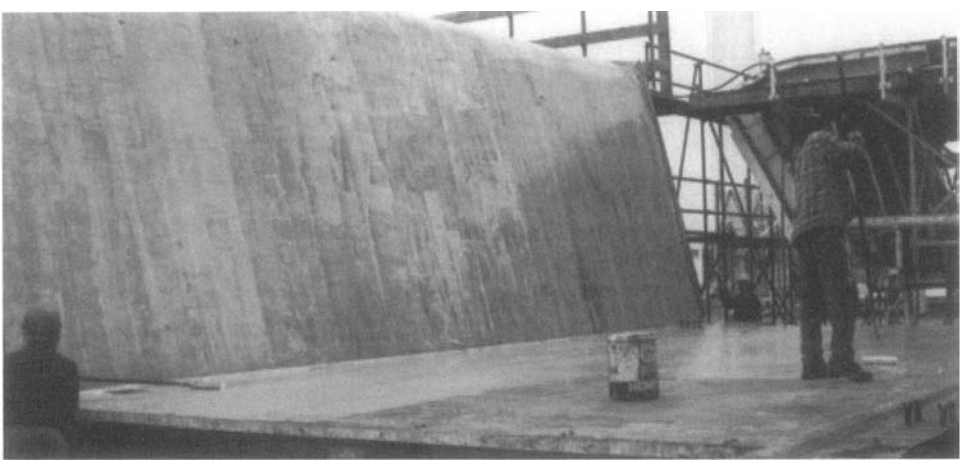

Figure 5-8. Application of release agents

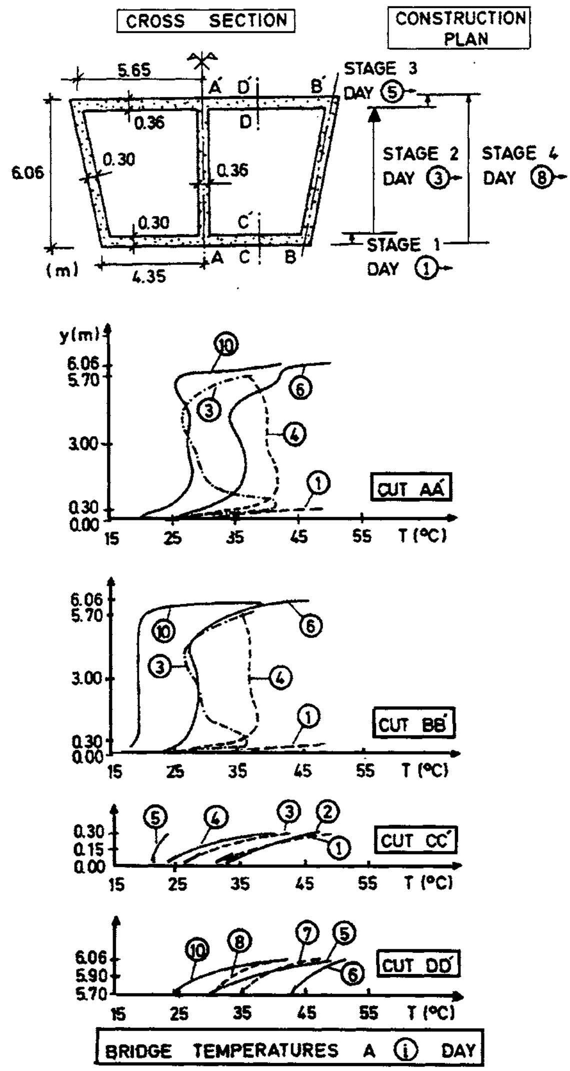

Figure 5-9. Hydration heat temperatures and construction phases for the S. João Bridge in Porto, Portugal

5.3.4. *Inspection During and After Concreting*

The following inspections and controls should be performed during concreting (prENV13670-1 1999, prENV206-1 1999):

- uniform placement of concrete;
- uniform compaction without segregation;
- maximum height for concrete downfall;
- maximum thickness of the layers;

- concreting speed and pressure against the formwork;
- maximum time between mixture and concreting;
- special measures in cold and hot weather;
- special measures under rainy conditions;
- location of concreting joints;
- treatment of construction joints before hardening;
- finishing operations of surface;
- concreting procedures and curing method according to environmental and weather conditions;
- damage in concrete due to vibration or chock.

After concreting and curing, a careful inspection of the surfaces should be performed to detect the existence of cracks or other surface anomalies. To better visualize the cracks, the surfaces should be cleaned and slightly wet, because cracks may appear when the surface dries.

5.4. Quality Control

5.4.1. *General*

The production, placement, and curing of concrete must be performed according to quality control procedures in order to fulfill the specifications. These quality control procedures are important and must be performed in a systematic way during the entire construction stage.

Quality control includes two activities concerning production control and conformity control. Production control is performed by the contractor and should consider the materials characteristics and the construction operations necessary to fulfill the specifications. Additional conformity control procedures are performed either by the owner or by a legal authority that checks samples of the construction products.

Quality control procedures are based on the performance of an inspector who checks to determine whether certain construction parameters are within specified limits and makes sure that certain necessary procedures have been implemented. These parameters and procedures are defined to guarantee the behavior of the materials and the characteristics of the structure, especially in terms of its durability.

Knowledge-based systems are being developed to help the contractor perform quality control checks. Also, the adoption of rationalized procedures for control is useful, if anomalies are found because the system suggests to the inspector the best procedures to overcome the problem (Branco 1999). Knowledge-based systems have been implemented mainly in the precast industry where the procedures are more standardized, thus leading to higher quality levels in these elements.

The main activities associated with the quality control of concrete bridges are related to the concrete and reinforcement characteristics and procedures for concreting, compacting, and curing.

In terms of bridge equipment, quality control tests should also include performance tests of bearings and seismic devices (load and displacement), electrical equipment and other specified equipment. These tests are frequently performed at the factory where the equipment is produced, within its own quality control plan, and at the bridge site, usually only by checking the main properties of the equipment. A visit to the factory where the equipment is produced by the contractor and/or the bridge owner is always worthwhile as a means of determining how quality control procedures are performed.

In terms of equipment durability, it is frequently difficult to define its service life, but data based on their use past experience with their use should always be collected and, if possible, an inspection of those sites where difficulties have arisen in the past should be performed.

5.4.2. Concrete Production Control

Equipment

The equipment related to production and control of concrete must be checked periodically, specifically the stockpiles, bins, weighing equipment, admixtures dispensers, water meters, batching system, testing apparatus, mixers (including truck mixers), and so forth. The frequency of these checks should be determined by the kind of equipment used, its sensitivity and manner of use, and the production conditions of the plant (prEN206 1999).

Components of Concrete

These production control procedures include the tests to be performed to check the characteristics of cement, aggregates, admixtures, additions bulk powder, additions in suspension, water, and so on. Usually the control checks are performed with each delivery, according to international specifications (prEN206 1999).

Reinforcement and prestressing should also be checked with each delivery, especially in terms of tensile strength and fatigue behavior.

Concrete Production

The controls related to concrete production are relatively standardized and consist mainly of the following tests, most of which are performed with each batch (prEN206 1999):

- characteristics of concrete components;
- consistence;
- density of fresh and hard concrete;
- cement content of fresh concrete;
- water content of fresh concrete;
- air content of fresh concrete;
- water/cement ratio;
- compressive strength of specimens.

In addition to these tests, other tests must be performed periodically to check the uniformity of the concrete characteristics related to the values obtained in the initial tests. These concern:

- Mechanical properties-creep and shrinkage characteristics, strength evolution;
- Durability-chloride content, water absorption test, AASHTO 277 test, gas permeability.

Concreting Inspections

During the production quality control, inspections must also be considered before, during, and after concreting to ensure that all the activities described in Section 5.3 are within the limits of the production control plan.

5.4.3. Conformity Control

Conformity control consists of a combination of all the actions that must be taken during construction to periodically check the conformity of the structure, in accordance with a predefined sampling and testing plan.

Conformity control checks are usually performed by a certification group or by the bridge owner (or his representative) and should be an integral part of production control.

The sampling and testing plan and the conformity criteria should be defined in the tender specifications. Concerning concrete, steel reinforcement and bridge equipment properties, the conformity controls are usually performed with the same type of tests used in the production control, but any test that is considered relevant by the owner can be added (prEN206 1999).

Conformity control can also include bridge acceptance tests (static and dynamic load tests) to be performed at the end of construction to check the global behavior of the bridge.

In the conformity control plan, actions should also be defined for the situations of nonconformity. Usually, further tests are performed to check the previous ones (including in situ tests for concrete nonconformities), and if confirmed, decisions must be made about repair or demolition. In these cases, the designer should be involved in the decision-making process.

5.5. Construction Anomalies

5.5.1. Concrete Elements Anomalies

The construction anomalies that have a greater influence on the durability of concrete structures are those that occur at its surface, thus causing a reduction in the protection given by the cover of the reinforcement. These anomalies usually arise from the concreting operation or from the curing phase.

Anomalies related to the concreting operation include honeycomb, subsidence cracks, excessive bug-holes or surface voids and, less importantly, layer lines or rust stains (Allen 1999). These last anomalies arise from a deficient vibration and from formwork contact with rusted bars, respectively.

Honeycomb (Figure 5-10) is a condition of irregular voids caused by failure of the mortar to effectively fill the spaces between coarse aggregate particles. It may arise from congested reinforcement, insufficient cement content, improper sand-aggregate ratio, or inadequate placement techniques. Homeycomb may be reduced with better vibration and improvement of workability.

Bug-holes (Figure 5-11) are normally caused by air bubbles and occasionally by water entrapped between the concrete mass and the formwork. Bug-holes are also associated with low workability mixtures or deficient vibration. Excess water also leads to bleeding channels or sand streaks on formed surfaces.

Aggregate transparency is associated with a mottled coloring of the surface that results from deficiencies in the mortar. It usually occurs in concrete mixtures with low sand content, porous aggregates, or high slump.

Figure 5-10. Honeycomb

Subsidence cracking results from the development of tension in upper layers of the concrete when it settles. It can be prevented with the prompt use of vibration, thus connecting the upper and lower layers. Increasing concrete cover and using low slump concrete can control subsidiary cracking over reinforcement (see also plastic settlement cracks).

Sand streaking (Figure 5-12) is a concentration of exposed fine aggregates at the surface, caused by bleeding along the form. Sand streaking can be prevented with the use of tight forms and fine aggregates that are well graded in the mixture.

Cold joints (Figure 5-13) usually arise from concreting that is poorly planned. They can be avoided, with good planning, back-up equipment, keeping the concrete surface fluid, and similar measures.

Form offsets (Figure 5-14) are related to inadequate stiffness of the forms and can be increased with a high rate of placement or an overly powerful vibration.

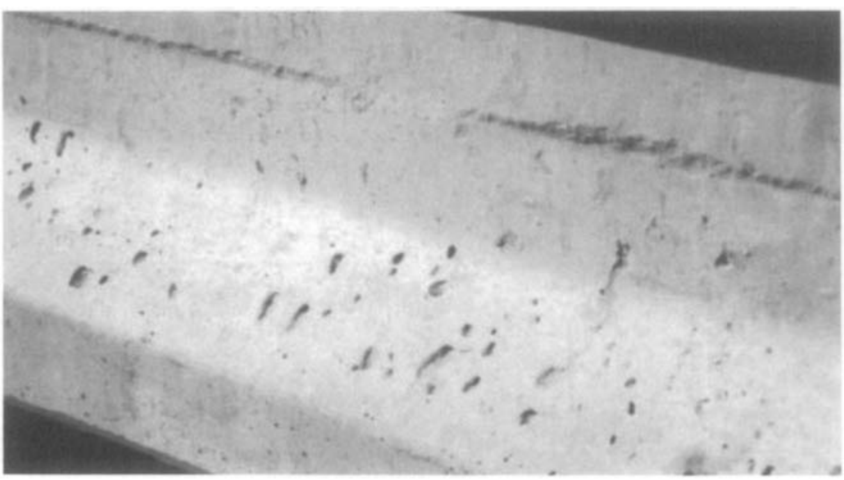

Figure 5-11. Bug-holes

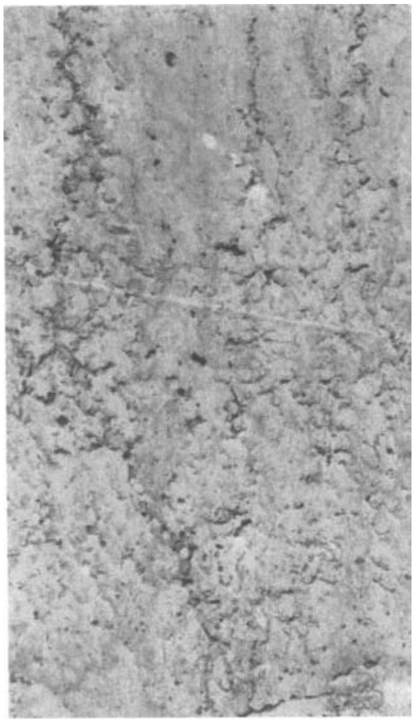

Figure 5-12. Sand streaking

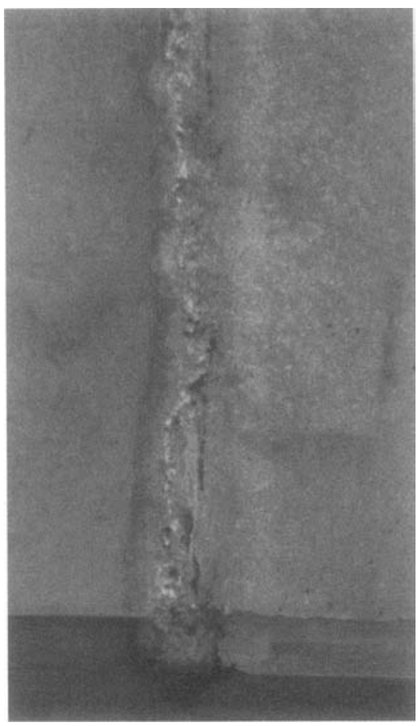

Figure 5-13. Cold joint

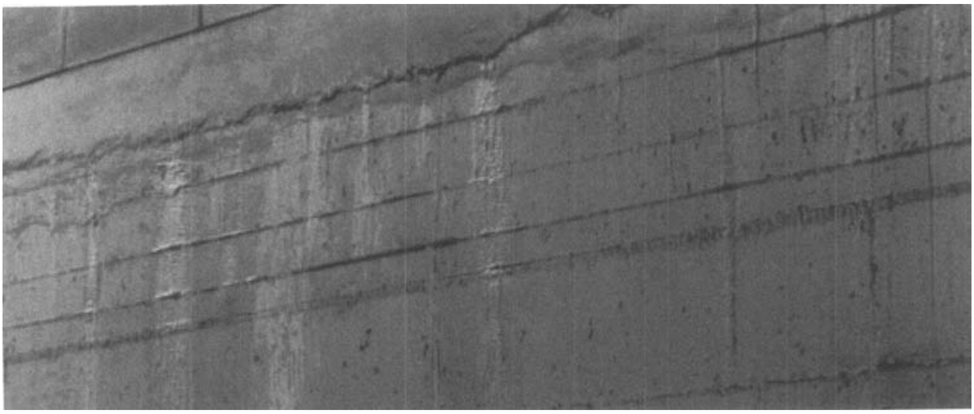

Figure 5-14. Form offsets

The following practices should be implemented to minimize these anomalies:

- the use of compositions with adequate aggregate size, consistency and workability;
- a sufficiently long vibration;
- vibrator insertions properly spaced;
- each concrete layer consolidated from the bottom upward;
- increased vibration periods when using impermeable forms;
- limited depth of placement layers;
- vibrators penetration in the previous layer;
- stiff formwork to prevent leakage;
- avoiding complex design details and congested reinforcement;
- design of placing ports when necessary;
- proper placement in order to prevent segregation and splashing of concrete.

In addition to the anomalies that occur during the concreting operation, other surface anomalies might occur during the curing period. These anomalies are usually superficial, such as random cracking, with small width (<0.3 mm) and small depth (<1 cm), but structural cracking due to shrinkage and hydration heat may also occur. They are caused by the following factors:

Superficial plastic shrinkage cracks occur soon after finishing and are caused by a rapid drying of the surface when the concrete mixture is still plastic. They usually have a random alignment and variable length. This shrinkage and cracking can be avoided by protecting the surface from wind and sun and by proper curing.

Superficial plastic settlement cracking (Figure 5-15) occur in deep sections over and along reinforcement bars or formwork ties and occur as a result of the plastic concrete settlement that is impaired at the location of the bars, thus leading to cracking. This effect may also occur in vertical surfaces where settlement is locally reduced by friction with the formwork.

Figure 5-15. Plastic settlement cracking

Hydration heat cracks are associated with Eigen stresses originating from thermal differences between the interior and the surface of massive elements, or as a result of deformation restraints in structural elements, when the concrete still has a low tensile strength. They are similar to shrinkage cracks, but occur a few days after concreting.

Shrinkage cracks (Figure 5-16) occur a few weeks after concreting and are associated with global shrinkage of the structure and the existing restraints to free displacements. Shrinkage cracks cut through the whole section that is in tension and frequently have periodic spacing that increases with time during the first year or two. Frequently they appear in cold joints, because such joints are weaker structural zones.

The detection of cracks in the bridge deck should be performed on a rainy day before placement of the bituminous pavement (or by soaking the surface) by observing wet zones under the deck.

Figure 5-16. Shrinkage cracks

In terms of durability, shrinkage cracks provide a path to a quicker attack of environmental agents, especially chlorides. To prevent this, the cracks should be sealed with low-viscosity resins as soon as they stabilize. If crack width is very small (<0.2 mm), it should be slightly beveled before sealing.

It should be noted that with the adoption of important reinforcement covers (>5 cm), the surface cracks may occur very easily, and if proper measures are not adopted to prevent them, the durability advantages of the thick cover disappear.

5.5.2. Other Anomalies from Construction

5.5.2.1. Earth Works

Approach fill. The connection between the road and the bridge is performed with an approach earth fill. The compaction of the earth close to the abutment is difficult and settlement frequently occurs as a result of traffic, which leads to the development of a road step that is an inconvenience to drivers (Figure 5-17). To prevent settlement, proper compaction should be performed, even with the use of geotextile and the placement of an approach slab (ca. 5 m in length) between the abutment and the approach fill (Figure 5-18).

This problem is more serious in bad foundation soil where the bridge is founded with piles and the earth fill suffers settlements for several years. In this case, periodic maintenance refills of the road are frequently needed.

Earth fill slopes. Earth fill slopes under a bridge are placed in a shadowed zone, in which vegetation rarely grows. In urban areas, superficial protection is necessary both for aesthetic reasons and to prevent soil erosion. Protection should also include gutters to collect water drainage.

These slopes should also be well compacted to prevent deterioration associated with movements of the earth fill settlement (Figure 5-19).

In areas that may suffer water flooding, the earth fill must also be protected with stone to prevent erosion or scour effects.

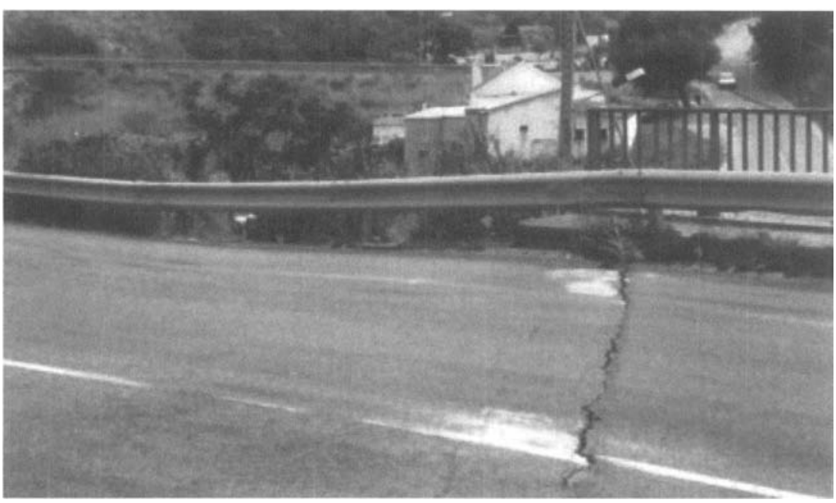

Figure 5-17. Settlement of the approach fill

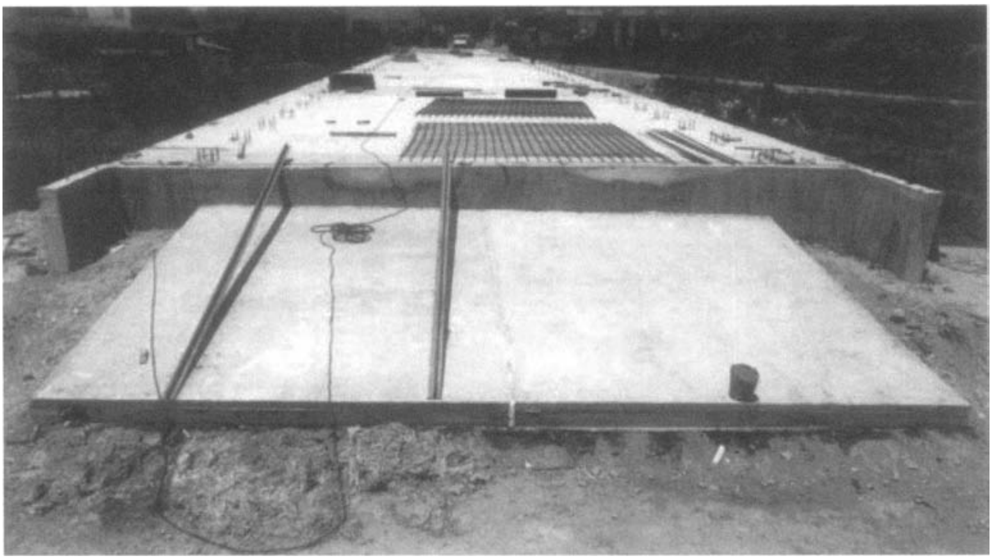

Figure 5-18. Approach slab in the abutment

5.5.2.2. Bridge Equipment

Expansion joints. As a result of shrinkage of the deck, expansion joints frequently remain too open, thus leading to the deterioration of the filling material and to an accelerated deterioration due to traffic (Santiago 1999).

The opening of expansion joints adopted at the construction stage should be defined based on structure temperature and an analysis of shrinkage evolution prediction (Figure 5-20). This is especially important in precast girder bridges where several joints exist. Defi-

Figure 5-19. Deterioration of earth fill slopes

Figure 5-20. Placement of a bridge joint

cient placement of expansion joints frequently leads to high repair costs soon after the bridge opening.

Bearings. The placement of sliding bearings should be performed taking into account the effects described for expansion joints (Figure 5-21). This prevents situations in which the bearings remain eccentrically loaded during the bridge life.

Fixed bearings may absorb additional forces from shrinkage or creep effects, leading to their malfunction under horizontal loads. This should be prevented with a position regulation after the development of time-dependent effects.

Frequent anomalies related to the transportation fixing devices of the bearings not being dismounted, or by installing the bearings in wrong free movement directions, can be avoided with careful control.

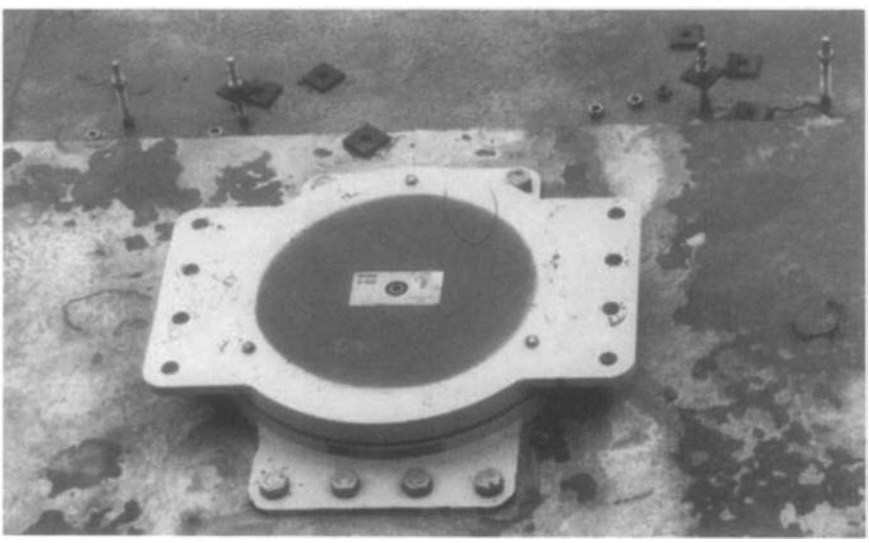

Figure 5-21. Placement of a bearing

Figure 5-22. Deficient placement of deck drain

Drainage. Deck drains must be carefully placed to prevent the obstruction of water drainage while simultaneously preventing the entrance of pieces of large debris. If obstruction occurs, sand deposits and vegetation will soon grow, thereby reducing drainage capacity and leading to water accumulation in the deck (Figure 5-22). Furthermore, water also will begin to penetrate the joint between the deck drain and the concrete, leading to additional deterioration (Figure 5-23).

5.6. Construction Data Files

Within a bridge management system it is important to have a file for each bridge that contains the most relevant information from construction, especially those aspects that can help in clarifying problems that eventually will occur during service life.

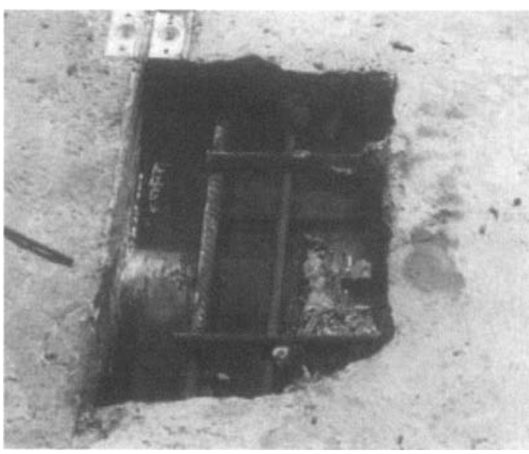

Figure 5-23. Lack of protection against water

The construction information to be kept should consist of the following topics:

- Schedule of the construction phases (with average temperature and humidity);
- Photos of the construction phase;
- Changes from initial design and updated drawings;
- Geotechnical characteristics (tests and photos for each foundation);
- Test results of concrete component characteristics (with date);
- Test results of materials mechanical characteristics (date and bridge location);
- Test results of materials durability tests (date and bridge location);
- Results of geometry control (date and environmental data);
- Results of load and other tests;
- Anomalies and repair measures;
- Recommendations from designer, contractor, and inspectors regarding service life.

PART 3

Overview of Bridge Management Systems

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CHAPTER 6

Bridge Management Systems in North America

6.1. Introduction

The need for rational bridge management is undeniable. Some of the trends caused by insufficient or inefficient management can be found in Kerman 1999. The existing backlog of bridges that are structurally or functionally substandard cannot be dealt using normal maintenance. On the contrary, the fact is that many bridges that have shown no distress have failed assessments. There is a very delicate balance between insufficient repair (leading to failure and traffic disruption) and overdoing it (causing frequent traffic disruption while the work is going on). Road or railway bridges are installed to serve the user, so keeping them open and operating must be a primary objective. Therefore, maintenance procedures must be designed to minimize any traffic disruption.

The implementation of bridge management systems, particularly those based on computerized systems, is relatively recent. Even in the most advanced countries, a complete computerized system, similar tó one presented in a European research project in Bridge Management in Europe (BMIRE) (Woodward et al. 2000), is either still in planning for the future or is undergoing a trial period. The management of bridges is different from country to country and, even within a country, only rarely does the responsibility fall to a single entity. This results in a multitude of different approaches to the problem, with many investigation teams working in parallel, often multiplying the total effort to achieve more or less the same result, even when the design process is different.

This chapter presents the highlights of the main management systems in North America as documented in the literature. In Chapter 7, succinct descriptions of some of the existing systems outside North America are presented and are divided by country of origin, with special attention to those systems for which more information is available and the system is more developed. For the other countries, only a brief description is presented. In Chapter 8, a brief summary of the main characteristics of existing bridge management systems is presented and a summary of the architecture of a standard bridge management system is presented.

6.2. The Pennsylvania Department of Transportation BMS (US)

6.2.1. Introduction

The Pennsylvania Department of Transportation organized a Bridge Management Task Group whose objective was to develop a Bridge Management System (BMS). The Task Group prepared several preliminary documents, (BMTG 1984; BMTG 1985a; and PDT 1982) before putting forward reference (BMTG 1985b), which was later updated (BMTG 1987). McClure et al. and McClure and Hoffman (1990) summarize the most important points.

In the system presented, little attention is given to the inspection module. However, the decision system, both at the maintenance and rehabilitation/replacement levels, is very clearly defined and allows for the easy use of computer programs for decision making.

The system had but a few years of practical use, and some of the coefficients proposed at this stage may have to be calibrated based on the experience acquired.

6.2.1.1. Objectives

The specific needs for a Bridge Management System (BMS) are to develop a system that is used on demand, but is used at least annually to:

- yield recommendations and associated cost estimates, for activities required to enable all bridges on public highways and roads to perform their functions in the most cost-effective manner; these activities include various levels of maintenance, various modes of rehabilitation, and replacement;
- predict present and future needs and associated cost to perform the above activities for all bridges in at least two scenarios, including "minimum acceptable" and "desirable";
- set state-wide and regional priorities for each of the above activities, based on functional and physical needs for each highway classifications system and provides a listing of candidate bridges;
- provide a basis for recommending regional distribution of budgeted funds.

6.2.1.2. Overall Organization

The Department's computerized Structure Inventory Records System (SIRS) forms the basis for the development of an overall BMS. Enhancement of the SIRS database, integration with other databases, and the development of a structure cost data inventory file (cost on the basis of dollars per square foot of bridge deck area, constantly updated) form the complete database. The two main parts of the BMS consist of the subsystems for Maintenance (BMTS) and Rehabilitation/Replacement (BRRS) as shown in Figure 6-1 (McClure and Hoffman 1990).

6.2.2. The Maintenance Subsystem

Seventy-six potential maintenance activities have been identified. During an inspection, the field inspector completes a form in which a tabulation of all maintenance needs with each item quantified and prioritized by urgency is included. A unit price table for the 76 activities has also been established. It is updated periodically based on actual cost experience.

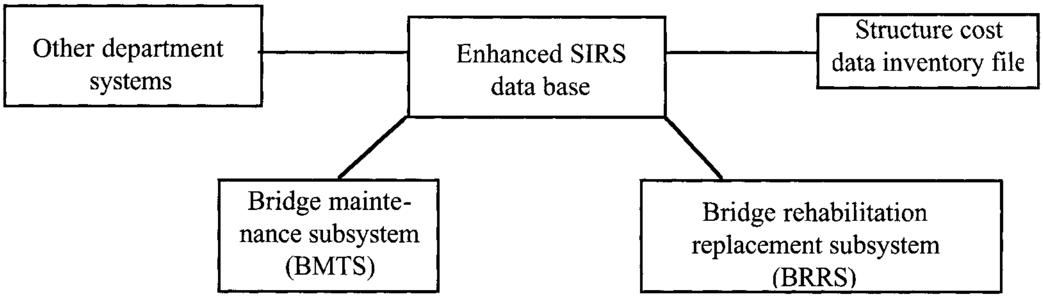

Figure 6-1. Pennsylvania Bridge Management System

The prioritization method procedure uses the following components: activity ranking, activity urgency, bridge criticality, and bridge adequacy.

As a general rule, activities that most directly impact the continued safety and structural adequacy of the bridge, immediately and positively, are performed first while those that have minimal immediate impact would tend to be performed last. The activities are divided in five groups from A to E based on their generalized relative importance to the current structural stability of the bridge. Deficiency points are assigned to each group according to Table 6-1 (McClure and Hoffman 1990). If the activity is fatigue related, it will be assigned as Group AF and given additional deficiency points.

The severity of a deficiency is a reason to increase its priority for repair. The District Bridge Inspection Unit codes the Urgency Factor for each activity need. It yields a funded assessment of how soon the work needs to be completed. As such, it is also a measure of the severity of the deficiency. It judiciously defines the promptness of action that is needed for each specific maintenance activity need. The priority codes used ranges from 0 to 5. A priority code 0 is used for a critical safety deficiency where prompt action is required, and a priority code of 5 is used for routine nonstructural bridge maintenance that can be delayed. Table 6-9 (Reel and Conte 1989) gives the deficiency points assigned to each priority code.

The importance of a bridge to the road network, as well as the impact of the loss of bridge service to traffic, are other factors that must be considered in deciding the order in which bridges are to be repaired. It is readily apparent that the road system hierarchy realistically defines importance. That is, if a bridge on an interstate highway and a bridge on a local access system have similar deficiencies, it is obvious that the interstate highway bridge would be repaired first. However, the impact of a bridge's closure also must be weighed. If the detour length is excessive and hence intolerable, the priority for repair should be raised. The assessment of the importance of the bridge should be based on the classification of the highway (kind of highway and Department Road Network Indicator), its ADT, and the detour length that would be imposed on traffic if the bridge were to be closed. The correspondent deficiency points are given in Table 6-1 (McClure and Hoffman 1990).

The condition rating of the most critical component of the bridge can be used to generally assess degradation. By considering both the current load capacity and the lowest condition rating of the structure's components, a measure of the inadequacy of the bridge can be obtained according to Table 6-1 (McClure and Hoffman 1990).

Although it is numerically possible for a single bridge to be assigned in excess of 100 deficiency points, the deficiency point assignment is limited to a maximum of 100. The higher the assignment on a bridge, the higher the priority.

Table 6-1. Maintenance deficiency points assignment

Deficiency points	Component	Element	Deficiency point assignment
25	Bridge maintenance activity rank (Note: AF = group A activity that is fatigue-prone and controls the inventory rating)	Group AF	40
		Group A	25
		Group B	20
		Group C	15
		Group D	10
		Group E	5
25	Activity urgency factor	Code 0	25
		Code 1	20
		Code 2	15
		Code 3	10
		Code 4	5
		Code 5	0
25	Bridge criticality		
	Part A: Interstate		5
	U.S. numbered highway		4
	State highway		3
	Country highway		2
	City, borough st & twp rd		1
	Part B: PCN		5
	PCN/coal haul		5
	Agricultural access		3
	Industrial access		3
	Part C: ADT × detour length		
	≥30,000		15
	≥15,000 but <30,000		10
	≥3,000 but <15,000		5
	<3,000		0
25	Bridge adequacy		
	Part A: Lowest condition rating <3		15
	>3 but ≤4		10
	>4 but ≤5		5
	>5		0
	Part B: Load capacity (inventory rating)		
	(H configuration) ≤12 t		10
	(H configuration) >12 to ≤20		7
	(ML80 configuration) >20 to ≤30		4
	(ML80 configuration) >30		0

The Maintenance Deficiency Point assignment for a bridge is based on the bridge maintenance activity that has the largest sum of deficiency points for activity rank and urgency. With a Deficiency Point Assignment being stored in BMS for every bridge, prioritized listings can be easily generated using the particular parameters desired.

6.2.3. The Rehabilitation/Replacement Subsystem

The prioritization for rehabilitation or replacement is based on the degree to which each bridge is deficient in meeting public needs. Deficiencies are evaluated in three general categories (level-of-service capabilities, bridge condition, and other related characteristics), and is then combined to yield a total deficiency on a scale that ranges from 0 to 100.

Four characteristics are included in the level-of-service capabilities evaluation: (1) load capacity, (2) clear deck width, (3) vertical clearance for traffic carried by the bridge, and (4) vertical clearance for traffic passing under the bridge. The level-of-service deficiencies are based on comparisons of the actual load capacity, clear deck width, and vertical clearances of the bridges with level-of-service criteria developed for the BMS. These criteria have been set at three levels: (1) minimum acceptable, (2) minimum design, and (3) desirable design. They are primarily dependent on the functional classification of the highway carried by the bridge, with some additional dependence on volume of traffic. Bridges that do not meet these goals have a level-of-service deficiency. Equations have been developed (BMTG 1987) to calculate load capacity deficiency (LCD), clear deck deficiency (WD), over clearance deficiency (VCOD), and under clearance deficiency (VCUD).

The evaluation of the bridge condition deficiency (BCD) includes an assessment of the condition of each of the three primary elements of the bridge: (1) superstructure, (2) substructure, and (3) bridge deck. The deficiency points for each element, which are directly related to the individual condition ratings contained in the SIRS database, are given by the equation (BMTG 1987).

$$BCD = SPD + SBD + BDD \tag{6-1}$$

where

SPD = condition deficiency for the superstructure

SBD = condition deficiency for the substructure

BDD = condition deficiency for the deck

Other deficiencies are related to the remaining life, approach roadway alignment, and waterway adequacy. The estimated remaining life entered into the BMS database is developed by the system as a function of the condition ratings of the superstructure, substructure, and bridge deck. The maintenance life deficiency (RLD) is then calculated using an equation (BMTG 1987). The approach roadway alignment may be the source of additional deficiency points. This deficiency (AAD), which is directly related to the appraisal rating contained in the BMS database, is also given by an equation (BMTG 1987). The adequacy of the waterway is the final characteristic included as a source of deficiency points. This deficiency (WAD) is directly related to the appraisal rating included in the BMS database and is also given by an equation (BMTG 1987).

$$TDR = \emptyset\ [LCD + WD + VCOD + VCUD + BCD + RLD + AAD + WAD] \tag{6-2}$$

where

TDR = total deficiency rating

$\emptyset$ = factor dependent on the functional classification of the highway carried by the bridge as given in Table 6-2 (McClure and Hoffman 1990)

Table 6-2. Functional classification factors

Functional classification	Ø
Interstate	1.00
Arterial	0.95
Collector	0.85
Local	0.75

Table 6-3 (McClure and Hoffman 1990) lists the maximum deficiency points for each category and the listing conditions necessary to obtain TDR.

After the total deficiency has been established for all bridges, two costs will be requested: the replacement cost and the cost of rehabilitation. Also requested will be the number of deficiency points removed by the rehabilitation or replacement. Total deficiency ratings, combined with cost information and other factors, will yield listings of bridge rehabilitation and replacement projects, prioritized to enable effective management of the bridge system.

6.3. The Inventory System PONTIS (US)

An interstate initiative of the Federal Highway Administration to make an inventory of the North American bridges state by state and to facilitate upgrading and maintenance through founded economic analyses has been summarized in (Golabi) and (Thompson 1993) and gave birth to a "knowledge-based expert" system. However, basically the system is a very well

Table 6-3. Development of the Total Deficiency Rating (TDR) for bridges

Deficiency category	Maximum deficiency points in category	Listing conditions (1), (2)	Listing conditions (3)	Listing conditions (4)
LCD	70	70		
			$\Sigma \leq 80$	
BDD	50			
SPD	50	$\Sigma \leq 50$		
SBD	50			
WD	15	15	15	
VCOD	15	15	15	$\Sigma \times Ø \geq 100$
RLD	5	5	5	
VCUD	10			
		$\Sigma \leq 15$	15	
WAD	10			
AAD	10	10	10	
Maximum totals	285	180	140	100

developed database, since it did not (as of 1992) include structural evaluation modules as well as mathematical models of evolution prediction of the defects detected through the bridge service life. The rating of the condition of bridges is made by dividing them into their structural elements, whose condition is defined by a set of parameters quantified during the inspection and integrated into one of the standardized degradation levels predicted within the system. There is a bring-up-to-date module to calibrate the deterioration probability when new data are gathered as time goes by. Therefore, the system is initialized based on common sense and the experience of the consultants, as a basis of prediction in the first stage of the process, and afterwards starts "learning" with the new data automatically adjusted with the time-dependent degradation equations.

The main components of the system are the database, the modules of optimization (maintenance/repair and upgrading), and modules of projects integrated planning.

6.3.1. Database

The Federal Highway Administration keeps records for every bridge greater than 6.1 m in length on public roads. The records contain details such as location, owner, material, traffic, and condition, totaling over 100 pieces of information (Chase 1999). Condition ratings for each element of the bridge are assigned every two years, and are then aggregated into overall condition ratings for the superstructure, the substructure, and the deck: from 0 (intolerable and dangerous condition) to 9 (pristine condition). Appraisal ratings (structural evaluation, waterway, deck geometry, and under-clearance), from 0 (closed) to 9 (excellent), are also calculated.

Bridges are considered structurally deficient if any of the deck, superstructure, or substructure condition rating is equal to or less than 4 (poor), if the structural evaluation appraisal is equal to or less than 2 (intolerable, high priority for replacement), or if the waterway appraisal is equal to or less than 2. Bridges are considered functionally obsolete if the deck geometry appraisal is equal to or less than 3 (intolerable, high priority for corrective action), the under-clearance appraisal is equal to or less than 3, or the waterway appraisal is equal to 3. Using these criteria, there were 101,518 structurally deficient bridges (low load rating is the single most common reason; 22,591 reinforced or prestressed concrete) and 81,208 functionally deficient bridges (narrowness is the single most common reason), in a universe of 469,638 (225,958 reinforced or prestressed concrete) (Chase 1999).

6.3.2. Optimization of Maintenance/Repair

The aim of this module is to determine long-term procedures for each bridge within a predetermined network and as a function of its human and environmental domain, thus minimizing the long-term funding necessities for maintenance and repair, while simultaneously guaranteeing the structure's safety. It also allows for the quantification of the additional costs of not implementing the recommended procedure at the proscribed time. These costs can also be considered a benefit of making the recommended measures. If the budget is limited and does not allow taking all the recommended measures, the system permits choosing the bridges for which action is most necessary to obtain the maximum benefit within the available budget. The additional budget necessary to guarantee that the bridge network reaches a stationary level of exploitation costs is also provided. It is important to note that with this system, the priorities are not altered even when the available budget is.

As stated previously, the degradation condition of each bridge element is defined for standardized situations that are defined, in general, by dividing them into discrete intervals of a predetermined variable that unequivocally defines the instantaneous element condition. For each element there generally are five predictable standard situations and, for each one of them there are one to three possible actions, one of which is to do nothing at all. The system comprises about 160 possible elements even though a normal bridge may consist of only six to eight elements (Golabi).

The prediction model estimates the deterioration rate for each element and quantifies uncertainties connected to the same prediction, arriving at conclusions by using all the data collected in the meantime. At a first stage, the system determines the probability of a predetermined element complete the transition from its standard situation to the one immediately below during the time period under analysis. To do that, experienced engineers are asked a series of questions and are helped by the system to obtain adequate answers. This stage is expected to last for the first two years of implementation of the system. At a second stage, the system analyses the collected data to verify whether the values are compatible with the deterioration rates inferred and generates a new set of rates based on both the new and the old data. The new data are integrated into the system's memory and will be used in the next time period.

The benefits from the computerized bridge system (see Chapter 10) are defined as the savings by realizing all the maintenance/repair actions recommended during the period under analysis, as an alternative to postponing them for the next decision-making date. The costs are defined as the expenses that the organization responsible for the network must undergo to fulfill the recommended measures. The system determines the benefit/cost ratio for each bridge, rates them according to the decreasing order of that same ratio, and selects the bridges that it is possible to rehabilitate with the available budget.

6.3.3. *Optimization of the Capacity Upgrading*

The aim of this module is to maximize the benefits obtained from any new investment previewed in terms of a reduction in the operation costs. The actions considered include deck widening, vertical under-clearance increase, and a set of structural strengthening operations including seismic actions upgrading and reduction/elimination of foundations scour. The replacement is considered within the integrated planning. These actions are different from those associated with maintenance/repair in the sense that the latter only aim at keeping the bridge in the best possible condition without changing its service level.

The benefits are defined as the savings that are obtained in terms of operation costs (accident related costs, vehicles running costs, and time expenditure costs) by performing the upgrading actions recommended for the period under analysis as an alternative to postponing them until the next decision-making date. Accident-related costs are estimated as a function of the bridge vertical under-clearance, rating the general condition of the accesses, daily traffic volume, and an accident cost factor. The vehicle running costs depend on the fraction of heavy traffic that is detoured due to axle load or vertical under-clearance limitations of the bridge. They are determined as a function of the fraction of the daily traffic volume, the percentage of cargo vehicles in traffic, the detour length, and the average running costs of a cargo vehicle. Finally, the time expenditure costs are a function of the fraction of vehicles detoured from the bridge, the detour length, and traffic speed (Golabi). The system delivers a bridge rating list with the actions recommended based on the estimation of the benefit/cost ratio, whose costs are discounted from the available budget.

6.3.4. *Project Integrated Planning*

This module allows the combination of the results from the modules of optimization described previously and is used as a tool in the prediction of the future service conditions of the network, its necessity, and additional budgets needed as a function of predicted budgets, traffic evolution, and acceptable and optimum service levels.

For each period in the planning stage of 10 to 30 years, the prediction model is used to simulate the deterioration of each bridge in accordance with the standardized situations for each component. Based on this, a prediction of traffic evolution is made. The optimization modules are used to list and rate the bridges subject to future actions according to predicted budget limitations. The budget deficits and the bridges that are not rehabilitated during each period are transferred to the next time period and so on. The system "output" consists of the expected service level during the period under analysis and the necessary budgets and predicted deficits for different investment scenarios.

6.3.5. *Recent Developments*

Khan (2000) reports that PONTIS was already in use in 38 states (four more intended to implement the system) plus several other bridge authorities. Marshall et al. (1999), in which a comprehensive list of recent references related to North American bridge management systems is found, identifies a number of items that can be enhanced in PONTIS:

- the preservation model in PONTIS considers each condition unit independently; the program simulation algorithm should be improved to cause particular combinations of action types and element categories to trigger other actions on other element types;
- the program simulation should be modified to allow users to specify a set of rules to satisfy all maintenance needs on the structure, not just the needs identified by PONTIS;
- the program simulation should be modified to improve handling of user projects by recognizing planned future projects, particularly replacements.

6.4. The BRUFEM Bridge Rating System (US)

A complete bridge rating system, BRUFEM (Hays Jr. et al. 1990), was being developed in 1990 to be able to do the complete ratings of the vast majority of the bridges in Florida using a finite element model for the analysis.

BRUFEM contains three FORTRAN programs:

1. A pre-processor, BRUFEM 1, which develops a finite element model from a relatively small amount of input. The pre-processor prepares the input file\element program for a variety of concrete and steel bridges.
2. A finite element program, SIMPAL, is used to solve the model for the parameters created using the pre-processor. SIMPAL provides an efficient and powerful finite element capability and can be used with or without a sector processor; it outputs the results of the finite element analysis for printing and post-processing.

3. A post-processor, BRUFEM 3, reads unformatted SIMPAL output files and performs bridge rating calculations based on the appropriate service level or strength criteria. A graphic post-processor is also available.

The post-processor allows three rating options: (1) inventory rating, (2) operating rating, and (3) load factor rating. It uses standard load factors, resistance factors, and impact factors for inventory and operating ratings. For load factor ratings, the user inputs similar factors.

The rating factors previewed are cracking, fatigue, ultimate moment, and ultimate shear. They are expressed with respect to the live load. The rating factors indicate how many times the vehicle live load, which the user specified, can be increased until one of the criteria is reached at one point of the bridge. A rating factor of 1.0 means that, under the vehicle loading, some service or strength criteria are at their permissible limit.

However, there is no mention as to how these rating factors are introduced into the decision–making system, especially when rehabilitation is being considered. There also seems to be no allowance for the aging and degradation of materials and components.

6.5. The AASHTO Manual for Maintenance Inspection of Bridges (US)

The AASHTO Manual for Maintenance Inspection of Bridges (AASHTO 1978) is the standard by which publicly owned bridges in the United States are inspected and rated.

Nondestructive load tests were to be included in the revised manual (Lichtenstein and Minervino 1990) as a valuable tool in the evaluation of marginal bridges with marginal live load capacities. Serviceability issues and posting requirements for substandard structures were also to be formulated and included in the manual. The 1978 version of the manual proscribes two basic load-rating procedures: (1) allowable stress method (ASM), and (2) load factor design method (LFD). The introduction of the load and resistance factor design (LRFD) was to be included as an alternative. The 1978 version uses two categories in the load rating of bridges, namely, inventory and operating. The operating category is used to issue permits for overloaded vehicles. A review of this situation may suggest that overloads should be treated differently.

6.6. Structural Evaluation of Existing Bridges (AASHTO-US)

The document "Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges" (AASHTO 1989), which was published by the American Association of State Highway and Transportation Officials, has a great practical potential. In the United States, bridges are periodically rated according to their structural capacity. By using this manual, the rating can actually increase with time in bridges inspected regularly and with adequate maintenance programs based on present codes and in which the live loads limits are respected and checked by the traffic police authorities.

The rating is obtained by determining factor *R.F.*, which is defined as the ratio between the load level considered safe for the bridge and the one corresponding to the code characteristic live load (vehicle). It is used to justify future limitations traffic (postings) and repair/replacement decisions and is given by:

$$R.F. = \frac{\varnothing R_n - \gamma_D D}{\gamma_L L(1+I)} \tag{6-3}$$

where

$\varnothing$ = capacity reduction factor to take into account uncertainties in resistance due to variations in dimensions, materials properties, and design model

R_n = resistance design value

γ_D = partial safety factor concerning dead and live loads that do not correspond to the type of code vehicle under analysis

D = design value of action-effects due to dead loads

γ_L = partial safety factor concerning the code vehicle under analysis

L = design value of action-effects due to the code vehicle under analysis

I = dynamic coefficient

This formula does not present anything novel when compared with the formula used for new structures. It is in the determination of several factors that novel results arise. In fact, the factors depend on the results of in situ observations during inspection: pavement degradation level (potential increase of the dynamic coefficient), general condition of the structure (existence of degraded structural elements), and traffic type (composition, excessive loads origin, degree of traffic police control, etc.).

The dead and live loads that do not correspond to the type of code vehicle under analysis are determined in the usual fashion, with the exception of bituminous pavement, whose theoretical depth is increased by 20% or, alternatively, is measured in situ. The design value for the vehicle under analysis is either the code value or the value that corresponds to a well-determined real vehicle.

The dynamic coefficient I is made to depend on the pavement degradation level, and it varies between 0.1 (pavement not in need of repair) and 0.4 (pavement already not functioning as originally conceived).

The design resistance is determined assuming that the values predicted for the materials during the initial design stage were attained or, if they exist, using the average value from tests performed during construction. If the concrete looks deteriorated, a reduction in its resistance capacity is made unless in situ tests prove that it is not necessary. Losses in rebar-concrete adherence and in reinforcement cross section should also be taken into account (AASHTO).

The vehicle under analysis must be positioned to maximize its action-effects on the structure. A more refined structural analysis allows the partial elimination of the conservative point of view implicit in more simplified analyses and is considered specifically in a table that shows the corrective factor that multiplies the action-effects obtained in the chosen analysis type. The more refined the analysis, the smaller the factor. It will also decrease if in situ measurements are used.

The factor γ_D is considered equal to 1.2 (less than the values 1.35 or 1.5 normally considered in new structure design) and the factor γ_L varies between 1.30 (roads with small traffic volume, reasonable patrolling, and apparent control of excessive loading) and 1.80 (roads with big traffic volume, a significant quantity of infractions in terms of loading, and no effective control). A simultaneity factor is also considered for live loads as a function of the number of lanes.

The general condition of the superstructure, its redundancy (hyperstaticity), and the inspection and maintenance assiduity are taken into account in the determination of the factor $\varnothing$, as shown in Table 6-4 (AASHTO 1989).

Table 6-4. Reduction of existent concrete bridge resistance capacity factor Ø

Superstructure general condition	Redundancy Yes	No	Inspection Detailed	Irregular	Maintenance Regular	Intermittent	Factor Ø (reinforced concrete)
Good or	x		x		x		0.95
acceptable	x		x			x	0.85
	x			x	x		0.95
	x			x		x	0.85
		x	x		x		0.80
		x	x			x	0.70
		x		x	x		0.80
		x		x		x	0.70
Some	x		x		x		0.90
deterioration	x		x			x	0.80
	x			x	x		0.85
	x			x		x	0.75
		x	x		x		0.80
		x	x			x	0.70
		x		x	x		0.75
		x		x		x	0.65
Advanced	x		x		x		0.80
deterioration	x		x			x	0.70
	x			x	x		0.75
	x			x		x	0.65
		x	x		x		0.70
		x	x			x	0.60
		x		x	x		0.65
		x		x		x	0.55

If the factor R.F. exceeds 1.0, it is considered from a legal point of view that the bridge is safe for the vehicle under analysis.

This entire process is schematically presented in Figure 6-2 (AASHTO 1989).

The general methodology just described (AASHTO 1989) has been applied by the Florida Atlantic University in collaboration with the Florida Department of Transportation in the development of a comprehensive microcomputer-based analysis and rating system, which is succinctly described in Arockiasant et al. (1993).

6.7. Other BMS References (US)

Marshall and Söderqvist (1990), Little (1990), and Diaz et al. (1990) deal fundamentally with the database of computer bridge management systems. These databases were developed by firms of consultants and are being used or are going to be used in Massachusetts, Florida, and Texas.

Diaz et al. (1990) use a powerful routine for filtering data both to the screen and to written reports that deserves some attention. Little (1990) also provides a good selection of pre-

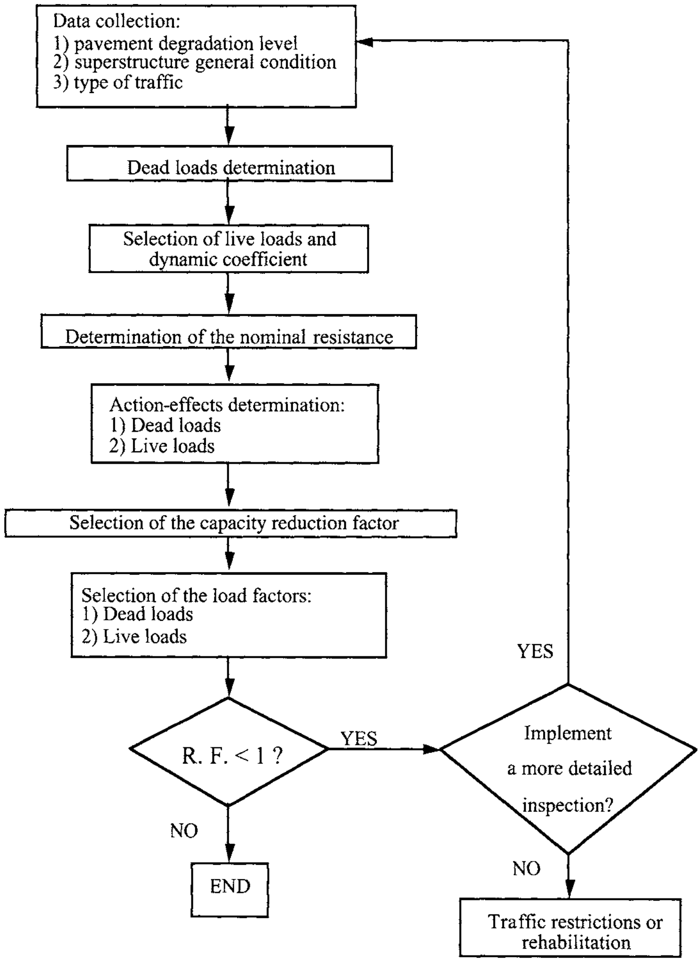

Figure 6-2. Determination of the reference factor (R.F. from Equation 6-3) for existing bridges

chosen reports from which the inspection schedule report should be outlined. It provides useful information for the preparation of future inspections (number of people and equipment necessary and the date of the next inspection) based on the bridge's geometric characteristics and data stored from previous inspections.

Frangopol et al. (2000) point out the need for future management systems that are based on reliability states instead of condition states. This would allow every maintenance and repair decision to be based on lifetime reliability and whole life costing. Table 6-5 (Frangopol et al. 2000) summarizes the reliability states proposed.

Frangopol and Estes (1999) describe the general methodology used to optimize the lifetime inspection/repair strategy for a deteriorating structure, consisting of the following 10 steps:

1. define the structure and the criterion that constitutes its failure;
2. specify how the structure deteriorates over time; develop a deterioration model;

Table 6-5. Definition of reliability states, attributes, and maintenance actions

Reliability States	5	4	3	2	1
Reliability Indexes	$\beta \geq 9.0$	$9.0 > \beta \geq 8{,}0$	$8.0 > \beta \geq 6{,}0$	$6.0 > \beta \geq 4{,}6$	$4.6 > \beta$
Attributes for Reliability	Excellent	Very Good	Good	Fair	Unacceptable
Maintenance Actions	Preventive 5	Preventive 4	Preventive 3	Preventive 2	Preventive 1
	Option 5a	Option 4a	Option 3a	Option 2a	Option 1a
	Option 5b	Option 4b	Option 3b	Option 2b	Option 1b
	Option 5c	Option 4c	Option 3c	Option 2c	Option 1c
	...	...	...	...	...

3. specify the inspection methods available to detect deterioration and quantify the detection capability and cost of these methods;
4. define the available repair options and compute their costs;
5. quantify the probability of making a repair if a defect is detected;
6. formulate the optimization problem based on the optimization criterion, failure constraints, expected service life of the structure, and any other constraints that should be imposed such as minimum and/or maximum time intervals between inspections;
7. use an event tree to account for all possible repair/no repair decisions that must be made after every inspection;
8. for a discrete number of lifetime inspections, optimize the timing of these inspections for a specific inspection technique;
9. repeat the problem for other numbers of lifetime inspections and inspection techniques to find the optimum strategy;
10. update the optimum strategy after every inspection using the new information provided from the inspection results.

References (ACI 1984), (AASHTO 1980), (AASHTO 1976) and (FHA 1971), may also be useful in understanding the American state-of-the-art computerized bridge expert systems.

6.8. The Ontario Ministry of Transportation BMS (Canada)

6.8.1. Introduction

The Highway Engineering Division of the Ontario Ministry of Transportation prepared four documents: (1) "Ontario Structure Inspection Manual-OSIM" (Reel and Conte 1989), (2) "Ontario Structure Inspection Management System (OSIMS) User's Manual" (Reel et al. 1988), (3) "Structure Rehabilitation Manual" (OMT 1988), and (4) "Structural Financial Analysis Manual" (Reel and Muruganandan 1990a). The inspection module is detailed in Reel and Conte (1989) and Reel et al. (1988), OMT (1988) deals with the maintenance (and rehabilitation) module, and Reel and Muruganandan (1990a) present a first step toward the decision module.

The system very clearly points to the widespread use of computers both in handling vast amounts of information stored and in making decisions at the inspection site and at headquarters.

6.8.2. The Database

6.8.2.1. Introduction

The Ontario Structure Inspection Management System (OSIMS) was developed to store and manage the inspection data collected during the detailed structural inspections of bridges and culverts on the Ministry of Transportation's highways in a database environment.

OSIMS is capable of creating, updating, and storing inspection rating data for structures owned and maintained by the Ministry of Transportation. The data are stored in a database and can be used to generate reports on condition ratings. General information for a structure is obtained from Ontario Structure Inventory System (OSIS) and is stored in the OSIMS database.

6.8.2.2. Options Available

The options available under OSIMS system are:

- add and update data: used to add new inspection data to the OSIMS database; can also be used to update existing data; the user can continuously add and update several site numbers in the same session;
- reports: generates reports based on data in the OSIMS database; the information in the reports can be chosen according to the user's needs;
- browse: allows the user to browse any regional OSIMS database;
- database maintenance: allows the user to rebuild, backup, restore, combine, and delete screens in the OSIMS database.

6.8.2.3. Users' Instructions

The program users' instructions are provided in the manual: how to access the system, notes on screen processing, initial entry of data, update existing data. and all the other options. The manual shows screen by screen how to enter and proceed within the program. In each screen, the variables that cannot be changed (they come directly from OSIS) are identified and their meanings are explained. For the fields that are intended to be filled or changed by the user, the type of entry and valid input data are given. For each option for the use of the program, a complete example is given showing every screen before and after the user has altered it. Specifically, it shows how to introduce field inspection data from the first detailed inspection to a structure and all subsequent inspections and how to obtain complete or condensed inspection reports.

Explanations on error messages and valid ranges of data for various fields and other useful information, as well as a list of structure names and their county site numbers can also be found.

6.8.3. The Inspection System

6.8.3.1. Support Manuals

In the past, inspectors relied on their own background and experience in reporting structural conditions. To modify this situation, the Ontario Structure Inspection Manual

(OSIM) sets standards for detailed routine inspection and condition rating of structures and their components. It provides a uniform approach for all structures on the Provincial Highway system in Ontario.

OSIM is divided into three parts. Part 1 (Technical Information) gives general details of inspection procedures, bridge components, material defects, and performance defects. Part 2 (Detailed Inspections) describes the requirements for detailed routine inspection and a condition rating for structures and their components. Part 3 (Programming Guidelines for Repairs and Rehabilitation) provides guidelines for the assessment and posting of structures and the need to carry out further detailed inspections, repairs, and rehabilitation of structures and their components, based on assigned material and performance condition ratings. The document starts with a list of general definitions of the most frequently used terminology.

The Structure Rehabilitation Manual (OMT 1988) is divided into four parts. Part 1 (Condition Surveys) describes how condition surveys are to be carried out. Appendices 1.A to 1.D include consultant agreements, standard forms, and standard legends. Part 2 (Rehabilitation Selection) describes methods of rehabilitation and shows how the information collected in the condition surveys is used to select the most appropriate method of rehabilitation for each different type of structural component. Part 3 (Contract Preparation) covers most of the activities likely to be encountered in rehabilitation contracts. Part 4 (Construction) summarizes the construction procedures used for each of the rehabilitation or repair methods included in the manual.

The Structure Rehabilitation Manual also covers procedures for the preparation of contract documents for the rehabilitation of various structure components.

6.8.3.2. Inspection Organization

The goal of a structural inspection is to ensure, within an economic framework, an acceptable standard for each structure in terms of structural safety, comfort, and convenience. The main objectives are to protect and prolong the service life of each structure; to identify repair and rehabilitation needs of each structure; to provide a basis for a structure management system for the planning and funding of the maintenance and rehabilitation of each structure. OSIM applies to the following structures: bridges, culverts and tunnels with spans over 3 m, movable bridges in fixed position, and retaining walls.

Structural inspections are divided into general inspections, detailed inspections, and condition surveys. The possibility of emergency inspections (of any of the types mentioned) is also considered when a structural component contributing to overall stability of the structure has failed or is in imminent danger of failure.

General inspections are based on direct visual observation and are either routine or nonroutine. Routine general inspections can take place daily (while traveling in the patrol vehicle), monthly (on foot), annually (detailed inspection of overhead sign supports), or biennially (all bridges with spans over 6 m). Nonroutine general inspections may take place whenever a problem has been identified in a structure or during or immediately after any of the following events: accident or vehicle collision with a structure, spring run-off, prolonged extreme temperatures, or other special circumstances.

Detailed inspections are thoroughly described in Part 2 of OSIM and include routine and nonroutine inspections. Routine detailed inspections are usually performed biennially even though the frequency may be increased for some types of structures and for those in which problems have been identified or at the request of a municipality.

The inspection equipment is light, simple to operate, and easy to read. Such equipment includes binoculars, camera, chalk and markers, flashlight, measuring tapes, plumb bob, safety belts, thermometers, screwdriver, and so forth. Special means of access (such as a boat, scaffolds, a bucket truck, etc.) may have to be used when strictly necessary.

Prior to field inspection, the inspector must review records available for the structure that were generated before fieldwork, including design and construction details, "as-built" drawings, previous inspection reports, correspondence, and details of repairs. The inspector should also prepare inspection forms and decide on the schedule for the inspection as well as any required special equipment including traffic protection devices. At the site, he should complete a brief overview of the structure and identify obstacles that may either interfere with the inspection or indicate a need for additional special equipment. Finally, it is necessary to check that all signs, temporary barriers, protective screens, and so on are in place.

For the inspection to proceed in a systematic fashion, the following must be entered into the General Information Sheet of Inspection Form: a list of names of the people present at the inspection, the date, weather and temperature, and all special equipment used. OSIM inspection forms for the particular components of the structure must be filled in, starting at the bottom and proceeding systematically to the top, making notes and recording observations. Also the inspector must make sketches where appropriate; take photographs, including an overview of the structure and areas where defects exist, noting the locations being photographed.

After the inspection, it is necessary to write all follow-up correspondence and reports and enter the date on the OSIM inspection forms into the OSIMS computer forms.

The standard inspection forms are given: the General Information Sheet of Inspection form; the inspection forms for each particular component of the structure (in which the condition ratings are noted as well as any particular remarks and recommendations); the Recommendations I form (additional work or investigations required); the Recommendations II form (repairs and rehabilitation).

Condition surveys require the measurement and documentation of all areas of defects and deterioration on a structure and, therefore, require access to all parts of the structure. Ladders, mobile platforms, snoopers (bucket trucks), scaffolding, or other means may provide access. Again, they are divided into routine and nonroutine condition surveys. Routine condition surveys are carried out every 5 years on a selected number of structures and include load carrying capacity assessment, surveys prior to repair or rehabilitation, surveys prior to approval of a load limit bylaw, and deck assessment by radar and thermographs. Nonroutine condition surveys are carried out on structures that are programmed for rehabilitation or when an evaluation is to be performed.

OSIM is to be used in conjunction with the Ontario Structure Inventory System (OSIS) as the inspection report is computerized and basic data about the structure must exist on OSIS. The data from the inspection forms are entered into the Ontario Structure Inspection Management System (OSIMS).

For condition surveys, detailed visual inspections of the structure and detailed surveys of the various structural components are carried out. The type of survey required is determined by the proposed rehabilitation, and this in turn controls the extent and detail of data to be collected.

A detailed condition survey is carried out only after a concrete component has been scheduled for rehabilitation. The procedure for carrying out a detailed condition survey involves the observation and recording of surface defects and may also involve a delamination survey, a covermeter survey, a corrosion potential survey, coring of concrete components (exemplified in Table 6-6) (OMT 1988), asphalt sawn samples, and physical testing of the concrete cores. Deck assessment by radar and thermography (DART) has been used to provide information on delamination and scaling for asphalt-covered bridge decks.

Table 6-6. Requirements for coring of bridge decks

	Number of cores required				Minimum number of cores	
	A		B			
Criteria	C	D	C	D	A	B
Electric potential more negative than −0.35 volts in an area between 0% and 10% of the deck	1 for each 100 m^2	2 for each 200 m^2	1 for each 500 m^2	1 for each 500 m^2	4	1
Electric potential more negative than −0.35 volts in an area between 10% and 25% of the deck	2 for each 100 m^2	1 for each 150 m^2	2 for each 500 m^2	1 for each 500 m^2	5	3
Electric potential more negative than −0.35 volts in an area bigger than 25% of the deck	3 for each 100 m^2	1 for each 100 m^2	3 for each 500 m^2	1 for each 500 m^2	6	3

A—first condition survey
B—other condition surveys
C—deck with a bituminous pavement
D—deck without a bituminous pavement

The detailed condition survey should be carried out no more than two years before the proposed rehabilitation.

A historical description of the protective treatments for structures in Ontario is provided. This information is vital in the selection of a rehabilitation method.

The requirements for data collection, sampling, and testing are based on the type of survey and the type of structural component being analyzed. The information needed is also listed.

The planning of condition surveys is also described. In advance of the field investigation, pertinent features of the structure should be identified and requirements for grid layout, sampling and data collection, equipment, manpower, and traffic control should be determined.

The latest version of the existing structural plan and as-built drawings should be reviewed for the following criteria: size and type of structure; unusual features in the design; structure location and topography at the site; direction and size of top reinforcing steel bars for cover meter check; location of utility ducts; location of stressing cables and void tubes on post-tensioned structures; year of construction; and details of previous rehabilitations. Also, previous routine detailed inspection forms should be reviewed for history of deterioration and for details of any previous repair. A copy of the latest inspection report should be obtained from OSIMS.

Some useful information can be obtained from a preliminary visit to the site.

In general, the crew will consist of a supervising professional engineer and two to four crew members. Additional personnel will be required for traffic control, concrete core drilling, and asphalt sawing operations.

When carrying out a detailed condition survey, data are usually collected with reference to grid lines. A 1.5 m × 1.5 m grid is used on bridge decks. A 1.0 m × 1.0 m grid is usually used on other concrete components but the size of this grid may vary depending on the dimensions of a particular component. A proposed grid layout should be established using existing structure drawings before going to the site. Copies of the grid sheets are used in the field to record data collected on surface deterioration, asphalt depths, half-cell potentials, concrete cover to reinforcement, and soffit deterioration.

The equipment used can be divided into three categories: (1) general tools and materials; (2) additional tools and materials for concrete components (electric generator, vacuum cleaner, electric core drill, voltmeter, copper-copper sulfate half-cell, covermeter, etc.); and (3) additional tools and materials for asphalt-covered decks (portable breaker/compactor, gasoline-powered saw, materials for filling holes, etc.). Whenever necessary, special access equipment such as a boat, ladder, or bucket truck will also be used.

The field procedures are also described. In the detailed visual inspection, the extent of the deterioration should be estimated but not measured. No physical testing is required. Color photographs should be taken of significant defects.

Before any measures are taken, standard forms are provided in which to document the data required and to calibrate the equipment used for checking the concrete cover and corrosion activity. A description of equipment used and the temperatures at the time of the test are also required. A thorough description of the various survey techniques is given: corrosion potential survey, concrete cover survey, delamination survey, concrete surface deterioration survey, expansion joint survey, drainage, concrete cores, and asphalt sawn samples. For each one, the technique is described and the procedure is given, including the various steps necessary to analyze the results and the minimum requirements (specifically the number of samples/cores required).

The sequence of operations, both for exposed concrete components and for bridge decks with an asphalt wearing surfaces, is also described. The first task is for the engineer to carry out a visual inspection of the condition of the structure, particularly those components that require a detailed condition survey, if this has not been performed on a previous site visit. This will enable the inspector to determine the scope of the survey and any unusual features or deterioration that will require special attention.

Cores should not be taken until corrosion potential testing is complete so that the concrete surfaces remain dry. If cores are to be taken in wheel tracks, they should be done early so that the concrete used to repair the core hole can set before the lane is opened to traffic. In early spring or late fall, when temperatures in the early morning are too low for potential measurements, the delamination survey and component inspection can be the first operation. The results of the corrosion potential survey and the delamination survey should be used to establish the locations for taking cores and sawn samples to be taken when the number is specified as a range in the Consultant's Agreement. The detailed visual inspection of components not requiring a detailed condition survey and photography may be carried out at the completion of the detailed condition survey, or simultaneously if crew size allows.

Laboratory testing of cores (physical testing) are described with a short description and the minimum requirements.

Guidelines are given for the preparation of the condition survey report. The material in the report is presented in the following order: table of contents; Structure Identification Sheet; key plan; summary of significant findings; Detailed Condition Survey Summary Sheet(s); survey equipment and calibration procedures; core photographs and sketches; core logs; sawn asphalt sample log; sawn asphalt sample photographs; and site photographs; drawings.

Guidelines are given for reviewing the condition survey report. Consulting engineers will normally carry out condition surveys for the Ministry. When the report is received, it is necessary for it to be reviewed and formally accepted. Omissions and anomalies should be resolved prior to approving final payment for the work. In this section, guidelines have been prepared in the form of a checklist for each section of the report that will aid in the review and assist in identifying inconsistencies in the data.

6.8.3.3. Classification Systems

OSIM presents the material defects that are normally found in concrete, steel, wood masonry, aluminum, asphalt pavements, and coatings. Each defect is briefly described and its causes are identified. Severity levels are established whenever possible.

For example, concrete defects include scaling, disintegration, erosion corrosion of reinforcement, delamination, spalling, cracking, alkali-aggregate reaction, and surface defects. To illustrate the severity levels (which are also proposed by the authors in order to classify the defects detected during inspections and make decisions within the scope of the maintenance/small repair decision subsystem), the concrete delamination severity classification is presented:

- light—delaminated area measuring less than 150 mm in any direction;
- medium—delaminated area measuring 150 mm to 300 mm in any direction;
- severe—delaminated area measuring 300 mm to 600 mm in any direction;
- very severe—delaminated area measuring more than 600 mm in any direction.

The material defects and their severity classification are abundantly documented with photos (Figure 6-3) (Reel and Conte 1989) and drawings.

The next sections of the same document are dedicated to the several parts into which a structure, its approaches, and adjoining area can be divided: streams and waterways, em-

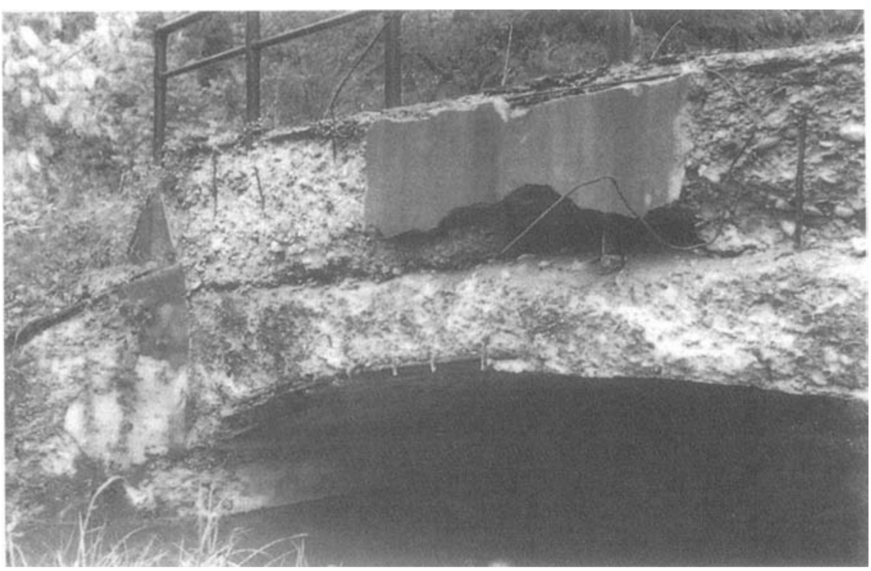

Figure 6-3. Exterior surface of a concrete deck suffering from highly advanced disintegration

bankments and slope protection, substructures, bearings, joints, superstructures, deck components, railing systems, structural steel coatings, signs, and utilities. In each section, a description of the element is given with a classification of the several types of each element that may occur in order to standardize the denominations. The defects are divided into material defects and performance defects.

Material defects are those defects considered in the previous section and are related to the building materials regardless of any further consequences to the structure. Performance defects are problems that may impair the structure as a whole or in so that it does not function as it is supposed to. A material defect may have the same causes and symptoms as a performance defect. Both the classification of the types and the material and performance defects are illustrated by the use of schematic drawings.

Historical information on design and evolution of live loads in Ontario and Canada in general is detailed next.

The condition rating systems that are the basis of the decision system at the maintenance/small repair level (OMT 1988) are presented. The material and performance condition rating systems are numerical systems in which a number from 1 to 6 is assigned to each component of the structure based on observed material defects (factor MCR) and the resulting effect on the ability of the component to perform its function in the structure (factor PCR). In addition, the number 0 (zero) is assigned to a component when it does not exist in the particular structure under inspection and the number 9 is assigned when a component is not visible or accessible at the time of inspection.

All components of a structure are classified as primary, secondary, or auxiliary. The classification is generally based on traditional behavior of the components. All components are rated for material and performance defects.

The material condition rating system for the components of a structure represents the condition of the component based on observed defects in the materials of the component. General guidelines based on the severity and extent of observed defects are given in Figure 6-4 (Reel and Conte 1989) for primary, secondary, and auxiliary components.

Additional guidelines for the material condition rating of components are given for material defects that cannot be generalized and for exceptions to the general guidelines.

The material condition rating should represent the worst observed material condition of the component.

The performance condition rating system for components of a structure describes the condition of the component based on its ability to perform its intended function within the structure. In most cases, the performance defect of a component is closely related to, or attributable to, defects in the component materials because material defects often lead to performance defects. The severity of the performance defect is not necessarily the same as the severity of the material defect. In some cases, performance defects exist as a result of defects in design or construction and may not be directly related to material defects. Also, performance defects in a component may be the result of unexpected behavior of the structure or may be due to performance defects in other components of the structure. General guidelines based on the percentage reduction in the capacity of the component to perform its intended function are given in Table 6-7 (Reel and Conte 1989) for primary, secondary, and auxiliary components.

Additional guidelines for the performance condition rating of components are given for performance defects that cannot be generalized and for exceptions to the general guidelines.

The performance of the structure is directly related to the performance of the primary components. The lowest performance condition rating of the primary components should

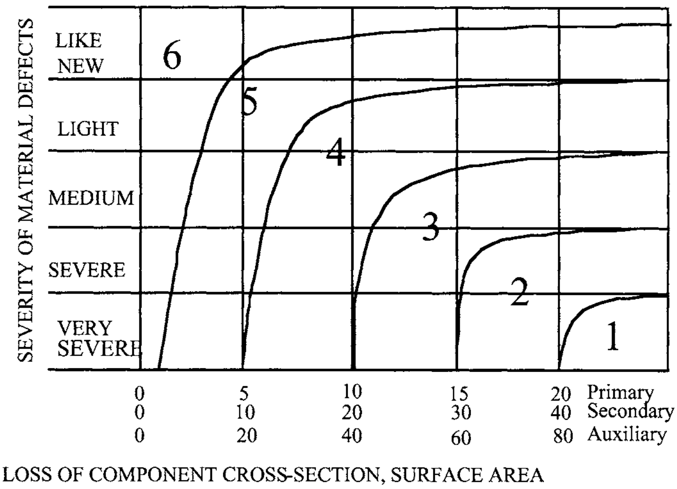

Figure 6-4. Material Condition Rating (MCR) of components

be the performance condition rating of the structure (factor PRCS). Ratings of different primary components are not combined to arrive at the rating for the structure.

The several parts into which a structure, its approaches, and adjoining area can be divided are described. The material and performance condition are particularized for each part of the structure. Examples of defects and condition rating are given and illustrated with photos (Figure 6-5).

6.8.4. The Decision System

The decision system is roughly divided into two subsystems: (1) maintenance/small repair, and (2) repair/upgrading/replacement.

Table 6-7. Performance condition rating (PCR) of components

Rating	Performance condition of components	Guidelines for approximate reduction in the capacity of the component to perform its intended function		
		Primary components	Secondary components	Auxiliary components
6	Very good	0% to 1%	0% to 2%	0% to 5%
5	Good	1% to 5%	2% to 10%	5% to 20%
4	Fair	5% to 10%	10% to 20%	20% to 40%
3	Poor	10% to 15%	20% to 30%	40% to 60%
2	Urgent, Inadequate	15% to 20%	30% to 40%	60% to 80%
1	Critical, Inadequate	Over 20%	Over 40%	Over 80%

Figure 6-5. Defect in column rated as MCR 2 (serious delamination and spalling in more than 15% of the surface, with reinforcement in full view) and PCR 4 (the deterioration results in a 5% loss in the load-bearing capacity)

6.8.4.1. Maintenance/Small Repair

The need for repair and rehabilitation should be based on material (MCR) or performance (PCR) condition ratings of the structure or the component being considered according to Table 6-8 (Reel and Conte 1989).

6.8.4.2. Repair/Upgrading/Replacement

The need for concrete deck condition surveys should be based on the material condition rating of concrete decks according to Table 6-9 (Reel and Conte 1989).

The need for posting the maximum weight of a structure should be based on the performance condition rating of the structure as inspected (PCRS) in accordance with Table 6-10 (Reel and Conte 1989).

Table 6-8. Suggested time periods for repairs or rehabilitation

Material or performance condition rating (MCR or PCR)	Suggested time periods for repairs or rehabilitation
6	Repairs should not be required for 10 years
5	Repairs required in 6 to 10 years
4	Repairs required in 3 to 5 years
3	Repairs required in 1 to 2 years
2	Repairs required within 1 year
1	Repairs required immediately

Table 6-9. Suggested guidelines for concrete deck condition surveys

Material condition rating (MCR) of concrete deck	Suggested time periods for concrete deck condition surveys
6	Condition survey not required for at least 8 years
5	Condition survey required in 4 to 8 years
4	Condition survey required in 1 to 3 years
3	Condition survey required now
2 and 1	May not have sufficient time to carry out condition survey

Ministry experience and research has shown that most bridges are able to carry heavier weights than the loads for which they were designed. Evaluations are not to be requested just because a bridge was built at a time when design loads were lower than present-day loads.

With the constant increase of traffic loads, the question of strengthening existing bridges is frequently under consideration. Because this is a very costly process, real resistance is usually greater than design resistance and real loads are smaller than design loads. Clause 12, "Existing Bridge Evaluation," was introduced in the Canadian Standard CAN/CSA-S6-88 "Design of Highway Bridges" and is described in Buckland (1990).

The limit state inequation becomes:

$$\alpha_R \geq \alpha_L(1+I) + \alpha_{D_1}D_1 + \alpha_{D_2}D_2 \tag{6-4}$$

where

R = resistance

L = live (traffic) load effects

I = dynamic load allowance

Table 6-10. Suggested guidelines for maximum weight posting requirements

Performance condition rating (PCRS) of structure	Suggested guidelines for maximum weight posting requirements
6 and 5	Maximum weight posting would not normally be required.
4	Maximum weight posting would not normally be required. An evaluation according to OHBDC should be requested only if considered essential.
3	Maximum weight posting may be required. An evaluation according to OHBDC should be recommended.
2	Maximum weight posting required immediately based on visual inspection.
1	Maximum weight posting required immediately based on visual inspection. Alternatively, the structure may have to be closed to traffic until repair or replacement is undertaken.

D_1 = effects of all dead loads except unmeasured asphalt

D_2 = effect of unmeasured asphalt

α = the appropriate factor

$\alpha_R = U\phi$

ϕ = resistance factor given in the code

U = adjustment factor to "fine tune" the values of ϕ and increase their accuracy

The live load rating factor, *LLRF*, is given by:

$$LLRF = \frac{U\phi R - \left(\alpha_{D_1} D_1 + \alpha_{D_2} D_2\right)}{\alpha_L \, L(1+I)} \tag{6-5}$$

If the element of the bridge being considered can carry exactly the load required, $LLRF = 1.0$. If $LLRF > 1.0$, there is a spare capacity, and if $LLRF < 1.0$, the element is substandard.

Live loads are made to depend on three factors: (1) how well the live load is known, (2) the amount of warning likely as collapse is approached, and (3) the consequences of failure.

Four kinds of traffic are defined: NP (non-permit), PS (permit, single-trip), PM (permit, multiple-trip), and PC (permit, controlled). From NP to PC, the live load increases, but so does the accuracy with which they are known.

Figure 6-6, **b.** (Buckland 1990) represents a case in which the traffic load is well known, for example, an escorted weighed vehicle at crawl speed with no other traffic on the bridge. In this case, the safety factor can be reduced without changing the shaded area of the probability of failure. In other words, despite the reduction of the "safety factor", the risk is unchanged and the bridge's safety is not altered from the condition in Figure 6-6, **a.** (Buckland 1990).

The probability of failure is associated with β, the reliability index, whose minimum values are given in Table 6-11 (Buckland 1990).

INSP1, INSP2, and INSP3 denote the levels of inspection to which the bridge has been subjected. It is assumed that all bridges have routine inspection. INSP1 refers to elements that are not possible to inspect, INSP2 is routine inspection, and INSP3 is for critical or substandard elements that have been inspected by the evaluator (who may notice clues about structural performances).

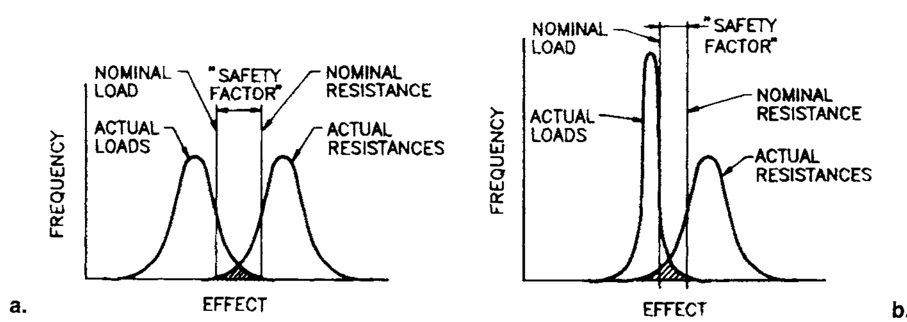

Figure 6-6. Relationship between "safety factor" and variability of loads and resistances with the real safety (i.e., probability of failure) kept constant: **a.**, loads and resistance equally variable; **b.**, loads known with more precision than in **a.**

Table 6-11. Target reliability index β for NP, PM, and PS traffic

		Inspection level		
System behavior	**Element behavior**	**INSP1**	**INSP2**	**INSP3**
S1	E1	3.75	3.50	3.50
	E2	3.50	3.25	3.00
	E3	3.25	3.00	2.75
S2	E1	3.50	3.25	3.25
	E2	3.25	3.00	2.75
	E3	3.00	2.75	2.50
S3	E1	3.25	3.00	3.00
	E2	3.00	2.75	2.50
	E3	2.75	2.50	2.25

S1, S2, and S3 relate to system behavior. S1 rating is when failure of one element can lead to total collapse; S2 rating is when one element failure will probably not lead to total collapse (e.g., multiple load paths); and S3 is the rating when element failure leads to local failure only.

E1, E2, and E3 refer to behavior of the element under analysis as it fails. An E1 element is subject to sudden failure with little or no warning. An E2 element also fails suddenly but will retain post-failure capacity. An E3 element is subject to gradual failure with warning of failure probable.

Once β has been selected, the load factors are selected from Table 6-12 (Buckland 1990).

β is a function of the projected target annual (not lifetime) probability of failure P. P is based on life safety criteria and is defined as:

$$P = A\,K/\,(Wvn) \tag{6-6}$$

where

A = activity factor, a measure of risk associated with the activity (i.e. driving a car); equal to 3.0 for NP, PM and PS traffic and to 10.0 for PC traffic;

K = calibration factor equal to 10^{-4};

W = warning factor equal to 1.0 for no failure warning expected;

n = number of people at risk; equal to 10 for NP, PM, and PS traffic on spans up to 100 m and to 1 for PC traffic (other traffic kept off the bridge).

For elements supporting larger or more important bridges, n can be increased but β is insensitive to n and a value of 10 was chosen as a typical value for most bridges.

These values lead to:

for NP, PM, and PS traffic	$P \approx 9.5 \times 10^{-5}$	$\beta \approx 3.75$;
for PC traffic	$P \approx 1.0 \times 10^{-3}$	$\beta \approx 3.25$.

Table 6-12. Live and dead load factors

Load	Load factors	Target reliability index β						
		2.25	2.50	2.75	3.00	3.25	3.50	3.75
NP	α_L	1.31	1.37	1.43	1.49	1.56	1.64	1.71
PM	α_L	1.01	1.05	1.09	1.14	1.19	1.25	1.31
PS	α_L	1.13	1.18	1.23	1.29	1.35	1.41	1.47
PC	α_L	1.05	1.09	1.14	1.18	1.23		
D1	α_{D_1}	1.05	1.07	1.08	1.09	1.10	1.12	1.13
D2	α_{D_2}	2.32	2.51	2.70	2.90	3.09	3.28	3.47

These values of β are reduced in a systematic way to account for improved failure warnings (which come from inspection and ductile behavior) and for the consequences of failure that are other than catastrophic.

The Structure Rehabilitation Manual (OMD 1988) provides guidelines on the appropriate strategy to be adopted in the repair, rehabilitation, or replacement of structure components to ensure that the most effective and economical rehabilitation method is selected. The selection process should take into consideration the data collected from the condition surveys, cost estimates, and the performance of rehabilitation methods and materials as well as other available information and data.

The factors that influence the selection of the rehabilitation method are defects and deterioration, load-carrying capacity, financial analysis and availability of funds, importance of the structure, future construction programs, type of structure and geometry, functional obsolescence, contractor expertise, and social and environmental concerns.

Prior to developing a rehabilitation strategy, the engineer needs to review the data available: condition survey reports; existing structure drawings; strength evaluation reports; inspection, maintenance and rehabilitation reports; and site conditions.

Several particular structural components are dealt with: concrete structure components; concrete bridge decks; cracking in concrete; and expansion joints, bearings, and deck drainage. Further updates include sections concerning components such as structural steel, timber, aluminum, masonry, as well as streams, embankments, and slope protection.

Guidelines for selecting the most suitable rehabilitation materials for each structural component and methods are provided. These guidelines are then summarized in tables, such as Table 6-13 (OMT 1988), and flow charts, like the one presented in Figure 6-7 (OMT 1988).

The Structure Rehabilitation Manual also contains miscellaneous design considerations on traffic control, roadway protection, jacking, environment, utilities, and engineering survey concerning the implementation of rehabilitation work. Forms for finalizing structure rehabilitation recommendations are presented.

The Structure Rehabilitation Manual discusses the preparation of contract drawings and documents that make up the structural portion of a rehabilitation contract. For each rehabilitation work undertaken (e.g., scarifying), guidelines are given for the tender items along with special provisions to be considered on quantity calculations and on contract drawings.

Table 6-13. Excerpt of the guidelines for selection of rehabilitation methods (all components excluding bridge deck surfaces)

Criteria	Most suitable rehabilitation method(s)	Comments
Concrete surface in good condition	Do nothing	Continue routine inspection and maintenance
Concrete lightly scaled and *not* exposed to chlorides	Do nothing	Correct any drainage deficiencies and monitor scaling
Concrete poorly air entrained and exposed to chlorides	Concrete sealant	Repair medium-to-very severe scaled areas; repaired areas do not need to be sealed
Concrete lightly scaled and exposed to chlorides	Concrete sealant	Correct any drainage deficiencies
Concrete cover to uncoated reinforcing steel is less than 40 mm and concrete exposed to chlorides	Concrete sealant	Concrete surface must be in good condition
Concrete cover to uncoated reinforcing steel is less than 40 mm and concrete surface shows areas of cracking, rust stains and exposed re-bars	Concrete overlay, refacing, encasement or replacement	If corrosion of reinforcement is extensive, replacement of component may have to be considered based on financial analysis
Concrete surfaces that have extensive areas of medium to severe scaling or erosion	Concrete overlay, refacing or encasement; shotcreting	Correct any drainage deficiencies that may have caused deterioration; most cost effective method should be chosen
Spalling and delaminated concrete and concrete with areas of medium to very severe scaling or erosion	Concrete patching; Shotcreting	Guidelines for selecting the most suitable patching material and/or shotcrete are given in another table

6.8.4.3. Financial Analysis

The Structural Financial Analysis Manual (SFSM) (Reel and Muruganandan 1990a) gives some guidance on financial analysis as a tool for decision making that concerns structural rehabilitation. It sets the basis of the decision system at the repair/upgrading/replacement level, as proposed by the authors later on (Reel and Muruganandan 1990b).

The financial analysis is relevant to decision making at both the project level and the network level. At the project level, the costs of alternative levels of improvements to a bridge are compared to determine the most economical option for the bridge. At the network level, the analysis involves resource allocation between different bridges in locations across the province.

The goal of structural financial analysis is to enable decision makers to make a rational choice regarding rehabilitation options within an economic timeframe. Its main objectives are to assist the regional staff and the head office staff to arrive at the most economic solution for rehabilitation or replacement of bridges and to ensure that funds are expended in an effective manner.

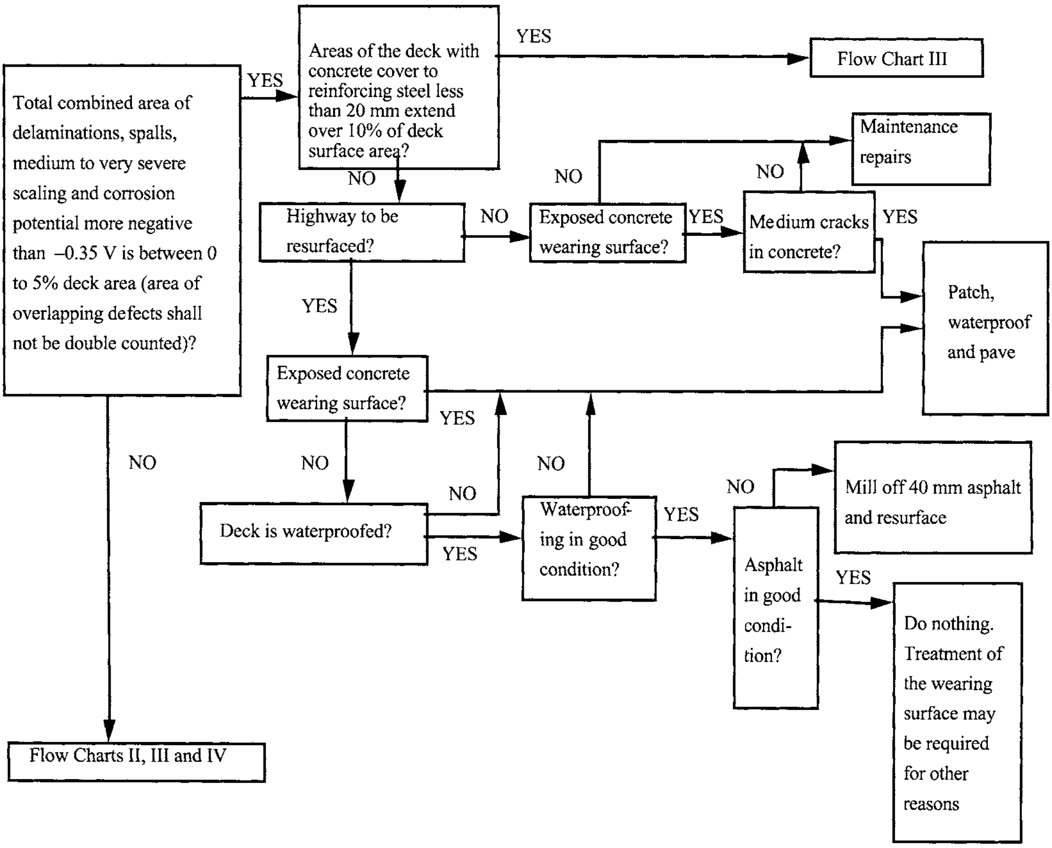

Figure 6-7. Selection of deck rehabilitation methods; deck in good condition

SFAM Financial Analysis

The technique used is the present value analysis. In general, when comparing alternatives, the alternative with the lowest present cost is the one preferred. This allows for the comparison of alternative schemes on an equitable basis. Variations in cost may occur as a result of general inflation or deflation. All cost estimates of engineering projects should be carried out in constant dollars. Therefore, these cost estimates should not be increased for inflation.

At the project level, the financial analysis may be undertaken for the whole bridge or for a major component of it. The present value analysis may be carried out at four levels of sophistication depending on the availability of reliable information pertaining to various costs. The reliability increases from Level 1 to Level 4.

Level 1 analysis uses the capital costs involved only during the assumed life cycle. The residual value of the structure and the maintenance costs are neglected. Level 2 analysis uses the capital costs and residual values but excludes maintenance costs. Level 3 analysis uses all associated costs including capital costs, residual values, and maintenance costs. Level 4 analysis is the same as Level 3 but includes a probability analysis that allows for uncertainty with respect to costs. It must be said that this probability analysis is very crude as it consists only of attributing probability percentages to different cost levels and finding a mean value. When the magnitude of the discount rate to be used is in question, the analyst can perform a sensitivity analysis by varying the discount rate by ±3%. This can be done at all four levels, which will assist the decision maker in arriving at a more cost-effective solution.

Concerning level 4 analysis, the following can be concluded:

- no allowance is made in SFAM for load testing;
- the inspection costs are not considered in SFAM;
- the repair costs are part of the capital costs in SFAM;
- failure costs and benefits are not considered in SFAM (that option is acceptable for comparative studies if those costs and benefits are considered the same for all options, which is not true quite often).

The parameters required for the financial analysis are defined as those parameters related to the rehabilitation or replacement of each alternative (capital costs, life cycle, residual life, future maintenance costs), parameters related to the existing condition of the structure (estimated residual life without remedial work), and discount rate (rate recommended by the Treasury Board of Canada, variations in discount rate).

The probability analysis (level 4) is based on the knowledge of probabilities of occurrence. In an environment of uncertainty, determining cost estimate is not accurate. Assigning probabilities to various estimated costs reduces the degree of uncertainty. If $c_1, c_2, \ldots, c_n$ are in estimated costs with probabilities of occurrence $p_1, p_2, \ldots p_n$, then the expected cost C is given by:

$$C = p_1 c_1 + p_2 c_2 + \cdots + p_n c_n \tag{6-7}$$

The PRVAL Program

PRVAL is a template overlay developed to perform financial analysis for bridge rehabilitation projects, in a worksheet format. There are four different options available to carry out the financial analysis at different levels of sophistication (the four levels referred to previously). Sensitivity analysis can be carried out for each level by varying the discount rates by ±3%.

The PRVAL input screens are shown in the manual, and comments are made about the several fields that the user must fill in. Some basic notions on the present value analysis and on the residual value analysis (using the second cycle replacement method) are given. The effects of inflation in the financial analysis are explained. A list of the assumed lifespans of several materials and components is given. An example of a financial analysis at all four levels is presented, showing the input necessary in each case, the results of the analysis, and how to interpret them.

The COSBEN Software

Subsequent to SFAM (Reel and Muruganandan 1990a), Reel and Muruganandan presented further developments of the proposed financial analysis. A new program, COSBEN, was developed to perform the incremental benefit ratio analysis at the project and network levels.

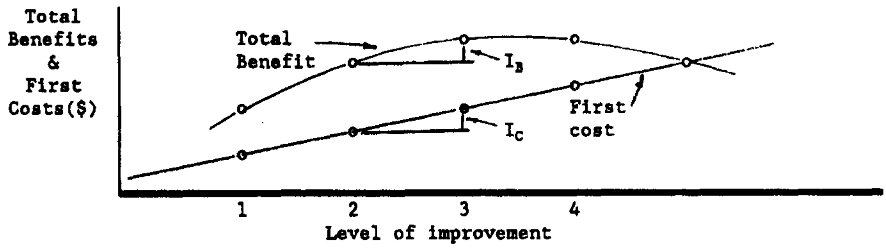

Figure 6-8. Benefits and costs versus levels of improvement

COSBEN is based on the theory of incremental benefit/cost ratio analysis. This ratio is obtained by dividing the additional benefits realized in moving from one improvement alternative to another by the corresponding difference in costs. This method not only optimizes the selection of alternatives efficiently but also ranks the projects in order of priority.

Figure 6-8 (Reel and Muruganandan 1990b) shows the total benefit and first cost curves plotted for the various alternatives for a structure. The slopes of the benefit and first cost curves support the theory of diminishing returns. For a particular level of improvement, there exist points on the curves, where the slopes of the two curves are equal, i.e. $I_B = I_C$. At this level of improvement, the net benefit is a maximum (Figure 6-9) (Reel and Muruganandan 1990b). Any option below this level where I_B/I_C is greater than one is a desirable option.

The procedure used is to list rehabilitation alternatives in the order of increasing costs and calculate the incremental benefit/cost ratios. Alternatives for which the incremental benefit/cost ratio falls below 1 are discarded. Usually, as the level of cost increases, the incremental benefit/cost ratio decreases. However, if the ratio I_B/I_C increases with an increase in cost, an adjustment is made for that particular option. The options are sorted in descending order of I_B/I_C. For an unlimited budget, the most net beneficial alternative is the one with the largest initial cost and whose incremental benefit/cost ratio is greater than or equal to 1. For a limited budget, the order of preference is the order from the highest to the lowest incremental benefit/cost ratio.

Benefits are added to the parameters required for the financial analysis. They are classified into two categories: agency benefits and user benefits.

$$\text{Agency benefit} = PV_R - PV_1 + C_1 \tag{6-8}$$

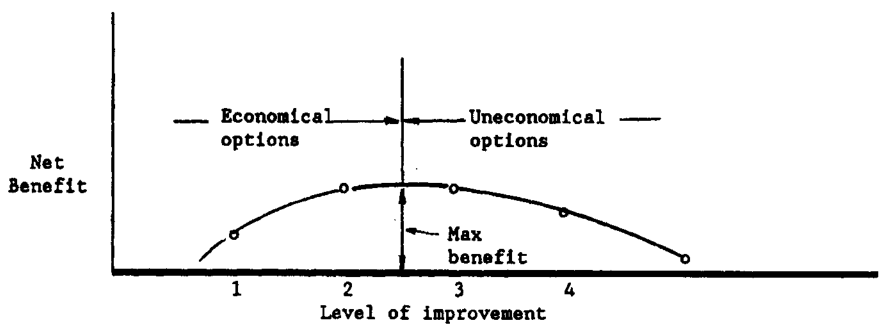

Figure 6-9. Net benefits versus levels of improvement

where

PV_1 = lifecycle cost of the rehabilitation option

PV_R = lifecycle cost of the replacement option

C_1 = initial cost of the rehabilitation option

User benefits of a bridge rehabilitation option are the reduction in cost to the users and to society as a result of the rehabilitation. In determining user benefits, it is assumed that deficiencies will be eliminated when the bridge is replaced. The reduction in the number of accidents based on a certain type of improvement is used as a measure of user benefit for that type of improvement. In the standard management system presented in Chapter 8, no allowance has been made for accidents except those due to structural failure of the bridge. The effect of bridge width on accident rates is shown in Figure 6-10 (Reel and Muruganandan 1990b).

Two common methods are used to place a monetary value on an accident. The human capital approach takes into consideration the direct and indirect costs. It does not consider the intangibles offered to society or loss in quality of life. The willingness to pay approach includes estimates of the value of life. As such, the willingness to pay approach is more conservative.

User costs associated with a functional restriction (functional failure costs in the authors proposal) are determined by the length of time that the restriction has existed. If a bridge is subjected to functional restrictions because of substandard conditions, such as load or clearance restrictions, then there is a need for detouring a certain class of vehicles.

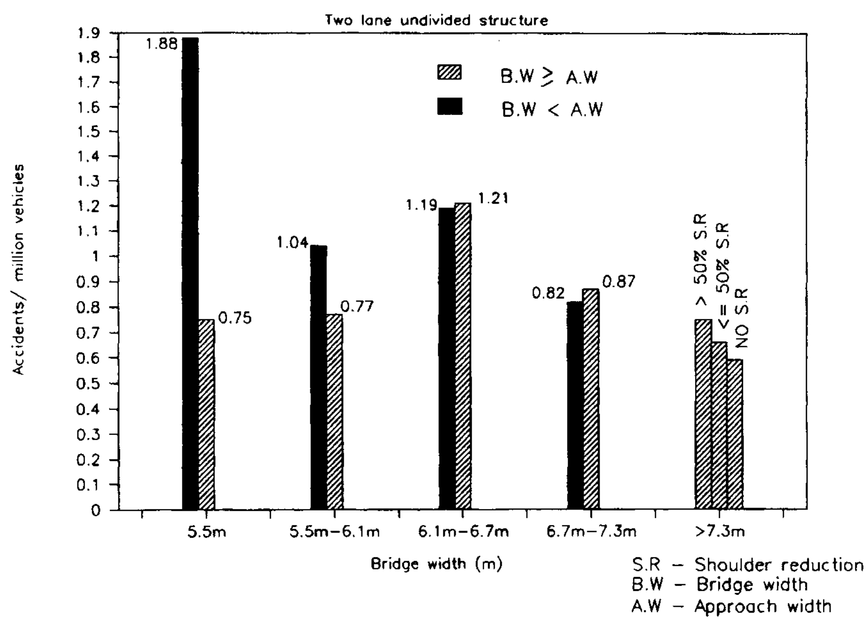

Figure 6-10. Accident rate versus bridge width

6.8.5. Further Canadian BMS References

Reel (1988), Manning and Reel, Reel and Conte, and RTAC (1977) are all related to the Ontario Management System and provide some very useful information.

Ryell et al. (1982) present the results of a study of concrete bridge deck conditions. Bridge decks are the most vulnerable component of bridges and thus require rehabilitation work more often. A deck slab condition rating (CR) is defined: CR1—replace; CR2—immediate rehabilitation; CR3—rehabilitate in 1 year; CR4—rehabilitate in 2 to 5 years; CR5—rehabilitate in 6 to 10 years; CR6 —rehabilitation not required. It will be apparent from the description of physical deterioration for same CRs that a bridge deck could fall into more than one category. The most sensible approach appears to be to assign the lowest CR for a specific deck slab. The authors believe that the establishment of priorities for bridge deck replacement or rehabilitation is influenced by many factors not considered in the condition ratings.

An interesting piece of information in the same reference is that Ministry costs for rehabilitation contracts (more or less equal to structural assessment costs within repair costs in the authors approach) represent 12% of total repair costs. Ministry costs include those for pre-engineering work and include condition surveys and inspections, design work and preparation of the contract package, and supervision and inspection of the rehabilitation contract.

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CHAPTER 7

Bridge Management Systems Outside North America

7.1. Bridge Management in Surrey (U.K.)

The document of Palmer and Cogswell (1990) refers to bridge management in the Surrey County Council. STREG is a computer management system that stores information about individual structures. It is a batch system on the Unisys mainframe computer that generates standard or provisional reports as necessary. Inspection forms are prompted to a predefined template, completed on site, returned, and maintenance work decisions are prioritized and programmed; maintenance lists are then produced. There are, to some degree, coarse financial records. It would appear that a decision system has not been included in this program.

The next development involved converting STREG to BRIDGIT, a more advanced management system consisting of eight modules, designated as system, inventory, inspection, maintenance, rehabilitation and replacement, analysis, models, and reports (Khan 2000). To obtain a quantitative measurement of the condition of individual and total bridge stock, the condition of each element is given at the time of inspection based on a scale of 1 to 5, with each classification being well defined. A multiplier is used to obtain the condition factor of each element of the bridge. Each element is given a location factor based on its structural importance. The bridge itself is given a road factor, which is dependent on class that is, motorway, A-class road, and so forth. These three factors are multiplied together to give a priority rating to that element of the bridge. These are totaled to give a condition factor. Life costing techniques are used to determine the most cost-effective solution for a particular structure during its life cycle. The Department of Transport recognized this necessity in its publication BD 36/88, which gives average maintenance costs.

Further development of the system gave rise to the COSMOS system, an interface between the user and an Oracle database system, which interacts with a series of maintenance subsystems within the bridge management system (Figure 7-1) (Brooman and Wootton 2000).

7.2. Department of Transport Rehabilitation Strategy (U.K.)

Holland and Dowe (1990) and Baker (1999) describe the Department of Transport's rehabilitation strategy over a 15-year time period. The work included in the program has been grouped as follows: steady-state maintenance, assessment and strengthening, and upgrading substandard features.

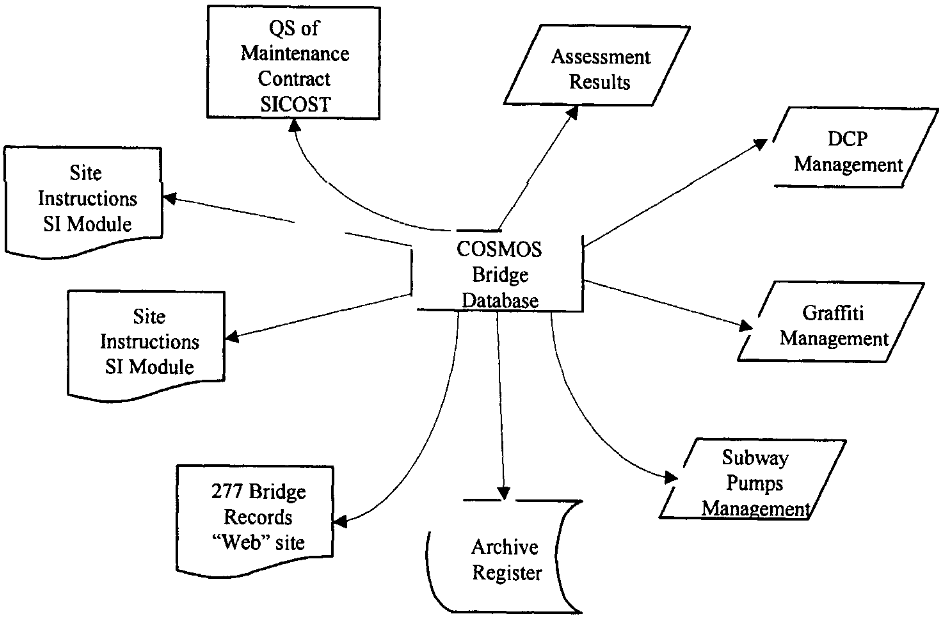


Figure 7-1. Surrey maintenance subsystems outside of the COSMOS bridge inventory and management system

Details of all department structures are maintained in a computerized database, which also contains information from inspection reports and details regarding maintenance expenditures. The bridges are subject to a general inspection every 2 years and to a more rigorous principal inspection every 6 to 10 years. However, such inspections involve close visual inspection and thus record only damage or deterioration that can be seen. Special inspections of particular areas or defects that cause concern are also carried out as necessary. To obtain better information about the overall condition of its concrete bridges, the department commissioned a study by consultants of 200 randomly selected but representative concrete bridges. In addition to a visual examination of each structure, half-cell potential, depth of cover, and depth of carbonation measurements were taken. Samples were taken for analysis of the cement, chloride and sulfate content, and cores were also examined petrographically to check for the presence of or susceptibility to alkali-silica reaction.

Dawe (1993) reports on the work that was done by the Department of Transport to give engineers as much guidance as possible in carrying out the bridge assessment and strengthening program through the development of the assessment version of the U.K. national design codes.

The 15-year Highways Agency New Bridge Rehabilitation Program that started in 1988 is near its end and a new program is already being prepared based on the following outline in Das (1999) and Das (2000).

In terms of inspection strategy, all bridges have to undergo a general inspection (visual inspection of all parts) every 2 years. Particular inspections of critical elements also must be carried out at prescribed intervals. During a 15-year period, every bridge in the network is to undergo a general assessment (present state assessment of all parts). When an element is assessed to be substandard at the time or is likely to be deemed substandard within the next 15 years, a whole life assessment must be carried out. Particular assessments (repeat assessments of critical elements) must be performed after each particular inspection.

These changes in the inspection strategy arise from the fact that the existing procedures have a number of shortcomings (Narasimhan and Wallbank 1999). Principal inspections require that the whole structure be inspected closely regardless of the likelihood of deterioration of any component. Also, little use is made of nondestructive testing. Table 7-1 (Narasimhan and Wallbank 1999) summarizes the existing and proposed inspection procedures.

A new bridge condition index (BCI), to be used by the Highways Agency, is being developed, tested, and adjusted (Blakelock et al. 1999):

$$BCI = 100 - F_1 \times [F_2 \times (E_{fp} \times S_f)/N_p + F_3 \times (E_{fs} \times S_f)/N_s] \tag{7-1}$$

where

E_{fp} = the element factor (from 1 to 10) of primary elements

E_{fs} = the element factor of secondary elements

S_f = the extent/severity factor (from 1 to 10)

N_p = the number of primary elements on the bridge

N_s = the number of secondary elements on the bridge

F_1, F_2, and F_3 = a series of factors

The factor varies between 100 (all elements are in pristine condition) to 0 (nonfunctional bridge). However, the target value of BCI is not necessarily 100, because it is accepted that bridges age by using an age factor.

The annual bids for maintenance works are based on the preceding yearly inspections and assessments, namely, the whole life assessments (Das 2000). All programmed maintenance work must be carried out without undue delay. Maintenance strategies rely heavily on preventive work, which makes it is possible to achieve better future estimates for the following parameters: rehabilitation rates (Figure 7-2, above) (Das 2000), total maintenance costs (Figure 7-2, below) (Das 2000), and load capacity (Figure 7-3) (Das 2000).

Based on risk analysis expressed as probability functions for whole life maintenance and traffic delay costs, it is again concluded that preventive maintenance provides the best value for money (Vassie 1999).

Bridge stock condition reports containing performance indicators should be published annually (Das 2000).

The National Audit Office produced specific recommendations for future motorways and trunk roads maintenance programs (Baker 1999): management of the work of maintenance agents (i.e., frequency of inspections, quality control enhancement, bids, funding, monitoring, remedial work discrepancies investigation), management of information (funding allocation enhancement, data validation processes establishment, easy access to management information, and efficient reporting) and measurement of performance (performance indicators, work done, assessment of time spent, and assessment of unplanned interventions).

Table 7-1. Comparison of current and proposed Highways Agency inspection procedures

Current procedures			Proposed procedures		
Inspection type	**Interval**	**Remarks**	**Inspection type**	**Interval**	**Remarks**
Superficial	When needed	Cursory inspection; no standard report	Superficial	When needed	Cursory inspection; simple report format
General	2 years	Visual inspection from ground	General	2 years	Visual inspection from ground; improved report
Principal	6 years	Close visual inspections; all defects recorded	Benchmark	6 years	Close visual inspections; all defects recorded
	6 years	Limited testing of specified areas	Particular	From 6 to 24 years depending on condition	Detailed testing of particular areas
Special	When needed	Detailed testing of particular areas to suit structure	Special	When needed	Detailed testing of particular areas to suit structure
Joint	At construction completion	New structures	Special	At construction completion	No change
Initial Principal	At end of maintenance period	New structures	Benchmark	At end of maintenance period	Close visual inspection
Underwater	6 years	Part of principal inspection	Particular	6 years	No change
Scour	When needed	Special inspection	Special	When needed	Detailed advice to be issued
Paint survey	When needed		Particular or Special	When needed	No change

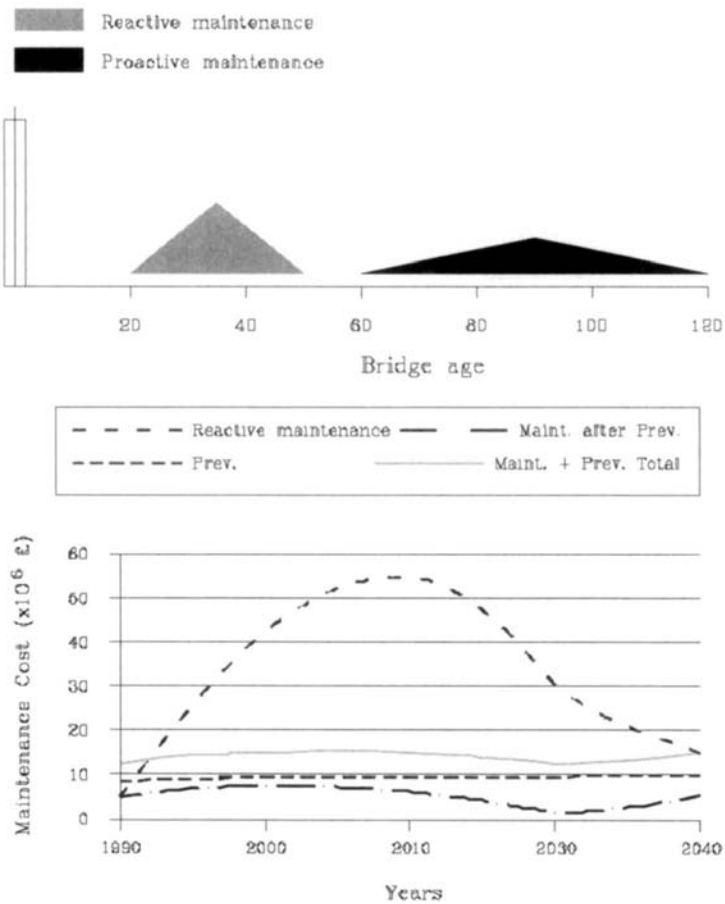


Figure 7-2. Bridge rehabilitation rates with time (above) and predicted total maintenance costs with time (below)

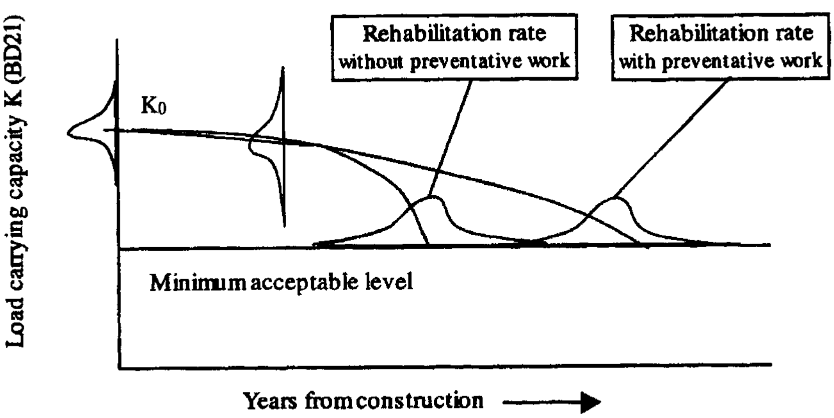


Figure 7-3. Probabilistic deterioration of load capacity with time

7.3. The London Underground Inspection System (U.K.)

Bessant (1993) reports on the then new bridge inspection manual and bridge marking system. Defect classification takes into account the following parameters (scores in parentheses):

- Extent:
  - A—no significant defects (in terms of area, length, etc.) (4);
  - B—slight defects, no more than 5% (of area, length, etc.) affected (3);
  - C—moderate defects, 5% to 20% (of area, length, etc.) affected (2);
  - D—extensive defects, over 20% (of area, length, etc.) affected (1).
- Severity:
  - 1—no significant defect (in terms of severity) (4);
  - 2—minor defects of a non-urgent nature (3);
  - 3—heavy defects of an unacceptable nature which must be included for attention within the next two annual maintenance programs (1);
  - 4—severe defects where action is needed, which must be immediately reported to the supervisor and require action within the next financial year (0).
- Recommended Action:
  - C—replace;
  - P—paint;
  - R—repair;
  - M—monitor;
  - I—inspect;
  - D—design remedial works.
- Priority:
  - I—immediate;
  - H—high (within 12 months);
  - M—medium (within 2 years);
  - L—low (before the next principal inspection);
  - R—review (at the next principal inspection).

The superstructure and the substructure are both divided into a number of elements, each of which receives a score of 1 to 8, obtained by selecting and scoring all the items (defects) within each element and then adding them together (total item score):

$$(\text{total item score}/\text{number of items} \times 8) \times 100 = \text{element rating }\% \qquad (7\text{-}2)$$

To obtain the overall condition rating for the superstructure and the substructure, the lowest corresponding element rating for each is chosen.

7.4. Further U.K. BMS References

7.4.1. Estimation of Bridge Costs

Lee (1990), Bouabaz and Horner (1990), Murray et al. (1990), and Lee (1990) concern the estimation of bridge costs, both for building and for rehabilitation.

Bouabaz and Horner (1990) deal with research into simple models as a means for predicting the new-build cost of bridges, which in turn led to the development of equally simple models for predicting the cost of repair contracts exceeding £10,000 in value.

The models for predicting the new-build cost of bridges are based on the principle of cost significance. It has been known for many years that 80% of the value of a bill of quantities is contained within only 20% of the items; those that are "cost-significant." The cost-significant items are easily identified as those whose value is greater than the mean. The ratio of the value of packages in the model to the total bill value is known as the cost model factor (CMF). Using only 10 items at the feasibility design state, it is possible to predict the cost of a new bridge within 10% (CMF = 0.73). Refinement of the model to include an additional 11 elements allows the cost of a bridge at the detailed design stage to be predicted with an accuracy of 5% (CMF = 0.82). The model, which has been built into a computer package called BRIDGET, allows the price of a new bridge to be calculated in less than 15 minutes. Murray et al. (1990) further refer to BRIDGET.

To model repair costs, the methodology was similar to that used in the new-build analysis. Cost-significant items were identified for each bill, and the results for bills within each category were then inspected for consistency. A variety of techniques were then used to determine the minimum number of cost-significant work packages that represented a constant proportion of the total bill value. The cost model used for repair of reinforced bridges has 14 elements and yields a CFN of 0.82 with a standard deviation of 0.08. Analysis of historical data enabled the proposal of a tentative relationship between the area of the bridge deck and the cost of repairs (Figure 7-4) (Bouabaz and Horner 1990). A relationship between the age of a bridge and the cost of maintenance and repair was not reliably established.

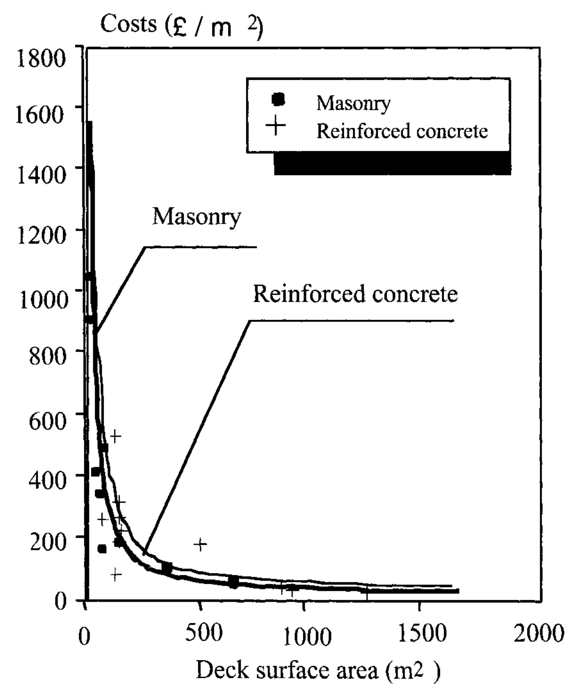


Figure 7-4. Relationship between repair cost and area of deck for masonry and reinforced concrete bridges

7.4.2. Risk-Based Assessment and Prioritization of Bridges

Shetty et al. (1996) present a risk-based framework for assessment and prioritization of bridges in need of remedial work, designed on behalf of the Scottish Office of the U.K. Transport Research Laboratory, which involves (Figure 7-5) (Shetty et al. 1996): (1) screening of bridges; (2) assessment to current standards; (3) risk evaluation of bridges that do not meet assessment criteria; (4) ranking of bridges in terms of risk; (5) design of remedial work for each bridge; and (6) optimal allocation of resources for remedial work on different bridges.

The risk is quantified in terms of the probability and consequences of a bridge failure. Modern methods used for structural reliability analysis evaluate the probability of failure, while the consequences of bridge failure are evaluated in terms of loss of life and injury, environmental damage, loss of assets, repair/rebuilding costs, and road user delay costs. For routine use by practicing engineers, simplified methods for evaluating risk have been proposed.

Multicriteria decision analysis techniques combined with optimal resource allocation algorithms are used for ranking of remedial alternatives for a given bridge and prioritizing remedial work on different bridges. A number of criteria such as cost of remedial action, maintenance costs, traffic delay costs, human risk, environmental risk, traffic risk, and economic risk are considered in choosing the best remedial alternative for each bridge and for

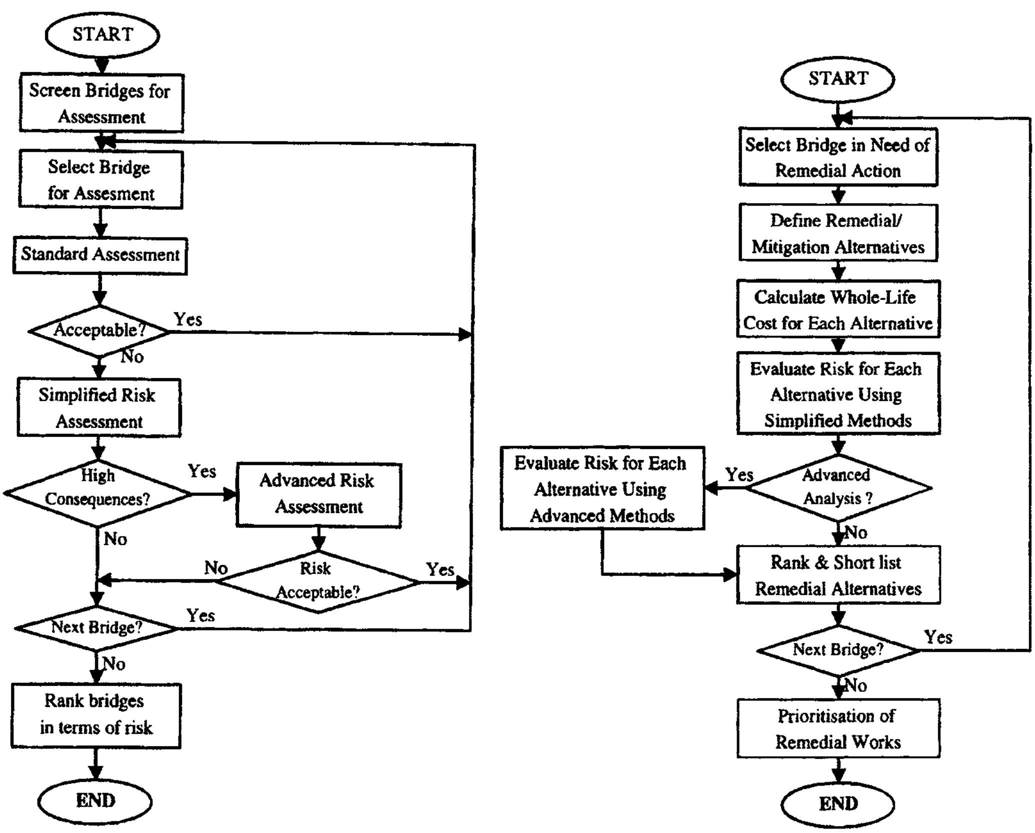


Figure 7-5. Framework for assessment and ranking of bridges in terms of risk (left) and framework for prioritization of bridges for remedial work (right)

selecting the set of bridges to be taken up for remedial action in a given year while still satisfying budgetary constraints.

To implement a workable repair program for the Midland Links Motorways, a risk-based maintenance strategy was developed (Cropper et al. 1999). Five levels of assessment were identified (conservatism in the assessment is gradually reduced as stages proceed from Level 1 to Level 5) (Shetty et al. 1999):

- Level 1: assessment using simple analysis, requirements, and methods;
- Level 2: assessment using more refined analysis;
- Level 3: assessment using better estimates (bridge specific design values of load and resistance, using probabilistic estimates where possible);
- Level 4: assessment using bridge specific target reliability;
- Level 5: assessment utilizing full-scale reliability analysis.

The main stages of the strategy development process are:

- identification of elements;
- screening of structures;
- a qualitative assessment and routine assessment of all bridges; risk is equal to the probability of failure (assessed by considering capacity ratios for the various failure modes, vulnerability to failure due to particular construction details, and condition of the structure) multiplied by the consequences of failure (assessed by considering the importance of the particular section of motorway carried by the structure and the significance of any hazard crossed);
- a quantitative assessment of bridges at greater risk; two approaches have been used: a deterministic one (Level 3) and a probabilistic one (Level 5); several analytical models are necessary to calculate the strength of a deteriorating structural element (Figure 7-6) (Cropper 1999);
- repair of bridges with higher risk within budget limitations.

7.4.3. Other Subjects

Maxwell (1990), Smith (1990), and Loe and Griffin (1980) give general descriptions of what a bridge management system should be. They highlight the most important qualitative factors to take into account.

The Scottish Office Development Department computerized trunk road bridges database (TRBDB), described in Johnstone and Brodie (1999), consists of the following modules:

- inventories of all structures;
- cyclic programming of principal inspections;
- monitoring of principal inspections and structural assessment programs;
- maintenance work prioritization;

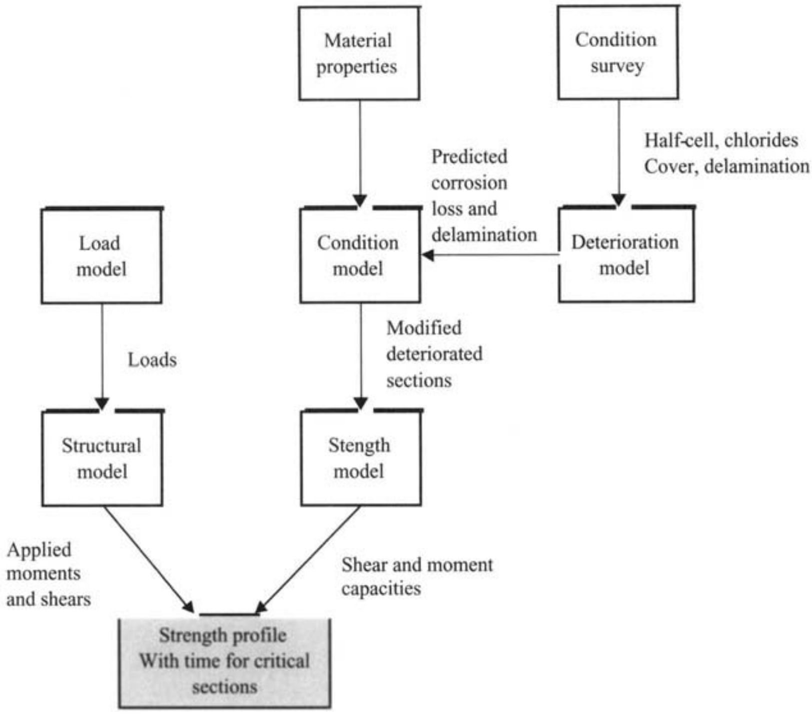


Figure 7-6. Analytical models for the quantitative analysis

- expenditure and works records;
- weather resistant steel bridge monitoring;
- abnormal vehicle movements;
- parapet priority ranking;
- technical approval of structures.

Inspection and maintenance requirements are generally found in SODDa and SODDb. There are four types of inspections: superficial, general (at intervals of no more than 2 years), principal (at intervals of no more than 6 years), and special (as required). Defective main elements are ranked from 1 (insignificant) to 4 (severe). When severe, there is safety risk and repair costs will likely escalate rapidly. Estimates of future maintenance work are made. The engineer responsible for the principal inspection must select one of the following eight actions (Johnstone and Brodie 1999):

- no defective main elements with maintenance prioritization ranking $> 2 \Rightarrow$ no maintenance works required;
- defective main elements having prioritization ranking $> 2 \Rightarrow$ maintenance works should proceed as soon as possible;
- special inspection required next financial year to determine the nature and extent of works required;

- await programmed strengthening or other upgrading and carry out maintenance concurrently with these works;
- where an improvement scheme and detrunking are involved, postpone maintenance work until after opening the new trunk road to minimize traffic disruption;
- postpone maintenance work so that it can be phased in with other future work to be carried out on the route or with land acquisition;
- demolition planned as part of trunk road scheme; structure can safely be neglected;
- beyond economical repair; replace.

In Loe and Griffin (1980), a classification scale of defects was proposed based on: gravity of condition (1—no defects—to 4—immediate attention required) and extension (A—isolated defect—to D—over 20% of the total area). These classifications would be an indication for bridge maintenance.

In Smith and Obaide (1993), the University of Manchester Institute of Science and Technology Bridge Management System, developed to rationalize economic decisions concerning the maintenance of existing bridges, is succinctly described.

Young (1999) presents a local authority perspective on bridge management, and identifies a number of problems leading to significant disruptions to local communities (Figure 7-7) (Wallbank et al. 1999) and a severe maintenance backlog (Figure 7-8) (Wallbank et al. 1999), unless maintenance budgets are strongly increased.

In Jackson (1993), it is argued that conventional analyses used to assess existing bridges are too conservative, especially for bridge slabs, infill decks, and slender piers. Alternative analyses with modified section properties, plastic or nonlinear analyses, and other methods are bound to keep bridges in service for longer periods without jeopardizing their safety.

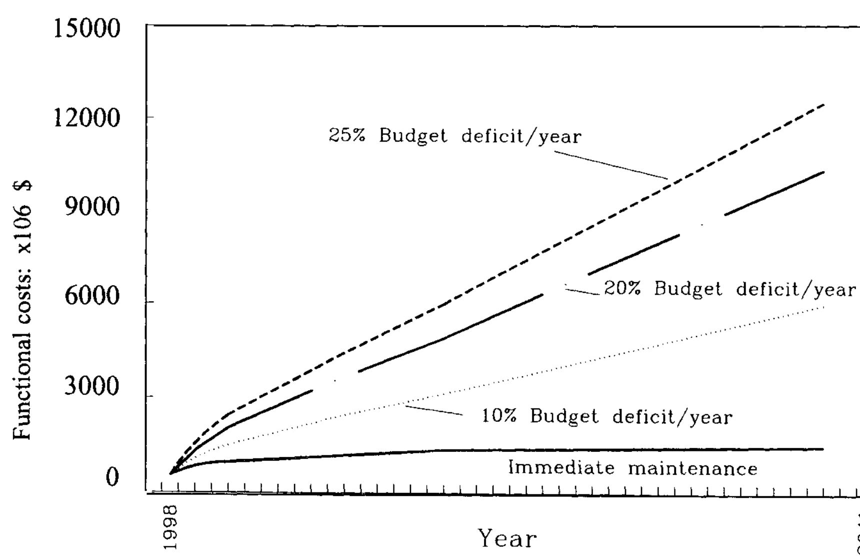


Figure 7-7. Traffic delay costs due to maintenance underfunding

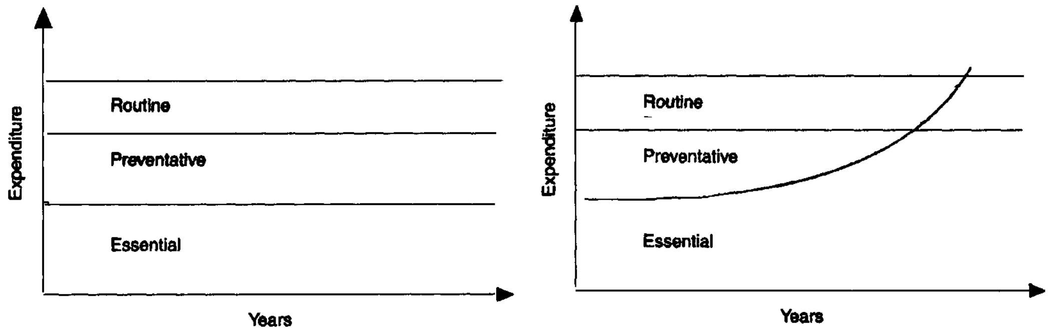


Figure 7-8. Ideal bridge maintenance program (left) and effect of long-term maintenance underfunding (right)

7.5. The French Ministry of Transportation BMS (France)

7.5.1. Introduction

The Roads Department of the French Ministry of Transportation prepared a document named "Technical Instruction for the Surveillance and Maintenance of Engineering Structures" (MTRD 1979). It is divided in many sections: general rules, bridge forms, current surveillance, high surveillance, foundations, bearings, approaches, auxiliary equipment, and so forth. There is also one section for each type of structure (masonry bridges, reinforced concrete bridges, prestressed concrete bridges, steel bridges, suspended bridges, mobile bridges, wood bridges, tunnels, bearing walls, etc.) in which the general rules are particularized by taking into account the features of the structure to be inspected. MTRD is a very thorough document that covers inspection and maintenance requirements.

7.5.2. Surveillance Scheme

The surveillance of the structures is made at two levels: current surveillance (which includes all structures) and high surveillance (which applies only to the structures where important defects have been detected).

Current surveillance is divided into continuous surveillance, annual inspection, and detailed inspection.

7.5.3. Continuous Surveillance

The aim of continuous surveillance is the preservation of all structures, by detecting abnormal behavior in time and to guarantee that all necessary measures are taken when such defects are found. For continuous surveillance, each structure is looked at quite frequently, taking advantage of every possible occasion for paying an informal visit to the site that does not last more than a few minutes. As a matter of fact, most defects that are capable of jeopardizing the level of service are revealed by signs that are easily detectable simply by visual observation: great deflections, drainage obstructions, humid spots, and so on. Road-signs and the auxiliary equipment on the structure and its approaches are also checked. A checklist for each type of structure is provided with many photos of what to look for.

Figure 7-9. French bridge in which traffic is seriously restricted

Within the scope of continuous surveillance, it is possible to find important defects for which an exceptional detailed inspection may be advised. The results of such an inspection will help in making the best decision about how to maintain the structure's safety: traffic restrictions (Figure 7-9) (MTRD 1979) or even closure to all traffic. In very urgent cases, such decisions may be made by the inspector in situ as soon as the defect is detected.

7.5.4. Periodic Surveillance

Annual inspections and detailed inspections are the basis of all periodic surveillance. Most engineering structures related to the road system are included in its scope. They include all viaducts and bridges with a span greater than or equal to 10 m and every structure that continuous surveillance points out as requiring a particularly thorough surveillance (due to the precariousness of its foundations, the risk of a fragile collapse, limitations of traffic weight, etc.). A list of structures within the scope of periodic surveillance is prepared each year indicating those to be subjected to detailed inspections and also include the time for such inspections. This list is based on the results of last year's surveillance and must take into account any bridges newly built or demolished.

7.5.5. Bridge Form

Before the visit, a thorough examination of the bridge form is mandatory. In a special section, the organization of the bridge form is given. Three dossiers are considered: design, construction and history; reference state; and structure's in service life.

The **first dossier** contains all the information concerning the history of the structure up to the creation of the reference state. The dossier must be complete and does not change unless new information from the design/conception is found or an important modification or repair to the structure is undergone. In this case, a new reference state is required and all the data collected concerning the in-service behavior is incorporated along with all documents concerning the up-grade and the results of the proof tests after the work has been done.

The **second dossier** contains all the information necessary to define the current reference state and does not change unless a new reference state has to be created, in which case all modifications must be filed. The reference state is created whenever a new structure is built or when an existing structure is subject to significant repair or upgrading. Whenever possible, the inspection must follow the construction end and precede the opening to traffic. It is also possible that an existing structure has never been subjected to periodic surveillance or has gone too long without it and may need to have a new reference state. In this case, the inspection team must be acquainted with the construction techniques and materials used when the bridge was built.

The **third dossier** stores all the information concerning the bridge after the reference state and must be constantly updated. Records of all surveillance and maintenance programs are kept along with written reports of each inspection made and any events that may influence the structure. Graphic information (photos, schemes) is also welcome.

From the bridge form, the inspector must be aware of all the conclusions made during past inspections as well as the interventions made (in the bridge and its approaches) and the particular aspects to look for. Simple drawings of all outer surfaces of the structure are prepared in advance so that the defects detected can be symbolically represented. A reference grid conceived after the first detailed inspection must be superimposed on the drawings. The drawings from the last inspection with all the data collected must be brought along in order to compare and plot the defects that have evolved.

7.5.6. Inspection Manuals

The sections of MTRD prepared for each structural type are a highly useful tool. A chapter dedicated to the causes and nature of the most common defects, another on the particular items to look for in an inspection, another on maintenance, and another on repair are included. A number of appendixes provide indications of documentation that is needed before the inspection showing how to produce the reference grid; show how to write the reports for each type of inspection; and even give some basic knowledge of current construction and repair methods. Some of this information is repeated in different sections, which makes the documents rather bulky but also self-sufficient.

The defects detected are classified according to the normative document "Apparent Defects of Concrete Bridges" (MEb) prepared by the Central Laboratory of Bridges and Roads (LCPC). The following classification is proposed:

B—defects existing from the construction stage onwards and with no other consequences than aesthetics;

C—defects whose evolution may be out of the ordinary;

D—defects that display a clear degradation evolution;
  DA—beginning of the evolution;
  DB—advanced evolution;

E—defects that clearly indicate a change in the behavior of the structure and which jeopardize its target service life;

F—defects that indicate the proximity of a limit state and imply a limitation to the use of the bridge, which may even include closure.

The document also presents a group of defect forms in which the several possible levels of seriousness for each one are described and illustrated with photos or simplified drawings. The probable causes of the defects are also proposed.

7.5.7. Annual Inspection

An annual inspection must be planned in advance to make the best of certain circumstances (traffic, weather conditions) that may help in the detection of defects. The necessary means of access for each bridge and the proximity of other bridges also must be taken into account in the planning. Should it be found useful, the annual inspection can be divided in several partial inspections.

Only light and simple equipment should be necessary: pencils, pens, chalk, thermometers, rulers, clinometers, a plumb bob, a hammer, a chisel, a flashing light, a camera, video equipment, binoculars, and so forth.

At the site, the weather conditions and temperature must be noted, because some defects (e.g., cracks, deformations, etc.) depend on them and may even influence the timing of the inspection. Abnormally heavy traffic must also be noted. The aim of this inspection is the examination of the structure's environment, the checking of a number of points listed in the section for the particular type of structure, and the readings of control equipment already installed. Even though the aim of this inspection is to observe and not to analyze, it is always useful to try to understand and compare the observations made to avoid having to go back to the site a short time later.

At the end of the annual inspection, a standardized written report must be prepared. Photos must be carefully referenced with a clear scale; they are very useful to complement the report. At inspection headquarters, more information may be added: a classification of the general state of the structure; proposals for maintenance or repair work to be done; and a proposal for a new inspection, either ordinary or exceptionally detailed.

7.5.8. Detailed Inspections

Detailed inspections occur, in principle, every 5 years but that period may be reduced to 1 year, particularly in certain areas of the structure. The inspections are thoroughly planned in advance and are headed by a specialist both in inspections and in the particular type of structure. Their aim is to quantify and analyze all the defects detected in order to estimate their evolution in time: a complete check-up of the structure.

Three types of detailed inspections are considered: the first detailed inspection, the periodic detailed inspection, and the exceptional detailed inspection.

7.5.8.1. First Detailed Inspection

The aim of the first detailed inspection is to define the reference state of the structure to which all future inspections will refer. This inspection is to be performed on every bridge, even in those that are too small or unimportant to be subject to periodic surveillance.

The first inspection must be more thorough than periodic inspections and must try to achieve the objective of detecting every defect resulting from errors at the design/construction stage (geometry defects, imperfections due to poor workmanship, disorders resulting from proof testing, etc.).

In addition to a written report of the inspection, reports on the proof tests and on the leveling of the bridge are needed. Also added are drawings of all outer surfaces that are easy to handle with an adequate scale to allow writing information on the defects detected during future visits. All drawings must have a reference grid for future use.

7.5.8.2. Periodic Detailed Inspection

Preparation for the periodic detailed inspection includes a careful study of the bridge form in order to determine the causes and evolution of the defects detected during the previous inspections and specific points to look into in detail. Special attention must be given to the report of the first detailed inspection and, after the last periodic detailed inspection, the annual inspection reports and the ones concerning the main maintenance and repair works. A pre-visit to the site may be useful to make sure the necessary means of access are requisitioned. In the scope of the detailed inspection, every part of the structure must be accessible and examined close-up. It will sometimes be necessary to use special means of access that are not supposed to be used in the annual inspection: special inspection vehicle, scaffolding, scuba diving, etc. (Figure 7-10) (Bois 1978).

In addition to the equipment required for an annual inspection, it would be helpful to take along additional equipment so that in situ measurements can be made (e.g., displacement transducers, strain gauges, etc.). In most cases, there is no need for overspecialized equipment because no specific problem is expected to be found. Drawings of the exposed surfaces with a reference grid and details of all points to be inspected are to be taken to the site as well as the fascicle for the particular type of structure to be inspected.

The aim of this inspection is to thoroughly examine the surrounding area and the structure itself. A photographic report of the main defects should always be made, referring them to well-known points of reference and making sure that a scale is included in each photo. An inventory of the site and functional conditions of the drainage system is to be prepared. All measuring equipment at the structure at the day of the inspection must be subject to an inventory in which its site and nature are recorded.

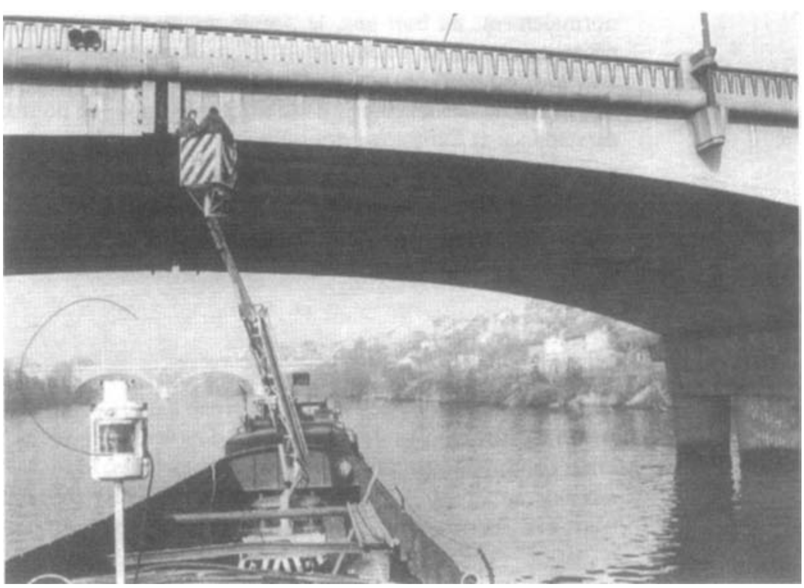


Figure 7-10. Use of a barge with a bascule cage for inspection of a bridge intrados

Another report must be prepared to include the defect classifications according to their nature, probable cause, and evolution; the degree of rusting on steel surfaces and the importance of the corrosion areas; the results of the analysis of the service conditions and of exterior factors susceptible to cause malfunctioning; the results measured during the visit to the fixed equipment installed at the bridge (e.g., crack measurements, piezometers, clinometers, etc.); an analysis of all current investigations and the results obtained to date; and a justification for the installation of new equipment.

Yet another report must be written consisting of a short description of the structure, its functioning, its main dimensional characteristics, and its service conditions. An inventory of every part of the structure (showing defects or not), its description, and evolution with reference to photos or drawings is also made.

A final report is prepared at the inspection headquarters classifying the general state of the structure in one of four categories, proposing maintenance and repair work and the complementary action of surveillance or investigation deemed necessary to better define the state of the structure (calculations, chemical analysis, long-term measurements, etc.). The report must also refer to whether the inspection team found it necessary to update the reference state or the bridge form.

7.5.8.3. Exceptional Detailed Inspection

The exceptional detailed inspection comes as the result of the detection of a potentially serious defect needing further looking into to clarify its nature and probable evolution. It differs from the periodic detailed inspection because quite often only part of the structure is inspected and only certain aspects are carefully investigated. The procedure is similar, but the equipment used may be more specialized and expensive.

7.5.9. High Surveillance

High surveillance is an exceptional measure aimed at monitoring the appearance or following the evolution of a structural state considered to be dangerous and allowing time enough to take the actions necessary to maintain a satisfactory level of safety (Bois 1978). The first step in the risk analysis is a very thorough inspection of the structure followed by a preliminary analysis of the causes of the disturbance. It is then possible to identify the mechanism that promoted the existing situation. It is also necessary to analyze the functioning of the deteriorated structure in order to define the possible mechanisms of evolution. Of these, the ones in which there is a possibility of effective countermeasures must be identified. To be complete and allow the definition of the means needed at the site, the analysis must also include the identification of the measurable parameters that may quantify the degree of evolution of the deterioration. It is also necessary to define for those parameters the levels over which the probability of collapse becomes excessive. According to this risk analysis, the bridge is classified into one of three levels of high surveillance. The higher the level of risk, the faster and more automated must be the actions to protect the user's safety.

To decrease the degradation risk and the probability of failure, a frequent measure under these circumstances is to restrict traffic (especially heavy traffic). The possibility of closing the bridge is so inconvenient, especially in the main roads, that sometimes it is preferable to install equipment (Figure 7-11) (Bois 1978) to measure the parameters that are supposed to function as forewarnings of a structural collapse. The inspector must be absolutely certain that, if the alarm level is reached, there is enough time to take all necessary safety precautions. The means used can be daily visual observations; equipment of automatic measurement

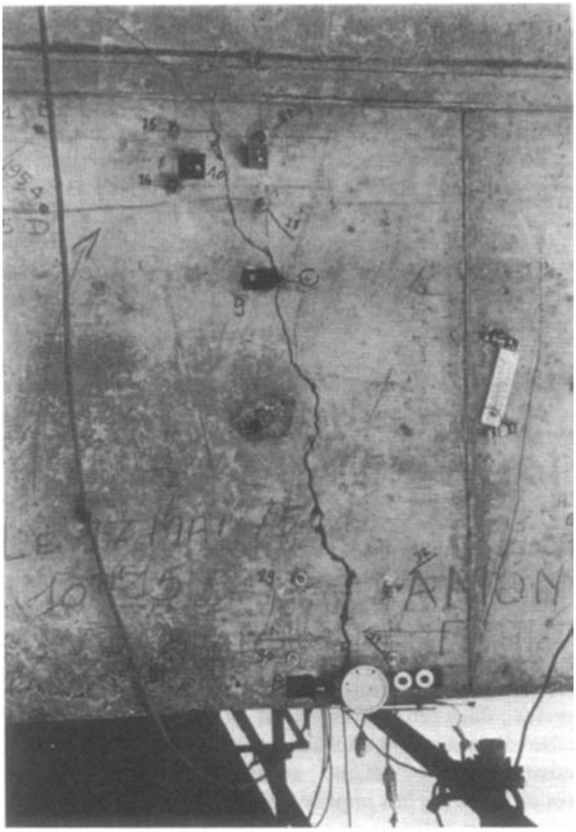


Figure 7-11. High surveillance of the evolution of a structural crack

(continuous or noncontinuous) and register connected to inspection headquarters; totally automatic devices that can stop traffic without human intervention (using traffic lights and barriers) when the measuring equipment detects an unacceptable level.

7.5.10. Post-Surveillance Measures

Depending on the results of the inspections, several actions can be taken:

- to promote an exceptional detailed inspection to analyze the real service level and safety of the structure or define the complementary investigative actions necessary to achieve these goals;
- without waiting for the results of the inspection, take the necessary measures for the safekeeping of public safety (traffic restrictions or complete closure);
- to investigate the causes of the defects detected;
- to study and evaluate the several possible rehabilitation measures available to eliminate the defects;
- to classify, by order of priority, the actions concerning the several bridges;
- to give the bridge high surveillance status.

7.5.11. Maintenance and Repair

Rehabilitation measures are divided into maintenance and repair. While maintenance is fundamentally preventive in nature, repair implies that the level of service has decayed and must be restored. Maintenance is divided into current and specialized. Current maintenance requires only small amounts of technical know-how and must be performed regularly in close association with continuous surveillance. It includes cleaning debris in the drainage system, the joints, the deck, the bearings, and the sidewalks; removing vegetation growth and debris accumulated in the water by the columns; and so forth. Specialized maintenance requires special techniques and includes repositioning and replacement of bearings; replacement of joints; repainting steel surfaces; works needing scaffolding or special inspection vehicles; underwater maintenance; and the like. All repair work must be preceded by a detailed inspection with a thorough study of the problem and an economic analysis of the several possible repair methods.

7.6. The "Bridge Maintenance: Surveillance Methodology" PhD Thesis (Switzerland)

7.6.1. Introduction

At the École Polytechnique Fédérale de Lausanne, a doctoral thesis titled "Bridge Maintenance: Surveillance Methodology" (Andrey 1987) was prepared. Even though its conclusions and recommendations seem not to have been officially adopted throughout Switzerland, it remains one of the most complete documents on bridge expert systems.

Although the document is very thorough in its description of inspection and maintenance modules, its approach to the decision system is not very satisfactory. Only the maintenance subsystem is approached (and the criteria mentioned are a bit vague and qualitative) and no reference is made to the rehabilitation/replacement subsystem. It must be stressed again that this document is a research document and was not devised for use in the field without some adaptation. No mention is made of the use of computer databases or management systems.

7.6.2. Architecture of the System

7.6.2.1. Introduction

The document consists of seven chapters and four appendices in which a methodology for bridge surveillance (inspection) is proposed. Bridge management is divided into three largely interactive domains: normal use, maintenance, and repair (Figure 7-12) (Andrey 1987).

Fundamentally, normal use has to do with making sure that the bridge fulfils its function, based on the assumption that maintenance and repair have been performed. Maintenance concerns the elimination of current small-scale problems in the service capacity of the bridge and its durability that arise when trying to guarantee user safety It includes two fields of action: surveillance and ordinary maintenance work. Repair, in the context of this document, includes both rehabilitation and strengthening and is classified as extraordinary maintenance work.

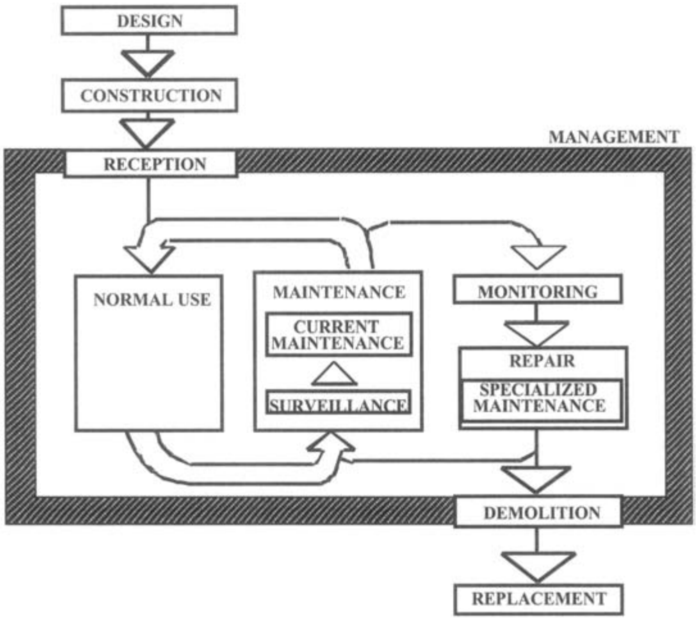


Figure 7-12. Representation of bridge management in Switzerland

7.6.2.2. Defects

A description and classification of potential defects present in concrete bridges follows. Material defects (concrete and steel) as well as structural defects (structure as a whole or parts of it) are described in some detail and illustrated with photos and graphics.

7.6.2.3. Diagnostic Methods

An inventory of the diagnostic methods that may be used for concrete bridges is produced. A short description of each of the listed methods is included (illustrated with photos and drawings). Based on predefined criteria, a rating is made for each diagnosis, much as the authors proposed. The most promising methods were selected and the others were set aside to be used only when indispensable.

A detailed description of the selected diagnostic methods is also presented. A description of the procedural methods is given, along with advice on where to perform the tests and how to report the results. Some innovations were presented: a defect form (later adapted by the authors) and the concept of a reference grid (to facilitate the location of the defects both on site and at headquarters). Whenever possible, the defects likely to be detected using the selected methods are classified according to their gravity, extension, or other convenient criteria. This classification is the basis of the decision system referred to in Chapter 8. Special attention is paid to the evolution of the measurements from fixed equipment, concrete cracking, steel corrosion, pavement watertightness, and middle span deformations.

7.6.2.4. Appendices

Terminology is defined in order to clarify certain notions and designations. An inventory of all diagnostic methods for concrete bridges is presented and a classification is proposed. An inventory of all surface defects in concrete bridges is presented with a tentative classification (similar to the one presented in Chapter 10). Finally, a long list of references is presented.

7.6.3. Inspection Strategy

In the proposed system, inspections are divided in periodic, routine, and special inspections.

Periodic inspections are the basis of the surveillance scheme and should provide a clear picture of the state and behavior of the bridge. The first inspection should take place 4 years after the bridge has been opened to traffic and subsequent inspections follow at 5-year intervals. The aim of the periodic inspections is to put into perspective the surface defects, structural cracking, materials deterioration, deformation, and displacements in the structure and the state of the equipment at the bridge. For execution of the inspection, it is necessary that personnel have the right degree of specialization, means of access are adapted to the bridge being inspected (it is mandatory that every part of the bridge be watched closely), and the bridge is put out of service, either totally or partially, for the extent of the visit or just temporarily. Based on the results collected, the inspection can have one of the following consequences: organize a special inspection or take complementary measurements; give the bridge a detailed (continuous) surveillance status; establish a list of the particular points for close control in the next inspections; organize the necessary maintenance work; establish a plan of maintenance work to be performed in the meantime.

To plan systematic surveillance of a structure, certain documents, including general plans and drawings as-built, a list of the particular points to look at, a surveillance timetable for each bridge, reference grids of the most relevant elements, and an inventory of all surface defects with a numerical code, are needed.

At the end of a periodic inspection, the following documents must be prepared: a graphic representation (using a reference grid) of the nature and location of all surface defects and cracking; a log containing more specific data on the defects and their evolution; a log containing measurements of sclerometer, carbonation and chloride content; the cartography of steel corrosion in all surfaces exposed; significant changes in crack width and middle-span deformations; an opinion on the gravity of the surface defects and cracking detected; an evaluation of the evolution of carbonation, chloride content, and corrosion; an evaluation on the tendency of the deformations; a list of the particular points to control closely in the next inspections; and a list of points that need continuous observation.

Routine inspections are performed between periodic inspections: three routine inspections with the time intervals of 15 months between every two periodic inspections. Routine inspections, being much simpler than the periodic inspections, must clearly show the existence of defects with a fast evolution and follow the progress of defects detected previously. They are limited to the visual observation of the bridge outer surface and the detection of surface defects. For its execution, it is necessary that nonspecialized personnel with some previous training and limited means of access perform the tasks. Traffic control may or may not be necessary.

Special inspections are undertaken only on request: to further confirm the detection of a certain type of defect where doubts still arise; to control the global behavior of the structure after some extraordinary event (heavy vehicle crossing the bridge, flood, earthquake, traffic accident, etc.); to establish a new reference state after important rehabilitation work has been performed on the bridge. For these reasons, special inspections cannot be planned far in advance.

When the contractor turns the bridge over to the owner at the end of construction and before it is opened to traffic, a special inspection is performed to establish the reference state. Its meaning is not elaborated upon in this document, but it should follow the general guidelines of the French inspection manual (MTRD 1979).

7.6.4. Decision Criteria

The decision criteria proposed in this document are not consistent with the use of computers but are based mostly on the inspector's experience and common sense. It is also limited to maintenance work, omitting the rehabilitation/replacement subsystem. Finally, no mention of budgeting is made because of the probability (nearly always inevitability) of limited funds.

Decision criteria are proposed for surface defects, concrete cracking, steel corrosion, water tightness defects, and deformations based on previous classifications usually made at the site.

Surface defects are classified according to degree of severity from 1 (defect purely of an aesthetic nature or the result of faulty finishing, with no direct consequence) to 5 (defect potentially liable to lead to the collapse of main structural elements or to traffic danger). A complementary rating factor measures the extent of the defect and goes from *a* (minimal extent when compared to the dimensions of the element affected and taking into account the defect's nature; first signs) to *c* (important extent when compared to the dimensions of the element affected and taking into account the defect's nature; advanced state of evolution). Based on this classification, Figure 7-13 (Andrey 1987) gives an indication of which action is to be taken next.

Concrete cracking is classified in accordance with three different severity-rating factors. Factor A defines the nature of the cracking and ranges from 1 (stabilized crack due to execution or to imposed deformations) to 5 (nonstabilized crack due to loading). Factor B defines the amount of cracking and ranges from 1 (single crack, small width, discontinuous outline) to 5 (single crack, large width, continuous outline). Factor C defines the extreme

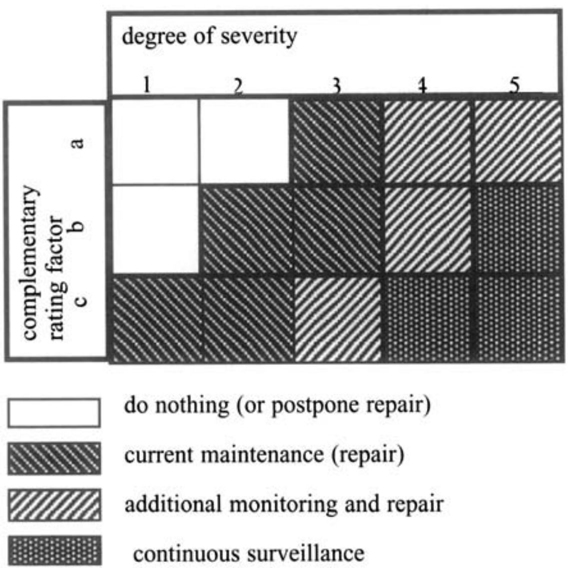


Figure 7-13. Decision criteria for surface defects

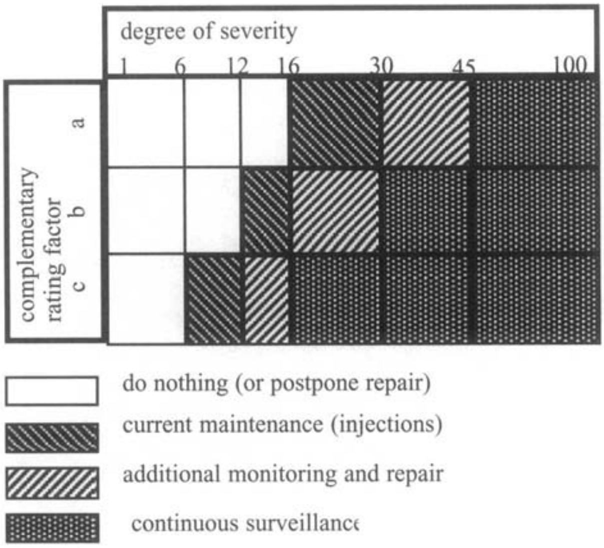


Figure 7-14. Decision criteria for concrete cracking

consequences of cracking and ranges from 1 (aesthetic, surface deterioration) to 4 (structural system modification). These three factors are multiplied to obtain a pseudo-quantitative degree of severity ranging from 1 to 100. A complementary rating factor measures the structural importance of the cracked element and ranges from *a* (secondary element not carrying loads) to *c* (main element from the load carrying system). Based on this classification, Figure 7-14 (Andrey 1987) gives an indication of which action is to be taken next.

Steel corrosion is detected by measurement of the potential field using an electrode. Numerical criteria (dependent on the type of electrode used) are given to choose the measurements that indicate an almost certainty of active corrosion and those that indicate the almost certain nonexistence of such corrosion. Based on these data and the respective percentage of the area measured, Figure 7-15 (Andrey 1987) gives an indication on which action is to be taken next.

For water tightness defects, the decision criteria are based on whether the defects are localized, generalized, or of uncertain extent. For each situation, different rehabilitation measures are proposed.

The decision concerning deformation concerns whether a special inspection, to find the origins of the defects and to make all the necessary measurements, should be performed. The parameters that condition the decision are the displacement of the bearing due to thermal effects (if its nonreversibility exceeds 10% of the maximum value of the displacement) and the deck middle-span deformation (if its increase drifts more than 20% from what is to be expected from previous measurements or if the ratio deformation/span exceeds 1/1500).

7.6.5. Maintenance and Repair

Maintenance work is divided into current maintenance and specialized maintenance. Current maintenance involves only simple equipment and personnel without special preparation and concerns the work that is likely to appear in every bridge of the network as a result of normal

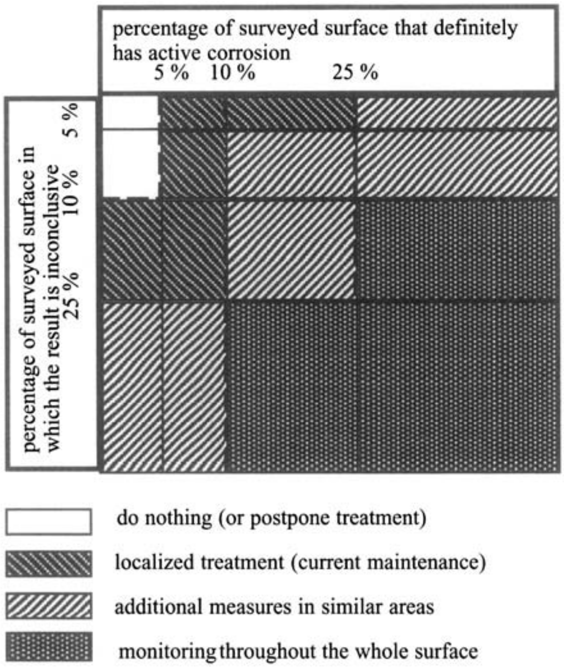


Figure 7-15. Decision criteria for steel corrosion

use. Specialized maintenance involves nonstandard equipment and personnel familiar with specialized techniques and concerns specific less common work on certain bridges.

Ordinary maintenance work can be either periodic (and therefore likely to be planned) or random as a consequence of the surveillance. On the contrary, extraordinary maintenance work is by definition nonplanned because it always follows the detection of unexpected structural problems.

7.6.6. Other Swiss References

A firm of consultants has been preparing a computerized database to help manage the maintenance of structures (bridges, galleries, tunnels, culverts, and retaining and protective structures) on Swiss highways. Grob (1989) highlights some of the main general particularities of the system without presenting any major innovations. Every computer database is in some way different from all others and this one is no exception. As interesting points worth mentioning, there are the implementation of the following aids intended to provide additional user-friendliness: automation of the data saving, access and function selection; extendable catalogues with expert knowledge; standard access paths and corresponding lists (repairs, defects) for quick access; automatic output of structure and structural element specific checklists; and structural component generators.

SSEA (1975) regulates periodic inspections of bridges in Switzerland. It presents no innovations worth mentioning when compared with (Andrey 1987) or (MTRD 1979). It gives the standard sequence of inspection procedures, a checklist of documents and equipment to take to the site, some inspection forms, a checklist of particular points to look into, and guidelines on the methods of investigation according to the type of structure and each particular element.

7.7. The DANBRO BMS (Denmark)

7.7.1. *Introduction*

As an initiative of the Railways of Denmark, a bridge management and maintenance system named DANBRO was developed (DANBRO). This system, in use in Denmark (2,500 bridges) and in Thailand (10,000 bridges), is probably, of the systems analyzed in this chapter, the one that is closer to the concept of a knowledge-based expert system. Sørensen and Berthelsen (1990), Lauridsen and Lassen (1999), and Sørensen and Davidsen summarize the main characteristics of the system.

The objective of the system is to provide bridge authorities with a tool that helps in:

- guaranteeing the safety and functionality of the network;
- collecting objective data about the bridges;
- optimizing the use of the funds assigned;
- guaranteeing a technical-economic backup.

The system consists of the following parts (Figure 7-16) (Sørensen and Berthelsen 1990): a database, three modules (the module of inventory, the module of inspection and bearing capacity, and the module of ranking and budgeting), and user's manuals for all activities.

7.7.2. *The Database and the Inventory Module*

The database is divided as follows:

- system registers containing the main data catalogs used by the system;
- basic registers that contain general information about each bridge;

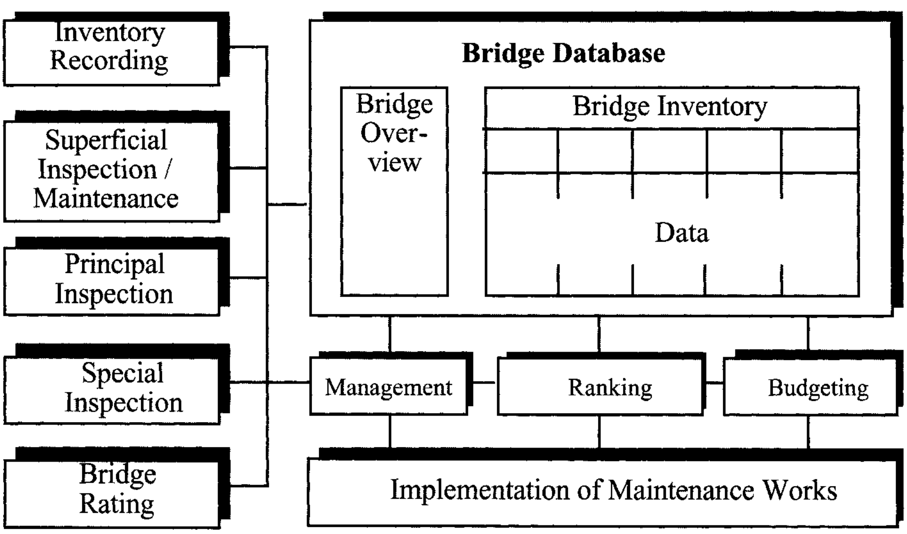


Figure 7-16. Main activities covered by the DANBRO system

- bridge information registers, which contain administrative data, the structure geometry, and its condition evolution;
- registers about the maintenance alternatives, which contain one or more repair strategies for each bridge;
- budget registers containing costs estimations for the several maintenance alternatives;
- intermediate registers and indexes.

Information about each bridge is categorized and associated with each specific element. The structural elements are ranked hierarchically. The system can (and should as much as possible) be operated with only a small amount of general information on each bridge, since it is intended to be used in personal computers of limited capacity. Later, detailed information about administration, geometry and structural model, materials, deterioration, residual service life, and maintenance strategies may be recorded.

The inventory module allows the user to create general reports with predefined information about the bridges that are being inspected. It also allows the user to create individual reports comprising selected information in accordance with his necessities. A very interesting application, is the possibility of the user to choose in a map the road stretch or crossroads in which the bridge under analysis is included (using the "mouse"). The screen provides a graphic representation of the requested information. The bridge can then be selected, which causes a schematic drawing of the bridge to appear on the screen. The whole process proceeds as if the user were using a lens with which he progressively zooms "down" until what is of interest to him is shown in great detail (Figure 7-17) (Sørensen and Berthelsen 1990). This permits quick and logical access to all information about the bridge.

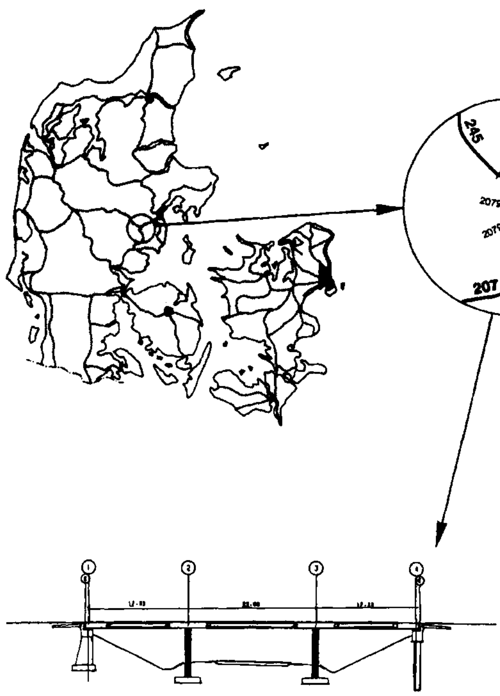


Figure 7-17. Using a "zoom" for access to the graphic information about a specified bridge

7.7.3. The Inspection Module

Three types of inspections, described and standardized in their respective manuals, are considered: superficial inspection, principal inspection, and special inspection (Sørensen and Berthelsen 1990).

Local personnel carry out superficial inspections at short intervals. The structural elements are cleaned and minor damages are repaired in accordance with the instructions in the manual. The maintenance level for each bridge may depend on its importance.

The principal inspections are performed by well-trained local engineers with average intervals of 3 years, with a range of 1 to 6 years, depending on the general condition of the bridge. The inspector evaluates the damage and gives a condition mark to selected elements in order to describe the bridge's general condition. Significant damage is registered for future documentation. The residual service life and the repair costs corresponding to each element are estimated. A date is proposed for the next principal inspection. Finally, it must be decided whether a special inspection is considered necessary.

The special inspection is a detailed investigation of the bridge or part of it and is performed when the estimated bearing capacity level is about to reach predetermined limits. The initiative to perform the inspection comes from the inspector's recommendation during a principal inspection or from the system itself based on the ranking list. Laboratory tests normally are needed, and the results of this inspection will generally lead to alternative rehabilitation strategies along with their respective costs and residual service life estimates.

7.7.4. The Bearing Capacity Module

Associated with the inspection module, this system presents a very innovative load bearing estimation routine (Sørensen and Berthelsen 1990) that, in fact, is already a part of the decision system. The bridges are rated based on inventory module and on the inspectors' reports about the structure's present condition. It is also possible to rate actual nonstandardized vehicles.

The system itself estimates the bridge bearing capacity and compares the result with loading due to the codes for standard vehicles. The bearing capacity class of the bridge is given as a percentage of the code loads. The materials deterioration, defined by the condition mark at the inspection, usually originates a decrease of the resistance capacity and, consequently, of the global bearing capacity. The system allows this deterioration to be taken into account by using pseudo-quantitative deterioration models.

For an actual predetermined vehicle, the system determines the respective action-effects in a representative set of bridge spans and compares the results with the action-effects corresponding to code standard vehicles. The maximum ratio value thus obtained defines the actual vehicle's class. In this way, the system facilitates the administration of heavy transports, since all that is needed is a comparison between the vehicle class and the bridge class to know whether the vehicle can pass over the bridge.

7.7.5. The Ranking and Budgeting Module

The class attributed to the condition of each element and to the general condition of the bridge during the inspection is the basis for bridge ranking in terms of repair. The final ranking depends on the following: present general condition, present bearing capacity when compared with the target value, importance of each element that is in need of repair for the global function of the bridge, and importance of the route that passes over the bridge.

The replacement costs for the several structural elements estimated during the principal inspection are the basis of an initial approximation of a long-term budget. To prepare a precise short-term budget, it is necessary to prepare alternative maintenance strategies, by implementing special inspections for the bridges at the top of the ranking list and for all those for which it is necessary to perform immediate maintenance work. The selection of the alternatives is based on current prices analysis, including possible considerations about traffic stoppage or detour costs during the execution of the work.

The estimates obtained from the maintenance strategies or directly from the principal inspections are resumed for all bridges and compared with available funds. When necessities are higher than available resources, the user must point out the specific changes that need to be made during investments planning so that the situation is balanced.

7.8. The BRISA—Portugal Highways BMS (Portugal)

BRISA, Portugal Highways, S. A., is the concession-holder responsible for most of the Portuguese highways and is in charge of designing, building, maintaining, and repairing the national motorway network and the bridges within it. Its profit comes from toll tariffs generated by these roads and bridges. In Santiago (2000), the bridge management system presently implemented is described in detail. The computer database is organized as follows:

- bridge reference module (stable information including the identification form);
- system reference module (inspection, decision and repair work selection information catalogues, including several tables);
- inspection and maintenance/rehabilitation backing module (variable information including the inspection form, an historical note and non-destructive testing);
- economic module to support the decision and management system (including technical-economic assessments);
- bridge seismic assessment module (including the seismic form).

The database input is the information contained in the identification form (i.e., main geometric and structural characteristics, location, encoding, and building plan identification); inspection form (standardized description of the defects detected, their locations, extent and severity, number of individual elements affected, and percentage of their area, length, etc. affected), and measurements form (geometric characteristics of the several structural elements).

The code number (Figure 7-18) (Santiago 2000) attributed to each bridge contains the following information: identification of the highway, identification of the assistance and maintenance center of the bridge, sequential numbering of the bridge, identification of the junction, and identification of the bridge type.

The inspection subsystem is defined as a set of procedures that allows the detection of defects in the various bridge components and the determination of their evolution with time, as well as their classification in terms of extent, severity, progression potential, and influence on the bridge condition and load-bearing capacity (Santos 1999).

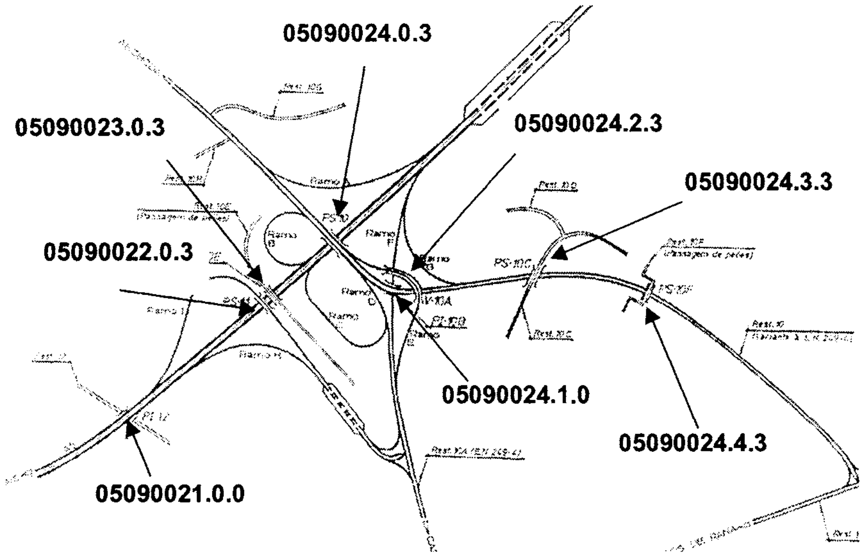


Figure 7-18. Bridge code numbers in a junction

When planning inspections, it is important to have an estimate of the time needed for each bridge. To complement the inspectors' experience, the concept of "equivalent element" has been devised with the following values:

- overpasses and underpasses: 1 equivalent element per span;
- viaducts: 1 equivalent element per span within the same lane;
- tunnels: 1 equivalent element per 100 m.

The inspections are classified into three different types (Santos 1999):

- first level or ordinary inspections—every 3 years (6 years for noncurrent bridges), based on visual observation and simple nondestructive testing; portable inspection equipment (Figure 7-19) (Santiago 2000); made on foot;
- second level or principal inspections—every three years (6 years for noncurrent bridges), based on close visual observation and nondestructive testing; special means of access may be used (Figure 7-20) (Santiago 2000);
- extraordinary inspections—when needed (the bridge's structural safety is at risk because of a defect detected in any of the periodic inspection or after exceptional happenings such as accidents, a seismic event, a flood, a fire, etc.), based on very close visual observation and nondestructive testing of certain elements.

To standardize the defects identification and location, a universal location system has been implemented, based on the identification of each structural element and piece of equipment in the bridge (Figure 7-21) (Santiago 2000) and on a lexicon of all the potential defects.

Figure 7-19. Ordinary inspection equipment

Figure 7-20. Principal inspection vehicle

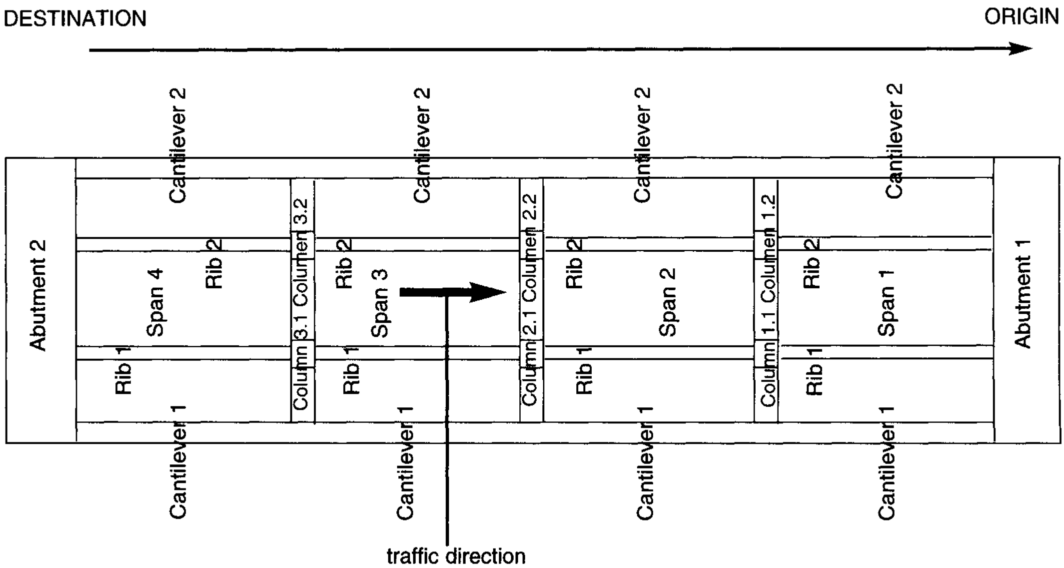


Figure 7-21. Location scheme for viaducts

To standardize the description of the defects, a defect rating (from 1 to 7) from a structural point of view has been devised:

- 1—the defect degree does not increase or lead to another defect;
- 2—preliminary stage; it may lead to another defect but does not require intervention;
- 3—it is evolving but does not require intervention;
- 4—it requires intervention but not urgently;
- 5—it does not influence the bridge structural behavior;
- 6—it does not impair the bridge structural safety;
- 7 —it leads to a reduction of the safety coefficients.

Additionally, a catalogue of defect forms has been developed containing a code number, a designation, the defect class (type and material affected), possible structural elements in which it may occur, a description of the defect, direct and indirect possible causes, correlation with other defects, extent classification, and defect rating classes.

The decision subsystem provides the bridge authorities with recommendations about the best option at relevant moments of the bridge's life. A computer program named STONE performs the following operations:

- defines and calculates each bridge structural degradation index (from 1 to 7), based on the global degradation index IDT (the sum of all the defects detected in the bridge multiplied by the weight attributed to the structural element in which each defect occurs, multiplied by the attributed defect rating, by the number of spans affected and by the average percentage of area affected in each span) and by the standardized degradation index IDN (the quotient of the IDT factor and the number of spans);

- estimates the intervention cost, based on the repair work unit cost, the defect extent and rating, the number of individual elements affected, the percentage of their area, length, etc. affected, and the measurements form;
- predicts the most probable evolution of bridge condition with time, based on multiple parameters and a Markov theory–based model;
- estimates the maintenance investments budget necessary; the bridges that require intervention are listed, together with the proposed work and the time needed for design and rehabilitation.

7.9. The Ministry of Transports and Public Works Database (Netherlands)

7.9.1. *Introduction*

The Public Works Department of the Ministry of Transports and Public Works promoted implementation of the DISK (El Marasy) system, which permits access to structures. This system is only an extremely powerful database. The inspection module is exterior to the database and the decision system is based, at all levels, on the options defined exclusively by the user taking into account the stored data.

Van Der Toorn and Reij (1990) also describe several interesting ideas concerning deterioration models.

7.9.2. *General Organization of the Database*

The information stored in the database consists of (El Marasy):

- basic information of an administrative and technical nature that is almost never altered but that can be updated;
- variable information related to bridge management activities, such as inspections and maintenance;
- financial information about maintenance.

Therefore, the database consists of the following modules: inventory and administrative information, inspection, maintenance, bridge history, and special vehicles.

7.9.3. *Static Information*

This information is divided into (El Marasy):

- identification (topographic map and bridge designation);
- location (identification of the route or water course; coordinates; council and province);

- administrative information (departments responsible for the design, inspection, maintenance and management; archive identification and location; dates of construction, structural modifications or demolition);
- technical information (structure type, materials and geometry; description and organization of the structural elements);
- heavy transports (parameters necessary for the calculations needed to authorize the passage of special vehicles; design loads and present bearing capacity; vertical and horizontal clearances over and under the bridge).

7.9.4. Updateable Information

The most relevant updateable information is (El Marasy):

- the name of the department responsible for inspections;
- the name of the inspectors;
- the inspections date;
- the atmospheric conditions during the inspections;
- the list of elements to inspect;
- the list of structure main components that contain the elements to be inspected;
- the deficiency level of each inspected element;
- the defects causes;
- the recommended maintenance work;
- the condition rating of each element, main component or the structure as a whole in terms of safety and functionality;
- maintenance cost estimates, etc.

7.9.4.1. Inspection

The system provides a list of structures that must be inspected during a certain period. This selection is made based on the date of the last inspection and the period recommended at that time for a new inspection. The report includes administrative information, the expected inspection duration and the personnel required, the necessary means of access, and possible restrictions to the inspection. The system also provides simplified drawings of the structure (plants, views, and transversal cuts) as inspection auxiliaries. Every element that needs to be analyzed is identified with a number that is shown on the drawings and on the checklist.

The in situ defect rating is made based on the period during which the inspector is convinced that the structure's safety and functionality are guaranteed (El Marasy): 0 (no defect was detected), 1 (there are defects but they do not affect the structure), 2 (5 years are guaranteed), 3 (2 years), 4 (1 year), 5 (6 months) and 6 (imminent danger). All inspection data as well as the recommended maintenance work are stored in the system, thus giving rise to generation of a defects provisional report. The system automatically selects the highest condition rating for each structure's main component and the elements that are part of it. The highest condition rating of the structure's main components is attributed to the structure as a whole. The user may correct this rating when justified (because the system does not take into account the extent of defects or the structural importance of the element affected).

From this point on, every decision is the responsibility of an "assessment committee" that has the following tasks:

- to standardize the inspection results (defects, their causes and maintenance recommendations);
- to calibrate the structures general condition rating;
- to decide if the inspection results are inconclusive, and if so to recommend a special inspection and further investigation:
- to propose a new inspection date;
- to determine the technical priority of the maintenance works;
- to prepare the definitive inspection report.

7.9.4.2. Maintenance

The system backs the maintenance strategy by delivering the following reports (El Marasy):

- priority ranking of all detected defects (in accordance with their respective rating);
- maintenance units selected by the user (similar defects groups that can be repaired in a single maintenance action); the report contains administrative information, the identification and description of the structure under analysis, the number of elements affected, their description and defect causes, the recommended maintenance work, and the deadline for its execution;
- intermediate maintenance units planning;
- maintenance projects also selected by the user (maintenance units grouped within the same structure that, for practical or efficiency purposes, can be executed simultaneously);
- maintenance project planning and necessary intermediate budgets;
- maintenance work delays resulting from lack of funding.

7.9.4.3. Bridge History

The system stores information related to bridge life that is accessible to the user. This information includes every inspection date, every maintenance work date, every main component condition rating after the work, inspection period, and the effective costs of maintenance.

7.9.5. Maintenance Models

The objective of maintenance models (Van Der Toorn and Reij 1990) is to predict the timing and scope of future maintenance such as inspections, repairs, and replacements. To do that, the models need the following information: characterization of the present situation; predicted future behavior (deterioration models); and estimated costs related to maintenance or the absence of it.

The behavior of materials and elements over time is different for traditional materials than it is for electromechanical elements. Therefore, two different deterioration models are proposed. The difficulty in applying these models in practice is that it is very difficult to simultaneously optimize the maintenance strategy for every element within the same structure.

7.10. The Swedish National Road Administration BMS (Sweden)

7.10.1. Introduction

The Swedish National Road Administration, which is responsible for maintaining about 10,000 bridges, prepared a bridge management system that would optimize the high investment made annually for the use, maintenance, repair, and replacement of its bridges. This system is described in Lindbladh (1990).

Ingvarsson (1990) presents the results of a statistical analysis in which the costs referred to previously were tentatively related to various parameters (total deck area, average bridge age within the network, average number of annual freeze-thaw cycles, quantity of deicing salts used annually, traffic intensity) through a bridge deterioration index.

7.10.2. General Organization of the System

In addition to a database that is described in Section 7.10.3., it was the intention of the Swedish authorities to include in the system routines for (Lindbladh 1990):

- inspections, condition assessment, load-carrying capacity classification;
- selection of planned action, optimization per shortcoming and bridge;
- prioritization, optimization per road network/bridge stock;
- specification of commonly performed maintenance and minor repair tasks;
- economic and technical follow-up;
- reporting;
- route finding for heavy transports.

7.10.3. The Database

The database consisted of five sections, with two more under preparation (Lindbladh 1990):

- drawing section (location and general information about the final drawings and their respective micro-film);

- administrative section (bridge name, responsibility for the bridge maintenance, year of construction, routes served or passed over, water course passed over, etc.);
- technical section (structural type, materials, spans, waterproofing and paving types, foundations type, bearings and joints, deck surface area, etc.);
- load-carrying capacity section (standard loads allowed, authorization for passage of specific vehicles, carriage width and vertical under-clearance, etc.);
- damage section (inspection type, structural elements condition rating, bridge global condition rating, defect types, location and extension, repair cost estimates, urgency of action, etc.);
- planning section (planned and factual costs, traffic blockage costs, discount rates, rehabilitation measure planning calendar, etc.);
- projecting section (information to facilitate the determination of the design actions).

7.10.4. Inspection

Four types of inspection are considered (Lindbladh 1990):

- superficial inspection (maximum period of 1 year);
- general inspection (maximum period of 3 years, between each 2 principal inspections);
- principal inspection (maximum period of 6 years);
- special inspection (nonplanned).

Principal inspections are the backbone of the inspection system. Their objective is to detect defects and shortcomings that may affect the bridge's functionality or safety over a period of 10 years subsequent to the inspection. The inspection must allow for short-distance daylight visualization of virtually every bridge element (even underwater) and its accesses. To do that, it very often is necessary to resort to special means of access (fixed scaffolding, ladders, boats, and bridge lifts) and divers. Whenever necessary, in-depth carbonation and chloride measurements will be carried out.

General inspections are a bit less detailed than principal inspections. Principal inspections take place whenever it is considered necessary to investigate any faults, shortcomings, or other observations made during periodic inspections in more detail.

Every defect is described in terms of location, type, cause, and consequence. They are also rated based on the limitations to the normal use that they cause for the element in which they were detected: 0 (normal use is guaranteed for the next 10 years), 1 (defective function after 3 to 10 years), 2 (the same before 3 years) and 3 (defective function detected by the inspection).

All bridges in operation have a load-bearing capacity rating that is stored in the database. This rating reflects the present bearing capacity taking into account the defects and shortcomings detected. This rating may be affected by the results of the inspection. If any of the bridge's main elements is rated 3, according to the criteria described previously, an investigation is immediately started to determine whether it is necessary to reduce the permissible traffic load.

7.10.5. Planning

The decision and action planning system defined four degrees of urgency of action: 3 (action required as soon as possible, if possible on the occasion of the inspection); 2 (action required within 3 years); 1 (the same within 1 year); and 0 (no action required within the next 10 years). Standards have been prepared in which explanations are provided on how to define these degrees for the most frequent defects.

At a second stage, actions are optimized within each bridge. Several alternative actions, all of them satisfying minimum requirements of both the economic and technical points of view, which take into account functional failure costs, can be utilized. Several alternatives are compared based on their respective present value costs for a fixed discount rate.

Enevoldsen and Pup (2000) present a more advanced approach to assessing whether individual bridges have a substandard load-carrying capacity classified, according to the specific requirements of general deterministic codes. As long as the overall level of safety defined by the codes, and determined with a probabilistic-based approach using the reliability index β and the yearly probability of failure p_f (Table 7-2) (Enevoldsen and Pup 2000), is satisfied, the bridge classification can be upgraded, thus cutting the strengthening/rehabilitation costs. The decision process for upgrading bridges within the Swedish National Road Administration is under revision from its traditional architecture (Figure 7-22, left) (Enevoldsen and Pup 2000) to the revised architecture (Figure 7-22, right) (Enevoldsen and Pup 2000).

7.11. The Finnish Roads Administration BMS (Finland)

The Finnish Roads Administration Bridge Inspection Manual (FRA 1989) describes all inspection procedures in Finland. There are the following types of inspections: final, general, special, and annual, as well as intensified monitoring. The inspection sequence, to be entered on the bridge standard inspection form, is as follows: structural member (a list of designations is provided) → material (a list of possibilities is provided) → type of damage (according to a set of 14 standard lists in Appendix 2) → damage class (on a scale from 0—in good condition—to 4—in poor condition)→ damage location (a standard procedure is proposed) → extent of damage (estimated in situ) → reason for damage → urgency class (from "to be repaired later than three years" to "to be repaired immediately") → cost (using the unit costs from Appendix 3) → extent → repair procedure (recommended depending on the type of damage).

Table 7-2. Safety requirements in the ultimate limit state according to the Swedish codes

Failure consequence (safety class)	Failure type I, ductile failure with remaining capacity	Failure type II, ductile failure without remaining capacity	Failure type III, brittle failure
Less serious	$p_f \leq 10^{-3}$	$p_f \leq 10^{-4}$	$p_f \leq 10^{-5}$
(Low safety class)	$\beta \geq 3.09$	$\beta \geq 3.71$	$\beta \geq 4.26$
Serious	$p_f \leq 10^{-4}$	$p_f \leq 10^{-5}$	$p_f \leq 10^{-6}$
(Normal safety class)	$\beta \geq 3.71$	$\beta \geq 4.26$	$\beta \geq 4.75$
Very serious	$p_f \leq 10^{-5}$	$p_f \leq 10^{-6}$	$p_f \leq 10^{-7}$
(High safety class)	$\beta \geq 4.26$	$\beta \geq 4.75$	$\beta \geq 5.20$

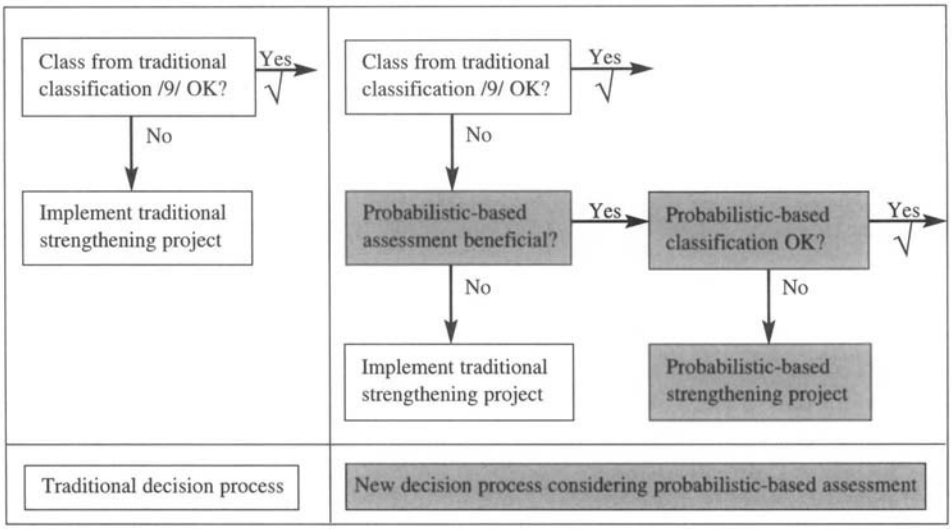


Figure 7-22. Traditional decision process for upgrading of existing bridges (left) and revised decision process including consideration of the probabilistic-based approach (right)

Söderqvist (1999) succinctly describes the Finnish National Road Administration bridge management system. It consists of the following elements: a bridge database, a network level bridge management system, and a project level (for individual bridges) bridge management system. Bridge deterioration is modeled using both a deterministic (a scale from 0—no damage—to 4—serious damage) and a probabilistic approach. Repair measures for each damage class and type of structure are recommended. To arrange the bridges in order based on urgency within the work program, a repair index, as a function of the estimated condition of the bridge's structural parts, the damage class, and the repair urgency class, is calculated.

Kölkönen and Marshall (1990), Marshall and Söderqvist (1990), and Söderqvist and Veijola (1996) also concern bridge management in Finland, but do not present any important innovations.

7.12. The Polish BMS (Poland)

Legosz et al. (1996) describe the bridge maintenance planning system that relies on an urgency of repair ranking according to the following primary diagnostic attributes:

- technical condition of the bridge (from 0—failure condition—to 5—excellent condition);
- road's maintenance standard (from 1 to 4);
- estimated durability (from 0—out of service—to 5—new structure);
- hindrances effect (from 2—without constraints—to 5—a detour or a temporary crossing is necessary);
- repair work urgency (from 0—absolute priority for work—to 1—all other cases).

In Legosz and Wysokowski (1993), a general description is given of the Polish management system, which basically consists of a basic Inventory Module, a Central Inventory of

Large Bridge Structures, a Collection of Bridge Structure Problems program, a Current Maintenance of Bridge Structures program, a Major Repairs of Bridge Structures program, a Warding Contracts by Tenders program, and an Overload Traffic program. Management is performed at three levels: basic, regional, and country. The system does not present novelties when compared with the other systems already described.

7.13. The SAMOA Program (Italy)

Camomilla and Romagnolo (1999) describe in general terms the SAMOA program (surveillance, auscultation, and maintenance of structures) and the correspondent bridge management system. It is a classical system whose general characteristics are very close to the general characteristics of a standard system that the authors present in Chapter 8, and therefore it will not be described here. Another feature, concerning the complete privatization of Italian highways, is, however, very innovative. It concerns toll tariffs and the fact that they are correlated with the overall quality of the road network, defined by the parameter Q:

$$Q = \Sigma\, I_i\, p_i \tag{7-3}$$

where

I_i is the indicator of performance and varies between 0 and 100

p_i is the weighting attached to the single characteristics of quality as measured by the relative indicator of performance.

Fourteen indicators have already been identified pertaining to the following groups: pavements, structures, and safety and services.

Camomilla et al. (1990), Gusella et al. (1996), and Malisardi and Nebbia (1987) concern several noninnovative aspects of bridge management in Italy.

7.14. BMS in Germany

A defects rating system used in a research program for concrete bridges is described in Wicke (1988). The following product is determined:

$$G\, k_1\, k_2\, k_3\, k_4 \tag{7-4}$$

where

G = reference value constant for each defect type

k_1 = extension factor ($0.5 < k_1 < 1$) that takes into account the area in which the defect was detected and/or the frequency with which the defect arises

k_2 = importance factor ($0.5 < k_2 < 1$) that defines the damage level

k_3 = structural factor ($0.3 < k_3 < 1$) that takes into account the effects of the defect in the load-bearing capacity of the cross-section or in the whole structure under analysis

k_4 = urgency factor ($1 < k_4 < 10$) that express the haste with which the repair of a certain defect type needs to be performed

In order to obtain the structure's general condition, the values obtained using the preceding equation for each type of defect are summed up.

Opitz (1993) describes a Bridge Control System that was designed to use active management and control of road bridges. By measuring real-time traffic and load flow on the bridge as well as environmental data, the system can restrict load violations caused by overloaded trucks and those violating the bridge speed limit from damaging the bridge. The data analyzed can be used for decision making in maintenance, bridge construction and environmental modeling. A defect detection system based on fiber optics is used. In this way, better utilization of the bridge and its resources can be optimized.

Other references on bridge management in Germany (Krieger and Haardt 2000; Lebek 1988; and Zichner 1987) do not present novelties in relation to the systems described previously.

7.15. BMS Japanese Experience

Isoura (1989) describes the maintenance program of a network of metallic railway bridges in Japan, with special emphasis on the fatigue problem, which is generally not a concern in concrete bridges. A summarized description of the system of inspection and the degree of rehabilitation based on urgency ratings follows.

All structures are inspected visually at least once a year (general inspection). According to the results, the structure condition is evaluated and categorized in one of the following classes: A (short-term action necessary), B (tending to evolve to class A in the meantime), C (minor damage), and D (no structural defects). The structures of class A are subject to a special inspection, more detailed and relying on specialized personnel and equipment. After this inspection, the bridges are divided into three subclasses: AA (immediate action necessary), A1 (action required during the year after the inspection), and A2 (the same for 2 years). Based on this classification, a strategy of maintenance and repair is prepared.

In Miyamoto et al. (1989), the knowledge-based system of concrete bridge rating in terms of structural defects is described. This system is intended to minimize the uncertainty related to the subjective character of the description of the defects made by different inspectors on different occasions.

The system consists of seven main components: (1) the knowledge base, (2) the inference program, (3) the reference data module, (4) the calculation module, (5) the user's help module, (6) the knowledge acquisition module, and (7) the user's interaction module. The inspector stores the results of the inspection by choosing standard answers to a set of standard questions. The system provides a rating table for each main element (in terms of design, construction, operation, durability, functionality, etc.) in which probability percentages are assigned to each possible rating (five degrees of relative safety/danger). A factor that "measures" the degree of uncertainty related to the collected inspection data subjectivity is provided.

Miyamoto et al. (1993) and Yokoyama et al. (1996) describe the bridge management system implemented by Public Works Research Institute of the Ministry of Construction. The system is based on a deficiency rating I (Table 7-3) (Yokoyama et al. 1996) for each type of defect within each structural part of each bridge within the network are provided in tabular form in a manual. The demerit rating d for each type of defect is determined using the appropriate equation:

$$d_{\mathrm{I}} = I;\ d_{\mathrm{II}} = 0.5 \times d_{\mathrm{I}};\ d_{\mathrm{III}} = 0.2 \times d_{\mathrm{I}};\ d_{\mathrm{IV}} = 0.05 \times d_{\mathrm{I}};\ d_{O.K.} = 0 \qquad (7\text{-}5)$$

Table 7-3. List of deficiency ratings

Deficiency rating	Description
I	Serious damage. There is a possibility of causing trouble in traffic. Detailed investigation or rehabilitation must be performed immediately.
II	Damage in large area. Detailed investigation is required. Following the investigation, necessity of immediate repair work should be evaluated.
III	Damage. Follow-up investigation is required.
IV	Slight damage. Inspection data are recorded.
O.K.	No damage.

The demerit rating of each structural part is the maximum value of the demerit rating for all defects found within it. The bridge condition rating is calculated by adding up the demerit ratings of all the parts and subtracting the sum from 100. As several of these assessments are made at different times, a standard deterioration curve can be devised.

Seriously damaged bridges are rehabilitated as soon as possible. Bridges in very good condition require no rehabilitation. As for the others, three rehabilitation plans for each bridge are made:

- repair only the most damaged part(s) of the bridge;
- repair all parts for which the deficiency ratings are less than III;
- replace the bridge.

For each plan, the rehabilitation cost is determined and its benefit, β, is calculated by:

$$\beta = C_0 - C_1 \tag{7-6}$$

$$C_0 = A\left(\frac{(1+i)^T}{1+i^T-1}\right)\left(\frac{1}{(1+i)^{t_r}}\right) \tag{7-7}$$

$$C_1 = A\left(\frac{(1+i)^T}{(1+i)^T-1}\right)\left(\frac{1}{(1+i)^{t_r+l}}\right) \tag{7-8}$$

where

C_0 = present value of the rehabilitation cost in the future if no rehabilitation is performed at present and the bridge is replaced in year t_r from now and is afterwards replaced at T-year intervals

C_1 = present value of the rehabilitation cost in the future if rehabilitation is performed at present and the bridge is replaced $t_r + e$ years from now and is afterwards replaced at T-year intervals (Figure 7-23) (Yokoyama et al. 1996)

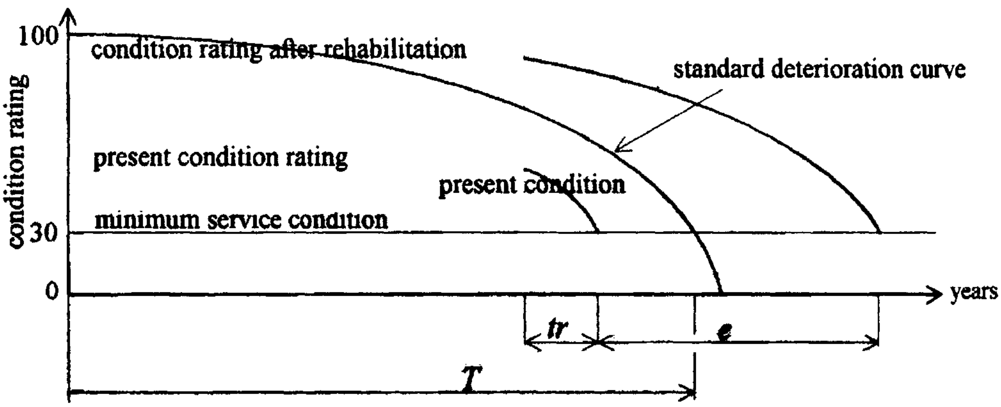


Figure 7-23. Standard deterioration curve used to calculate benefits

A = replacement cost of the bridge

I = discount rate

The benefits and costs of all plans for all bridges are determined. A list is prepared of all plans in order of increasing cost and the incremental benefit cost/ratio for each plan is determined. Plans with a ratio below 1 are removed. All plans are ranked in descending order of the ratio. The plans are selected according to ranking until the annual budget is spent.

Matsui and Muto present a rather complex rating for evaluating the deterioration of reinforced concrete slabs of highway bridges, in terms of residual load-carrying capacity and residual lifetime; both cases based on damage patterns.

A research committee on behalf of the Japanese Prestressed Concrete Contractors Association produced a Manual of Maintenance of Concrete Bridge Structures (JPCCA 1994) to be applied to the Hanshin Expressway. In it, it is advised that concrete structures are planned taking into account maintenance space (several tables with minimum requirements are given) and durability (depending on the structural element considered). Concrete structures are to be designed and constructed taking into account durable materials, durable structures, structures with easy maintenance, design drawing requirements for proper execution of construction works, and maintenance data requirements (specific guidelines with photos and drawings are given on each of these topics).

7.16. The South African National Roads Agency and Taiwan Area National Freeway Bureau Rating System

A rating system is used by the South African National Roads Agency, within a bridge management system adapted from the one used by the Taiwan Area National Freeway Bureau (Nordengen et al. 2000). This system has been used in the approach to condition assessment and has the following components:

- D represents the degree of severity of the defect;
- E is the extent of the defect on the item under consideration;

Table 7-4. Defect rating system for the Taiwan and South African bridge management systems

Category	0	1	2	3	4
Degree (D)	Not applicable	None	Fair	Poor	Critical
Extent (E)	Unable to inspect	Local			General
Relevancy (R)	Uncertain	Minimum	Minor	Major	Maximum
Urgency (U)	Monitor only	Routine	Within 5 years	Within 2 years	As soon as possible

- R is the relevancy of the defect, which considers the consequences of the current status of the defect with regard to the serviceability of the bridge and the safety of the user.

The inspector is also required to rate the urgency, U, to carry out the remedial work to repair the defect as well as to select from a standard list the remedial work activity (and estimated quantity). Since each activity has a unit rate within the budget module, it is possible to determine an estimated budget for the repair of the structure.

The rating is essentially a four-point system (1 to 4) summarized in Table 7-4 (Nordengen et al. 2000).

7.17. BMS in Other Countries

A number of references have been collected that are directly or indirectly related to bridge management systems in several other countries: India, Belgium, Austria, Cyprus, among others. However, the data on these references were rather scarce and too widespread in nature, giving no precise indication of how bridges are managed, particularly the implementation of expert systems. It was decided that the information gathered and summarized in Sections 7.1 to 7.14 of this chapter already gives a clear indication of how existing management systems function outside North America. In the references, the documents are listed according to the country of the system described.

In Woodward et al. (2000) a comparison of the bridge management systems of 11 different countries (including Denmark, Finland, France, Germany, Norway, Slovenia, Spain, the United Kingdom, and the United States) is presented. Most countries do not use past condition data or a deterioration model to predict future conditions, and financial consequences of traffic disruption caused by maintenance work and the associated traffic management are not calculated. Most countries also do not use management systems to make decisions on maintenance and repair, to generate an optimal (minimum cost) maintenance strategy subject to constraints such as the lowest acceptable level of condition or to produce a prioritized maintenance strategy for the bridge stock when the maintenance budget is insufficient.

Kaschner et al. (2000) give a brief description of the work carried out under Work package 2 of the BRIME project, briefly described in Woodward et al. (2000), the objective of which is to develop a framework for the structural assessment of bridges. The following tasks have been identified:

- a review of current procedures and standards used for bridge assessment in Europe;
- the development of models that take into account bridge-specific traffic conditions and material properties;
- the use of reliability methods based on a probabilistic approach for bridge assessment including the use of measurements for updating the reliability of structural elements;
- recommendations for methods and procedures that can be adopted for the assessment module of the management framework highlighting where further development will be beneficial.

The reasons for initiating a bridge assessment have been identified and can be summarized as follows (reason 3 is still the least common):

1. when there is a need to carry an exceptional heavy load;
2. where the bridge has been subjected to change such as deterioration, mechanical damage, repair or change of use;
3. where a bridge is of an older type built to outmoded design standards or loading and has not been assessed to current standards.

The main conclusion of the work so far is that semi-probabilistic methods are suitable for many assessments even though reliability methods may be preferable, namely where deterioration of the structure is significant.

Daly (2000), yet another reference concerning the BRIME project, describes a project to study the modeling of deteriorated structures, focusing on the common forms of deterioration found in bridges and the effects they have on assessed capacity, with the ultimate aim of using these models to estimate reliably the bridges load carrying capacity within a general assessment procedure. A questionnaire provided an estimate of the percentage of substandard bridges in each of the countries involved in the project: 39% in France, 42% in Germany, 42% in Norway, 15% in Slovenia, and 30% in the United Kingdom.

Blakelock (1993) reports on the experience of developing and applying a computerized bridge management system in the United Kingdom, Sri Lanka, and Hong Kong. The system database consists of the following modules: inventory, inspection, maintenance/financial, history, and program/study and system administration. The system also includes a query and reporting system and obviously does not have a decision-making module.

Lauridsen and Lassen (1999) report that the Danish DANBRO system has been implemented for the national highway administrations in Saudi Arabia, Mexico, Colombia, Honduras, Croatia, and Malaysia.

CHAPTER 8

Management During Service Life

8.1. General Summary of the Bridge Management Systems Analyzed

8.1.1. General Organization of the Systems

Although some of the systems described in Chapters 6 and 7 are incomplete, their organization includes, in general terms, the following modules (de Brito 1992):

- a computer database where all the information concerning a bridge (from design to demolition) is stored, along with the reference data to be used by the inspection and decision modules;
- an inspection module in which the inspection's periodicity, personnel and necessary equipment, standardized procedures, defects susceptible to detection in situ and their causes, actual data to be collected, reports preparation, and so forth are all stored;
- a module that takes advantages of the inspections data, which, in the most developed systems, is divided into two submodules: maintenance/small repair and repair/strengthening/replacement; in each of these submodules, a decision system is defined that, based on the results of the inspections and social, economic, and functionality considerations, facilitates the definition of the works that need to be implemented by ranking them by priority.

8.1.2. The Database

The databases used in bridge management share the following main characteristics: user-friendliness (in both reading stored data and adding new data), capacity to deliver specific reports adapted to the users' needs, data selectivity to avoid having to demand an overly high memory capacity, the possibility of having part of the data transferred to portable microcomputers susceptible to be used in situ, and so on.

Generally, existing databases are organized as follows (de Brito 1992):

- bridge's reference module, including all the data that rarely undergo any change during the bridge's life (identification, administrative data, technical data with the description of the reference state, load-bearing capacity, etc.);
- system reference module, which includes the main data catalogs used by the system at the inspection stage (defects and diagnosis methods classification systems, inspection manuals), at the decision stage (unit costs of every maintenance/rehabilitation action, discount and inflation rates, standard or otherwise vehicles, etc.), and at the work proposal stage (repair actions list);
- inspection and maintenance/rehabilitation supporting module, of a fundamentally variable nature, in which all the relevant information collected during the inspection (team composition, equipment used, atmospheric conditions, date, location of defects, extension and rating, reports of the campaigns of measurements collected with the in situ surveillance equipment, work to be done recommendations and their respective schedule, etc.) and after the maintenance/rehabilitation work has been performed (planned and factual costs, end of the work date, work description, personnel and equipment used, defects reclassification, etc.) is stored.

8.1.3. The Inspection Module

The general organization of the inspection varies significantly from country to country and within each country according to the different bridge authorities. However, with rare exceptions, the general pattern is as follows:

- superficial inspections, which from this point on will be called "current," occur approximately every year and are based simply on visual observation of the structure and unsophisticated portable support and measurement equipment;
- thorough inspections, which will be called "detailed" from here on, with a period equal to a multiple of the current inspections period, and representing a kind of "checkup" of the structure; they are general in nature since no particular defect is investigated in detail; in addition to a detailed visual observation made by personnel with some degree of experience, some control equipment is used; the use of special means of access is also a possibility;
- special inspections, which will be called "structural assessments" are nonperiodic by nature since they are performed only after defects capable of jeopardizing the safety or the functionality of the structure are detected; they have a specialized character and are frequently limited to a specific phenomenon in a restricted part of the structure; very specialized equipment and personnel are mandatory.

Table 8-1 (de Brito 1992) summarizes the general organization of the inspection within the management systems described previously.

8.1.4. The Inspection Actions Examination Module

In several of the systems described previously, this module is still scarcely developed and systematized, and the decision-making responsibility lies totally with the user since the system

Table 8-1. Inspection strategies in several countries

Inspection	Periodic			Non-Periodic	
Country	**Current**	**Detailed**	**Others**	**Structural assessment**	**Others**
Belgium (de Buck 1987)	Routine inspection (1 year)	General inspection type A (3 years)		General inspection type B	
Cyprus (May and Vrahimis 1990)	Routine condition (1 year)		Quarterly checks (3 months)		Follow-up checks; detailed condition
Denmark (Sørensen and Berthelsen 1990)	Superficial inspection (<1 year)	Principal inspection (3 years in average)		Special inspection	
Finland (Kähkönen and Marshall 1990)	Annual inspection (1 year)	General inspection (3 years)	Final technical inspection (at handing over); special control (2 to 6 months)	Special inspection	
United States (Fla.) (Little 1990)	Routine inspection (1 or 2 years)		Initial inspection	Special inspection (at handing over)	
France (MTRD 1979)	Annual inspection (1 year)	Periodic detailed inspection (5 years)	First detailed inspection (at handing over)	Exceptional detailed led inspection special inspection	Continuous surveillance
Germany (Zichner 1987)	Visual inspection (3 months)	General inspection (3 years); principal inspection (6 years)			
Italy (Malisardi and Nebbia 1987)	Visual inspection (3 months)	Specialized inspection (1 year)		Structural assessment	
Japan (Miyamoto et al. 1989)	General inspection (1 year)			Special inspection	
Ontario (Canada) (Reel and Conte 1989)	General inspection (1 or 2 years)	Detailed inspection (2 years)	General inspection (1 day/1 month)	Condition survey	General inspection detailed inspection
Portugal (Santiago 2000)	Ordinary inspection (3 to 6 years)	Principal inspection (3 to 6 years)			Extraordinary inspection
Sweden (Lindblath 1990)	Superficial inspection (1 year)	General inspection (3 years); principal inspection (6 years)		Special inspection	
Switzerland (Andrey 1987)	Routine inspection (15 months)	Periodic inspection (5 years)	First special inspection (at handing over)	Special inspection	
United Kingdom (Holland and Dowe 1990)	general inspection (2 years)	Principal inspection (6 to 10 years)		Condition survey	

does not propose any solutions. Conversely, the vast majority of systems goes no further than maintenance and do not propose solutions for the resolution of the scarcity of the available resources. Only the Canadian, Pennsylvanian, Danish, and Swedish systems go a bit further and propose the decision subsystems that are described next.

8.1.4.1. Maintenance/Small Repair Subsystem

At this level, decision making is based fundamentally on the results of the current/detailed inspections in terms of defect ratings and the approximate cost estimates. Very elaborate long-term economic studies are not necessary, and the work is performed according to a priority list proposed by the system until available funds run out.

8.1.4.2. Repair/Strengthening/Replacement

At this level, decision making is always preceded by a structural assessment of the bridge during which the most worrisome defects detected and the work needed to neutralize them are evaluated and quantified in some detail. Fine estimates of the costs of the various executable repair hypotheses for each bridge are prepared, along with possible strengthening, deck widening, or replacement of the existing bridge. For each alternative, the functional failure costs are also estimated and the social consequences are taken into account. Based on a medium to long-term economic study calibrated by criteria that change from system to system, it is possible to rank the different alternatives within each bridge.

This analysis is then extended to the bridge network under the responsibility of management to maximize the benefits, thus resulting in an action priority ranking for all bridges. The best alternative at the bridge level may not be the best alternative for the same bridge in the global context of the network.

8.2. The Architecture of a Standard Bridge Management System

8.2.1. Introduction

In Vassie (1996), existing bridge management systems are divided into four types, progressing from simple to complex, with each one providing an additional feature: (1) inventory database; (2) basic inspection and maintenance scheduling and recording; (3) maintenance scheduling taking into account the rate of deterioration; (4) maintenance scheduling minimizing whole life costs and prioritizing where the budget is constrained. The more sophisticated the system is, the greater are the needs in terms of experience and know-how of the personnel, software and hardware available, running costs, and especially the amount and complexity of input data. This means that more time and resources are spent in gathering the data and that very often the management officer is faced with incomplete data.

Vassie argues that one interesting feature of the evolution of bridge management systems is that much of the information required by the most sophisticated systems is obtained through operation of simpler management systems. For example, data on deterioration rates can be derived from the change in condition state with time, and so on until an improvement in condition state following maintenance and the useful life of maintenance systems is derived. Therefore, it is wiser to start with a simple bridge management system and gradually increase its sophistication as required than to start with a complex system. This concept can easily be applied to the proposals made in the remaining part of this chapter.

8.2.2. Requirements of a Knowledge-Based Bridge Management System

As seen in Chapters 6 and 7, only a few management systems in existence or being implemented can be defined as knowledge-based systems and simultaneously include the complete process of managing bridges from inspection to replacement. The systems that come closest to that concept are those from the Highway Engineering Division of the Ontario Ministry of Transportation (Reel and Conte 1989) and from the Railways of Denmark (DANBRO).

The functions of a bridge management system are shown in Table 8-2 and in Woodward et al. (2000).

The desirable characteristics for a knowledge-based bridge management system are (de Brito 1992):

- possibility of storing all the static or updateable information relating to each bridge or system in general in locations to which access is easy and swift;
- standardization of all the process stages from the inspections and their respective reports to the decision making, also including the design/construction survey;
- whenever possible, use criteria that can be programmed into computers instead of making subjective decisions;
- optimization of available resources, in terms of personnel, equipment, time and money;
- minimization of bureaucratic procedures that tend to delay decision-making process;
- total transparency of all decisions made, based on criteria of an economic, structural, or functional nature;
- clear separation of the various management process stages and their respective interdependency.

In Darby et al. (1996), it is argued that a further characteristic required of a bridge management system is that it be flexible enough to retain the engineering judgment of the

Table 8-2. Functions of a bridge management system

Functions of a bridge management system
To provide an inventory of bridges
To record/predict the historical and future condition of elements and components
To record/predict the historical and future load carrying capacity of the bridge
To assess the rate of deterioration
To select the most cost-effective maintenance
To evaluate the cost of various maintenance options
To evaluate traffic management and delay costs
To calculate discounted costs to give a lifetime cost
To assess the implications for safety and traffic congestion of deferring maintenance work
To produce optimal and prioritized maintenance program
To assist with budget planning

bridge manager responsible for individual structures as a key element in the decision-making process. By doing so, the many complex factors that go into making a sensible decision are drawn together. A similar point of view is stated in Brooman and Wootton (2000).

Table 8-3 (Woodward et al. 2000) lists the primary and secondary outputs of the management system at the project level. The outputs at the network level are (Woodward et al. 2000): list/count bridges satisfying specific criteria; list/count bridges overdue for inspection; list/count bridges that are substandard; list/count bridges in the poorest condition; list/count bridges with traffic restrictions; budget for optimal program; count bridges with deferred maintenance; long-term cost of maintenance; prioritized maintenance programming. In the same reference, the proposed process for developing a bridge management system is: requirements → outputs → algorithms → inputs → data fields.

Table 8-3. Project-level outputs

Primary	Secondary
General queries	List and count bridges meeting specified criteria
Inspection history	List and count bridges overdue for inspection
Test history	List and count substandard bridges
Maintenance history	List and count bridges in the poorest condition state
Traffic history	List and count bridges with traffic restrictions
Condition history	Budget needed for the optimal maintenance program
Load carrying capacity history	Number of bridges with deferred maintenance due to a suboptimal budget
Posting history	Long-term cost of deferring maintenance due to a suboptimal budget
Optimal maintenance program	
Prediction of variation of load carrying capacity with time	Prioritized maintenance program for a given suboptimal budget
Prediction of the variation of condition with time	Prioritized maintenance programs based on other constraints
The effect of maintenance and/or strengthening on the future rate of change of condition and/or load carrying capacity	Predictions of load carrying capacity for a specified budget
Cost of optimal maintenance program	Prediction of the condition for a specified budget/maintenance program
Estimation of the cost of maintenance based on the load carrying capacity and condition	Routing of heavy, high, wide, or long vehicles
Estimation of the cost of traffic disruption due to maintenance or traffic restriction	History of different types of maintenance
	History of occurrence of different types of defect
	History of occurrence of substandard bridges
	History of performance of different element types and component types
	Cost rates for different maintenance options
	History of performance of different maintenance methods

Bridges must be monitored by the management system throughout their service life. Specifically, the following stages must be considered (de Brito 1992):

1. **Design**: every project and preliminary study must be stored (in the bridge dossier described in further detail in Chapter 10); a résumé of the most important data therein included must be introduced in easily accessible files (by using a database detailed in Chapter 9);
2. **Construction**: the whole contracting process as well as the documents prepared by the superintendent during construction and the documents generated when the bridge is handed over to the owner are stored in the bridge dossier; the actual state of the bridge after construction when it initiates its service life ("reference state," defined in Chapter 10) is described in the bridge dossier and on a form prepared for that effect contained in the database;
3. **Normal use**: traffic studies of the bridge and corresponding road stretch must be stored in the bridge dossier;
4. **Inspection**: inspections must be standardized by resorting to inspection manuals (Chapter 10) and personnel training manuals, and they must, when feasible, be periodic; the defects detected, their probable causes, and diagnostic methods are standardized and classified to enable an objective rating of the problems detected; reports must be succinct and must be stored either in the bridge dossier or in the database; informatics backing to the inspection must be provided through a computer inspection in situ backing module;
5. **Maintenance**: the works performed on a bridge throughout its life in order to maintain its proper functional capability are only rarely of a structural nature and must, therefore, be separated from structural repair; the inspection reports must contain recommendations concerning the work needed for each bridge; because the needs are always greater than available resources, the system must include a decision module (Chapter 11) that rates the work in terms of priority; all documents related to the work actually performed must be included in the bridge dossier;
6. **Repair**: when the bridge needs structural repair work to regain its initial load-bearing capacity or its functionality, a special inspection must be performed; the system must generate a precise warning about the necessity and planning of that inspection (inspection strategy submodule in Chapter 13); the respective report must be included in the bridge dossier and its main results must be stored in the database; it must contain recommendations about the work needed; a new decision module (repair work selection submodule in Chapter 13) allows the rating of the repair work for different bridges to face budget limitations; the description of the work performed is included in the bridge dossier;
7. **Structural strengthening/Deck widening**: if the functional demands of a bridge surpass the demands previewed at the design stage, the possibility of upgrading its capacity must be analyzed; the system must include a module that performs long-term economic analysis (Chapter 10) of the advantages and drawbacks of possible solutions; the decision module referred to previously treats this situation as a particular case of structural repair;
8. **Demolition/Replacement**: ending the bridge service life is also a decision that can be made by using a knowledge-based system, by performing an economic analysis that proves that it is the best solution.

A general synthesis of the functional aspects of the management system proposed is presented next. Each module of the system is then described in further detail in the following chapters.

8.2.3. General Description of the System

The management system monitors a bridge throughout its service life basically in three ways (Figure 8-1) (de Brito and Branco 1998):

1. Collected data storage
   a. bridge dossier: contains bridge data concerning all service life stages, not especially selected but following a preestablished internal organization;
   b. database: contains only selected data about predetermined bridge service life stages. These data are used by the decision system. The database also contains catalogues of static or semi-static information about the classification system, decision criteria, costs, and other factors.

2. Procedure and report standardization
   a. Inspection: inspection manuals contain the classification system (defects, possible causes, diagnostic methods, and repair techniques; see Chapter 10), the defect and repair forms (Chapter 10), and the identification, reference state, graphic information, and inspection database files.
   b. Maintenance/repair/rehabilitation: rating criteria of the defects detected and the standardized procedures followed within the economic analyses.

3. Decision making
   a. maintenance;
   b. inspection strategy;
   c. rehabilitation work selection/replacement.

The management system can be divided into three different modules (Figure 8-2) (Branco and de Brito 1995):

I—Database module;

II—Inspection system;

III—Decision system.

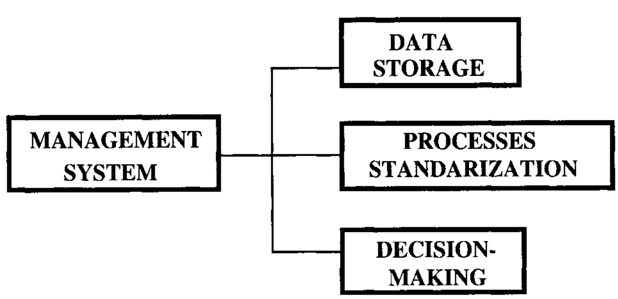


Figure 8-1. Main management system tasks

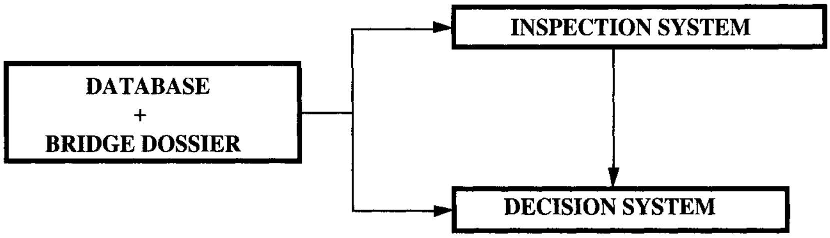


Figure 8-2. General architecture of the management system

The database module includes both the computer database and the bridge dossier.

The inspection system controls the entire process of surveying the bridge from the reception tests until the end of its service life. It allows the generation of part of the information necessary for the decision system.

The decision system is responsible for all options made throughout bridge life concerning its maintenance, the implementation of nonperiodic structural evaluations, and repair, as well as capacity upgrading and replacement.

The decision system is divided into two subsystems (Figure 8-3) (de Brito and Branco 1998):

III. 1—Maintenance/small repair;

III. 2—Rehabilitation/replacement.

The maintenance/small repair subsystem concerns current maintenance work that is performed almost continuously at the bridge and the adjacent access roads to ensure that traffic flows smoothly. The work involves either nonstructural work or work performed on secondary elements.

The rehabilitation/replacement subsystem involves important nonperiodic structural repair work that eventually will be required. As described in Chapter 10, this work is always preceded by a special inspection called a structural assessment. The subsystem is then divided in two submodules (Figure 8-3) (de Brito 1992):

III. 2.A—Inspection strategy;

III. 2.B—Repair work selection.

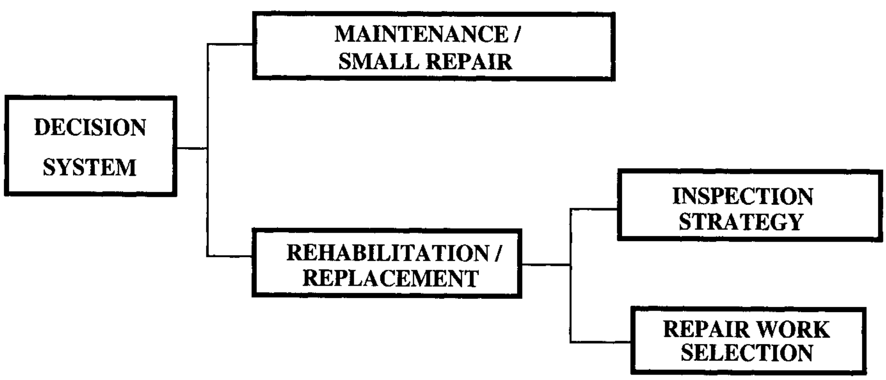


Figure 8-3. Architecture of the decision system

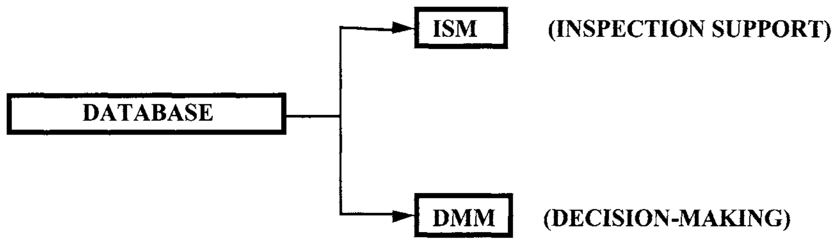


Figure 8-4. Computer-based modules within the management system

The inspection strategy submodule (ISS) checks to determine whether it is necessary to perform a structural assessment and if so, produces a plan for accomplishing that assessment.

Based on a long-term economic analysis and the results of the structural assessment, the use of the repair work selection submodule allows the user to choose from one of the following options concerning a bridge in which significant structural or functional inadequacies have been detected:

- no action (eventually posting the bridge);
- repair (recover the initial structural capacity);
- strengthen the structure;
- widen the bridge deck (usually also strengthening the structure);
- demolish and replace the existing bridge with a new one;
- keep the existing the bridge and build a new one in parallel to it.

Totally automatizing the management system is impossible and certainly would not be convenient. Even when the management system is based on computer programming and can provide a list of specific recommendations, the final decision must always be the responsibility of the person in charge of management system, who will rely on his common sense and experience. The following computer-based modules are proposed (Figure 8-4) (de Brito 1992):

- a database whose function is to store, manage, and provide the other modules with all the basic information as well as the information collected during inspection;
- an inspection support module (ISM) whose function is merely to support the inspector in situ;
- a decision-making module (DMM) that manipulates all the data collected in order to provide sound recommendations at all levels of the decision system (Figure 8-3) (de Brito and Branco 1998).

8.2.4. The Database

A detailed description of the proposed database is given in Chapter 9. Only the most important aspects are described here.

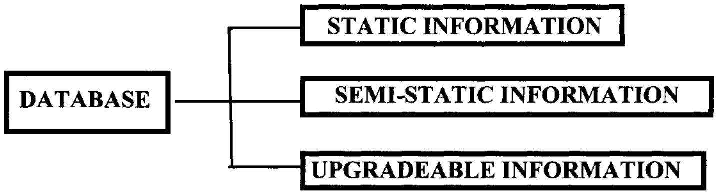


Figure 8-5. Summary of the information contained in the database

In Section 8.1 it was concluded that most existing databases are organized as follows (Reel et al. 1988; Sørensen and Berthelsen 1990; El Marasy):

- bridge reference module;
- system reference module;
- inspection and maintenance/rehabilitation support module.

The database stores three types of information: static, semi-static, or upgradeable (Figure 8-5) (de Brito 1992). The information is listed next, and the chapters in which detailed descriptions can be found are shown in parentheses.

STATIC INFORMATION

- administrative data;
- classification system (Chapter 10);
- correlation matrices (Chapter 10);
- defect forms (Chapter 10);
- repair forms (Chapter 10);
- inspection manuals (Chapter 10);
- defects rating (Chapter 11);
- costs quantification (Chapter 12);
- structural reliability (Chapter 13);
- computer programming (Chapter 12);
- load-bearing capacity;
- identification forms (Chapter 9);
- graphic information (Chapter 9).

SEMI-STATIC INFORMATION

- general nature costs files;
- individual bridges costs files;

- annual budgets;
- load-bearing capacity;
- reference state forms (Chapter 9).

UPGRADEABLE INFORMATION

- inspection forms (Chapter 9).

The following options for general access to the database are proposed (Reel et al. 1988):

- data management;
- reports generation;
- self-maintenance.

The data management option is the most visible part of the database and consists of the following information:

- identification form (the bridge "identification card"; a part of the ISM input);
- graphic information (including general views and details);
- reference state form (including a description of the design plans and the actual situation after construction) as defined in Chapter 10; contains detailed information about the bridge that is essential when an important structural repair is under consideration or if a structural assessment is to be performed;
- inspection forms (describe inspection characteristics, defects detected, and inspections history); they thus represent the final inspection report essential to all DMM steps.

A database user's manual must be prepared, in which all the possible options are described and examples are presented depicting the screens as the user sees them.

8.2.5. *The Inspection System*

8.2.5.1. Inspection Strategy

The inspection system is based on a number of visits to the bridge at fixed time intervals between them, complemented by visits on certain special occasions. Inspections performed at fixed intervals are called periodic inspections, whereas special visits are referred to as nonperiodic inspections. In Table 8-4 (de Brito 1992), similar to Table 8-1, the proposed system is compared with similar systems adopted in France (MTRD 1979) and the system proposed in Switzerland (Andrey 1987).

A detailed inspection replaces a current inspection every 5 years. The type of analysis performed is similar, albeit more detailed. In addition to direct visual observation, nondestructive in situ tests are performed and special means of access are used. The whole structure must be analyzed, with no special emphasis on any localized area (unless previous current inspection reports indicate otherwise) and, in principle, no serious structural defects are expected to be found.

Table 8-4. Inspection system strategy proposed compared with existing ones

Inspection	Periodic			Non-Periodic	
Country	Current	Detailed	Others	Structural assessment	Others
France (MTRD 1979)	annual inspection (1 year)	periodic detailed inspection (5 years)	first detailed inspection (at handing over)	exceptional detailed led inspection	continuous surveillance
Switzerland (Andrey 1987)	routine inspection (15 months)	periodic inspection (5 years)	first special inspection (at handing over)	special inspection	
Proposal (de Brito 1992)	current inspection (15 months)	detailed inspection (5 years)	initial characterization of the bridge (at handing over)	structural assessment	

When serious structural defects *are* found, a structural assessment is mandatory. This type of inspection is thorough and focuses on the localized part of the structure in which the defect(s) were found in order to clarify the findings of the detailed inspection. To do that, it is necessary to use very knowledgeable personnel, along with sophisticated equipment and means of access. Any structural repair or rehabilitation work must be preceded by a structural assessment.

Current inspections occur at 15-month intervals in order to check all aspects that are influenced by the seasons. The bridge and its surroundings are analyzed, resorting almost exclusively to direct visual observation. Personnel, equipment, and means of access are very restricted.

To define the bridge reference state (to which all the inspections refer), it is necessary to perform an initial characterization of the bridge. This generally occurs when the bridge is delivered to the authorities, but it may also occur whenever there is reason to think that the present reference state is no longer valid. The level of detail involved in this type of inspection can be compared to a structural assessment even though it covers the structure as a whole.

In a bridge integrated into the management system from its beginning, the inspection procedures are summarized in Figure 8-6 (de Brito 1992) and Figure 8-7 (de Brito 1992).

8.2.5.2. Inspection Procedures

In order to standardize inspection procedures and the reports generated, a classification system must be created that includes all defects (functional and structural) that predictably may be detected in bridges. Possible causes, repair techniques, and diagnostic methods (Figure 8-8) (de Brito 1992) must also be classified. Defect forms (Figure 8-9) (de Brito et al. 1994) and repair forms have been created as well as a rating list that itemizes diagnostic methods, which are of greater interest during inspections.

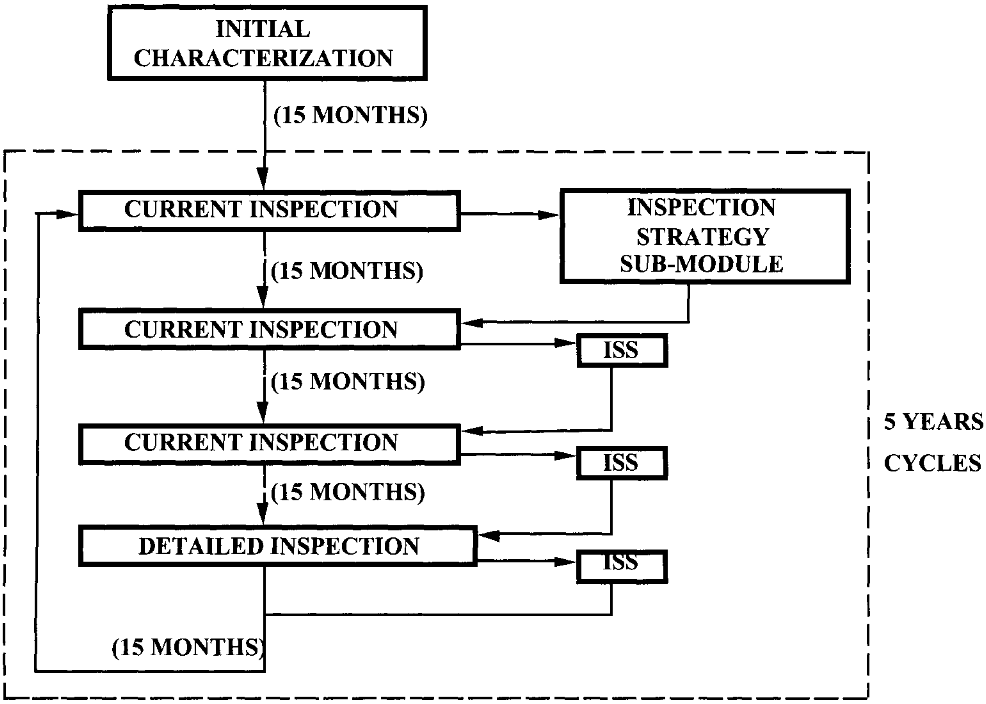


Figure 8-6. Periodic bridge inspections

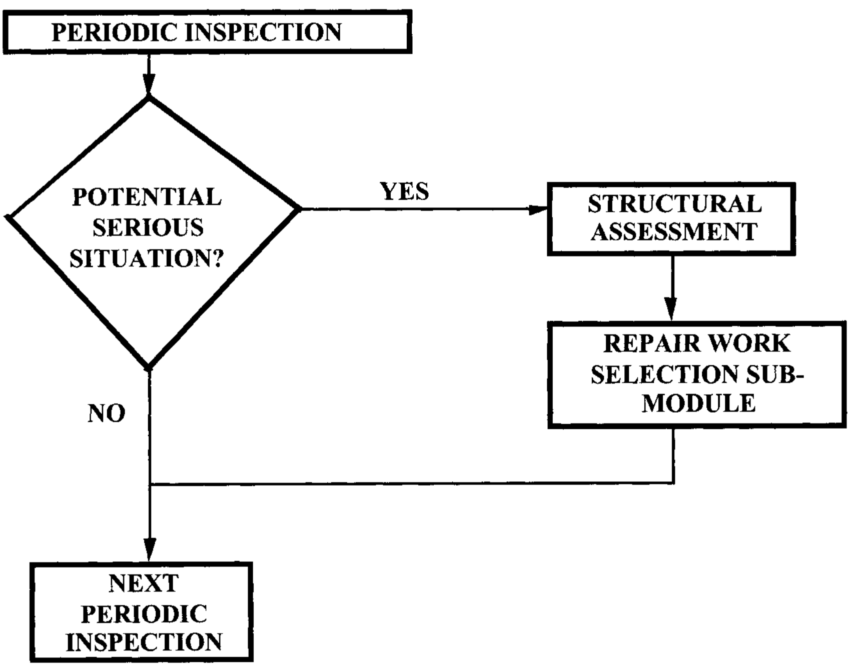


Figure 8-7. Inspection strategy submodule (ISS)

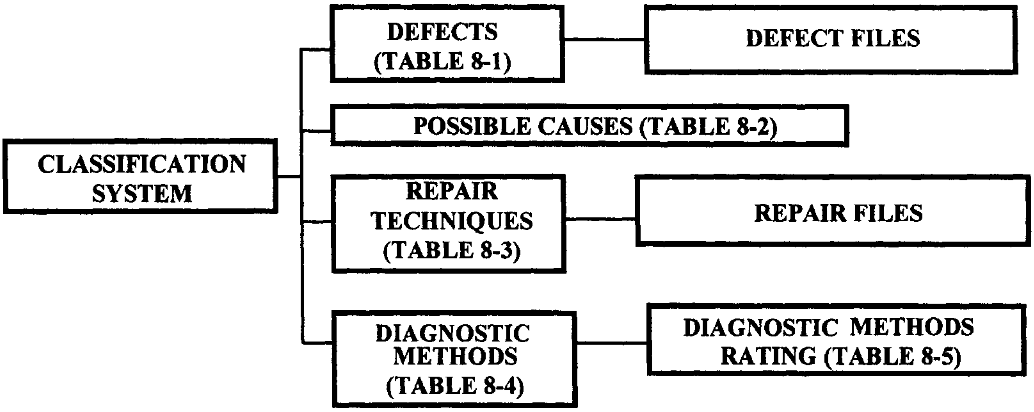


Figure 8-8. Inspection classification system

To facilitate the inspector's in situ diagnosis, a set of correlation matrices (Figure 8-10) (de Brito 1992) related to defects susceptible to detection during inspection has been created. These matrices are the backbone of the inspection support module (ISM) and facilitate the selection of the maintenance and repair works done at the headquarters.

Simplified schematic drawings of the structure are also used to support the inspection. Reference grids adapted to the geometry of the structure are created on these schematics, facilitating easy and clear location of any defect detected. These drawings are brought to the bridge site and written on, an information piece that may or may not be inserted into the database.

All information concerning the bridge, from design to demolition, is stored in the aforementioned bridge dossier, whose internal organization is presented in Figure 8-11 (de Brito 1992).

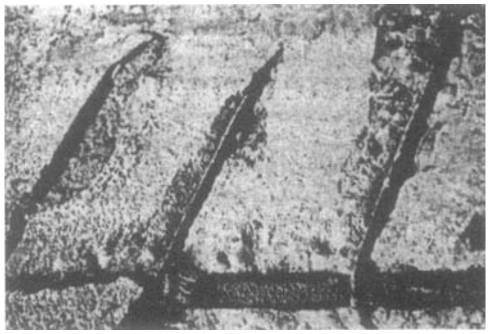
The planning, in situ procedures, and inspection reports all must follow standardized rules so that they do not depend on the subjectivity of the inspector. To accomplish this, inspection manuals for each type of bridge within the network must be prepared. The main topics of a typical manual, which are further developed in detail in Chapter 10, are presented in Figure 8-12 (de Brito 1992).

8.2.6. Computer-Based Inspection Support Module

This computer-based inspection support module (ISM) is but a knowledge-based database that facilitates the inspector in situ task. Unlike the decision-making module (DMM), it is a closed system whose usefulness ends after the inspection and does not directly provide data to the decision system (Figure 8-13) (de Brito 1992).

A portable personal computer must be included with the equipment brought to the inspection site. The computer contains the ISM as well as the data files concerning the bridge(s) about to be inspected. On arriving at the site, the inspector identifies the bridge with its code number and consults the respective identification form. He initiates the inspection and, as he detects the defects and when he feels the need, he accedes to the data provided by the ISM (Figure 8-14) (de Brito 1992). Finally, he prepares a preliminary report that will function as a memory aid in the making of the definitive report and inspection file.

DEFECT FORM



TYPE: REINFORCEMENT / CABLES

CODE NUMBER: A-D05

DESIGNATION: bar with loss of cross-section.

DESCRIPTION: spalling of reinforcement cover and loss of cross-section area.

POSSIBLE CAUSES:
-insufficient cover
(C-A14, C-B11, C-A28, C-B1, C-B2, C-B26).
-chlorides
(C-F-3, C-G3, C-B6).
-carbonation
(C-F2, C-G2).
-faulty drainage
(C-A24, C-A23, C-A25, C-B20, C-B26, C-H5).
-water infiltration (deficient watertightness)
(C-F1, C-G1, C-A26, C-B5, C-B9, C-B17, C-E2, C-E3, CE-4).

POSSIBLE CONSEQUENCES:
-further spalling caused by rust.
-cracking.
-loss of resistance of the element.
-reinforcement loss of bond.
-structure deformation.
-aesthetic defect.

FURTHER INSPECTION FACTORS:

-rust colour: **black** (chlorides - higher loss of section) or **reddish** (carbonation).
-corrosion of nearby reinforcement.
-cover bond.
-carbonation, chlorides, water infiltration.
-cracking in the area.
-deformations.
-drainage system functioning.
-distance from the sea.
-present or past use of de-icing salts.

INSPECTION PARAMETERS:

-rust predominant colour: black / reddish.
-location of main section losses: maximum moments / intermediate areas.
-maximum local section loss (%).

DEFECT RATING:

In terms of Rehabilitation Urgency:
0 - mainly black rust in areas of maximum moments with a local section loss over 3 %.
1 - mainly black rust in areas of maximum moments with a local section loss under 3 %.
2 - predominantly black rust in intermediate areas.
3 - predominantly reddish rust.

In terms of Importance to the Structure's Stability:
A - reinforcement in the deck, main beams, columns, abutments or foundations.
C - reinforcement in the auto-safes, parapets, sidewalks, surface and approach slabs.

In terms of Volume of Traffic Affected by the Defect:

γ - assuming that this defect does not disrupt the normal traffic

Figure 8-9. Example of a defect form

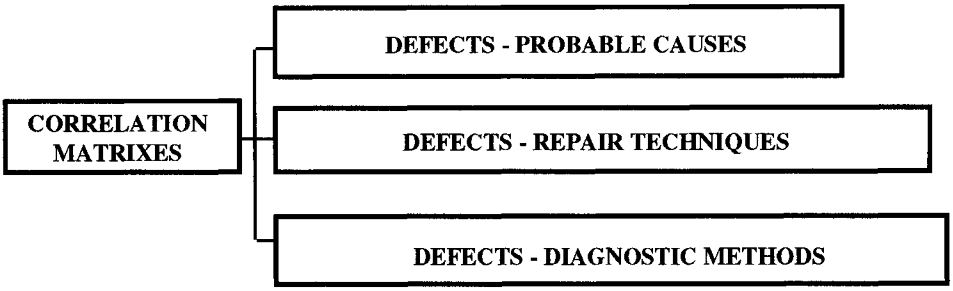


Figure 8-10. Correlation matrixes within the management system

The ISM input includes:

- general information about the bridge (the identification form includes location, general project data, and construction-related general data) stored in the central database;
- correlation matrices (Figure 8-11) (de Brito 1992) stored in ISM files;
- bridge and detected defects identification, both introduced by the inspector at the site.

The ISM output includes:

- type 1 parameter lists (see Chapter 11) necessary to classify each defect detected by the inspector;
- lists of the diagnostic methods related to each defect detected (separated into low and high correlation methods);
- lists of the probable causes of each defect detected (low/high correlated);
- lists of other defects associated with each defect detected (with a correlation index);
- lists of repair techniques recommended for each defect detected (low/high correlated);

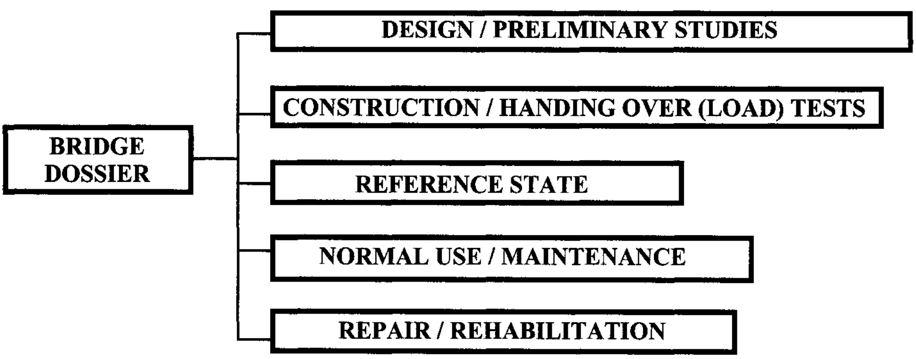


Figure 8-11. Bridge dossier organization

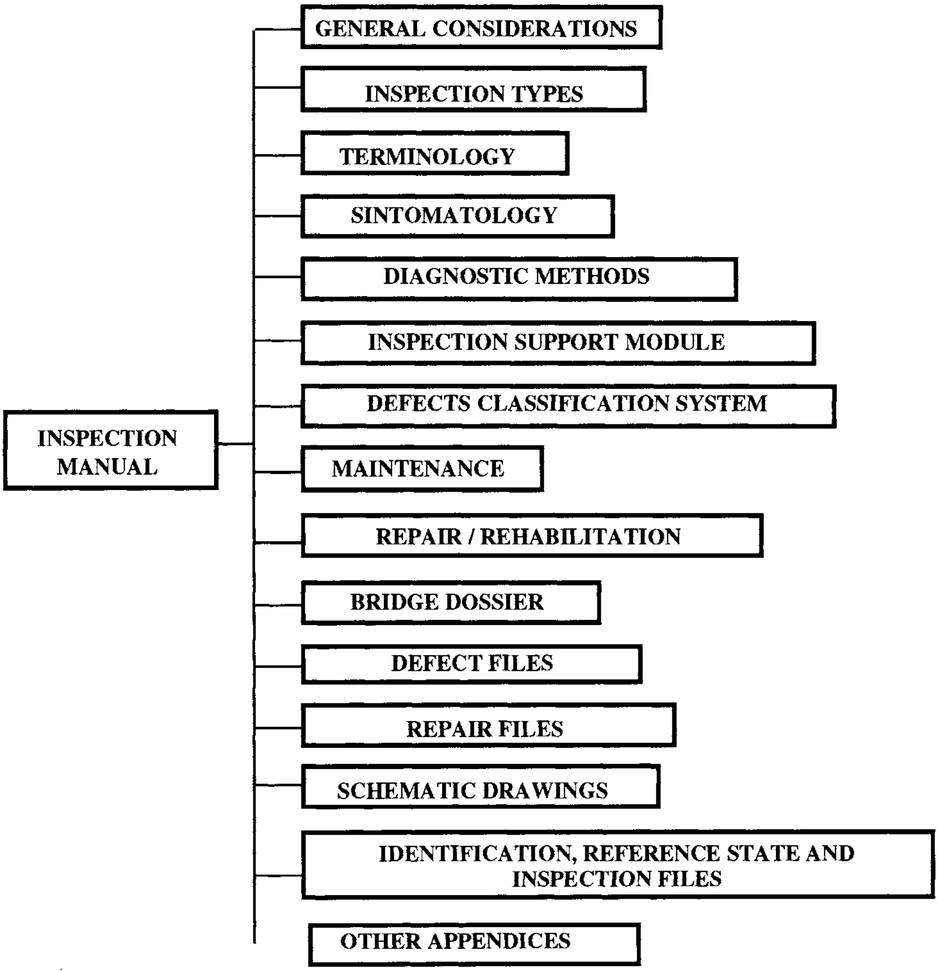


Figure 8-12. Bridge inspection manual, internal organization

- type 2 parameter lists necessary to "measure" each defect detected and to calculate type 3 parameters (see Chapter 11);
- type 3 parameters used in budgeting the maintenance and repair work needed to eliminate each defect detected;
- type 4 parameters necessary for the selection of the most adequate repair technique taking into account the characteristics of each defect detected (see Chapter 11);

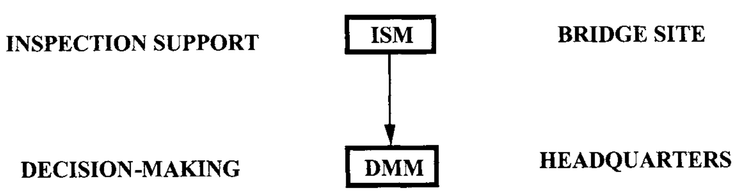


Figure 8-13. Jurisdiction of the management system, computer-based modules

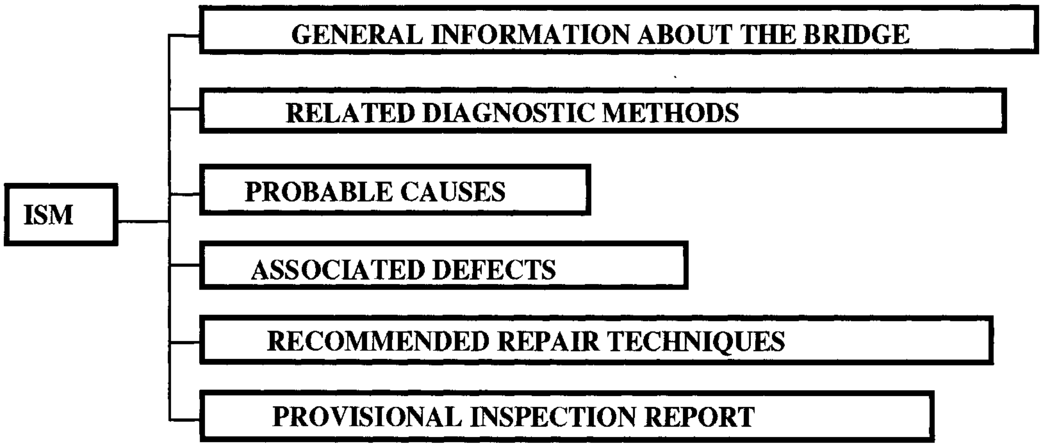


Figure 8-14. Information provided by the ISM at the inspection site

- a provisional inspection report with all the information that it was possible to collect on each detected defect during the inspection.

These basic notions about ISM are described in further detail in Section 8.6.

8.2.7. Decision System

The decision system is the management system module that determines whether the system is knowledge-based and makes up the DMM. By using it, the management authorities are provided with sound recommendations regarding the best option to take during certain relevant moments in the life of the bridge. Figure 8-7 (de Brito 1992) illustrates such moments:

- after periodic inspections, two kinds of decisions must be made:
  - — select which maintenance related work must actually be performed;
  - — verify whether there is a need to request a structural assessment;
- after a structural assessment, it becomes necessary to select the repair/strengthening/replacement/posting solution that provides the best long-term dividends.

In the description that follows, the following simplified denominations will be adopted:

- maintenance/small repair subsystem → maintenance;
- inspection strategy submodule → inspection strategy;
- repair work selection submodule (from the rehabilitation/replacement subsystem) → repair.

Each part of the decision system, its scope, time of use, and decision criteria are now described.

8.2.7.1. Maintenance

Scope

This subsystem is used for every repair technique that is defined as maintenance in Table 10-3. The criterion used to separate work associated with maintenance from that associated with repair is that maintenance is strictly nonstructural (in global terms) and, therefore, does not affect the bridge's structural reliability, but does concern the bridge's functionality and the user's comfort.

Time of Use

This subsystem must always be used after a current or detailed inspection is performed. Independent of the necessity to perform a structural assessment and repair work, it is always necessary to plan and execute maintenance work for the time that ends at the next periodic inspection. Budget limitations are not very important because the amount spent each year for bridge maintenance is more or less stable, and, consequently, it is easy to predict and take into account in the general budget.

Decision Criteria

All the defects detected during the inspections are classified according to three basic parameters (see Chapter 11) (McClure and Hoffman 1990): rehabilitation urgency, importance to the structure's stability, and volume of traffic affected by the defect. The computer then prepares a list with pseudo-quantitative ratings of all the defects based on the data collected in the last inspection forms of every bridge within the network.

In theory, the defect with the highest rating indicates the first bridge on which work must be performed. All maintenance work relative to defects of the same type detected for the same bridge (even if with a lower rating) must also be performed as well as all defects that can be eliminated using the same repair technique. To do that, estimates of all the work related to the repair techniques considered within the maintenance scope must be prepared. It is also necessary to select at least one repair technique for each defect detected. The estimates are made by the inspector at headquarters, taking into account the data collected during inspection, which are introduced under the heading "Maintenance Work Needed" in the database inspection files. Initially, the inspector, with the help of the respective correlation matrix (which lists all the recommended repair techniques to eliminate the defect), establishes the correlation between defects and repair techniques. At a more advanced stage, numerical criteria for the selection of the repair technique will be established as a function of the characteristics of the defect detected (using type 4 parameters defined in Chapter 11).

The global costs relative to the work referred to previously are determined according to the units cost of each repair technique, which is included on their form (Chapter 10). These costs are then deducted from the global budget available for maintenance. The process is repeated for the bridge in which the defect with the second highest rating was detected, and so on.

In practice, this type of decision may not result from technical criteria. Even when this is so, it is always useful to consult the rating obtained with this subsystem before making any decisions.

The input, output, and detailed flowchart of this decision subsystem are presented in Chapter 11 (Figure 11-1).

8.2.7.2. Inspection Strategy

Scope

As discussed previously, this submodule concerns the decision of whether to promote the performance of a structural assessment before the next periodic inspection. The main characteristics that distinguish a structural assessment from a periodic inspection are:

- much higher costs;
- probable necessity of interrupting traffic at least partially and for a limited period;
- specialized equipment and personnel;
- strictly structural nature of the analysis;
- strong emphasis on a particular pre-determined problem of the structure leaving aside all the other defects detected;
- need of a measurements program;
- probable necessity of extracting cores and samples of the bituminous;
- very meticulous description of the problems, their causes and consequences.

Contrary to the maintenance subsystem, this submodule is directly related to structural reliability and only takes into account the bridge's functionality (or lack of it) when the cause is structural.

Time of Use

This submodule must always be used after performing a current or detailed inspection. Independent of the result, it is always necessary to know whether the bridge structural reliability is high enough to do without any repair works until the next periodic inspection. Even though a structural assessment is a costly operation, its cost is very small when compared with the repair itself and, when considered necessary, it is supposed to be implemented independent of budgetary constraints.

Decision Criteria

In Chapter 13, two proposals are presented for this decision submodule. The first and simpler one is based on the rating of the defects detected in the periodic inspections as described for the maintenance subsystem. The second one, theoretically more accurate, is based on the determination of a structural reliability index that defines the collapse probability of the bridge or of its main elements. However, it is very complex and requires many more data items, which are not always available.

The proposal associated with the defects rating involves collecting all the necessary information from the last periodic inspection form. The condition of main structural elements (deck, beams, columns, abutments, and foundations) and their defects of a strictly structural nature are evaluated. If among these there is one that, according to the rehabilitation urgency criterion, is rated 0 or 1 (action required immediately or within the next 6 months), the bridge in which the defect has been detected must be subjected to an immediate structural assessment. If the defects detected are rated no higher than 2, action will be required within the next 15 months, and a structural assessment must be promoted before the next periodic inspection.

The proposal associated to the determination of the structural reliability is based on the reliability index β, which depends on the bridge structural collapse probability P_f, which is a function of time:

$$\beta(t) = -\phi^{-1}(P_f(t)) \tag{8-1}$$

For each bridge, the most conditioned cross sections are defined along with their respective ultimate limit states with the greatest occurrence probability. The most plausible global collapse mechanisms are identified. The index β evolves in time according to the degradation mechanisms that it is necessary to implement. The parameters (type 5 from Chapter 11) that govern them must be identified and must be capable of being measured (directly or indirectly) during the periodic inspection. The index can therefore be updated based on the inspections report and, using a computer program, a prediction of its evolution with time can be made (Thoft-Christensen).

According to the decision criteria, if, according to the estimate obtained based on the last periodic inspection, the value of β goes below a certain limit β_{min} during the time until the next periodic inspection, a structural assessment must be proposed. The urgency of this assessment would also be based on the β index value: if $\beta < \beta_1$ ($\beta_1 < \beta_{min}$) during the referred period, the structural assessment must be performed immediately; if the opposite happens, the assessment must be performed only before the next periodic inspection. If, during the period mentioned, β never goes below β_{min}, no structural assessment needs to be implemented before the next periodic inspection, when the β index is updated again.

Regardless of the proposal of decision criteria, the last word belongs to the head of bridge authority and should take into account the limitations in terms of personnel and equipment as well as the bridge's location.

The input, output, and detailed flowchart of this decision submodule are presented in detail in Chapter 13 (Figures 13-1 and 13-5).

8.2.7.3. Repair

Scope

This submodule concerns all repair/rehabilitation techniques defined as structural repair in Table 10-3. The repair work is of a structural or semistructural nature and may or may not have consequences with regard to the functionality of the bridge. In a larger sense, this submodule also conditions situations in which the possibility of capacity upgrading (deck widening or structural strengthening), functional limitations (by posting), or bridge replacement is under consideration.

Time of Use

This submodule must always be used when a structural assessment is performed and its use is suppressed if, within the inspection strategy submodule, it is concluded that no structural assessment is needed until the next periodic inspection. Even when it is decided that the best solution from an economic point of view is to do nothing about the structural defects found, this decision must be made by resorting to this submodule and must be based on an economic analysis. Budget limitations are paramount in the decision-making process, since it is almost impossible to predict the number of bridges that will need to be repaired each year and the extent of work needed. A bridge may not need to be repaired for 10 or more years and, in the following year, it may be subjected to a rehabilitation program with associated costs similar to the initial costs of the bridge.

Decision Criteria

As described in Chapter 13, the decisions made in this submodule are basically the result of an economic analysis. This analysis can be made at three different levels:

- Level 1—elimination of the defect(s);
- Level 2—bridge repair;
- Level 3—bridge network management.

All the decisions are made according to the cost efficiency index (CEI) (Aylon 1990). The CEI index gives an indication of the planned action option as compared with the no action option. The bigger the index of a certain action, the bigger the dividends obtained from the investment made. In the calculation of CEI, the repair costs (C_R), the failure costs (C_F), and the benefits (B), defined in detail in Chapter 10, are considered.

$$CEI = \frac{(C_R + C_F - B)_{\text{repair}}}{(C_R + C_F - B)_{\text{no-action}}} \tag{8-2}$$

The initial costs (C_0), the inspection costs (C_I), and the maintenance costs (C_M), discussed in the same chapter, are excluded from the analysis. This is possible because these costs are not relevant to the analysis (being identical to all the options under analysis) or because they have already occurred when this decision is made.

At decision level 1, the objective is to select the best repair technique to eliminate a specific type of structural defect from the options presented in the "Repair Work Needed" database file of the last inspection. Associated with this technique are its cost estimate and service life. This period can be determined using deterioration models that take into account local aggressiveness. If the mathematical models are not reliable enough or if the necessary data to implement them are not available, tables that provide a statistical average of service life for different materials, elements, and repair techniques can be used.

After this type of analysis is applied to every defect detected in a certain bridge, decision level 2 begins. A list of the structural defect types detected with their respective optimal repair techniques and values CEI_{max} is available. Due to budget limitations, not every defect can be repaired. The type of defect with the highest CEI_{max} value (CEI_1) is the first to be

Table 8-5. Scope of the decision modules

Module		Scope
Maintenance		Selection of the repair techniques classified as maintenance work
Inspection	A	Decision about the performance of a structural assessment before the next periodic inspection
Strategy	B	
Repair (work selection)		Selection of the repair techniques classified as (structural) repair work

Table 8-6. Time of use of the decision modules

Module		Time of use
Maintenance		Always after a current or detailed inspection is performed
Inspection	A	Always after a current or detailed inspection is performed
Strategy	B	
Repair (work selection)		Only after a structural assessment is performed

repaired and so on. Costs C_1, C_2, and so on are deducted from the available budget for the bridge under analysis until all of it is spent. If there are no individual budgets for each bridge, the decision as to whether to repair each type of defect must be made at level 3.

Level 3 manages the global bridge network budget available. It is at this level that the options of capacity upgrading and replacement of each bridge are analyzed. Level 2 analysis of all bridges within the network provides new lists in which, for each bridge, the defects are grouped and their respective accumulated costs C and indexes CEI are determined.

$$ACEI_i = \frac{\sum_{j=1}^{i} C_j CEI_j}{\sum_{j=1}^{i} C_j} \tag{8-3}$$

$$AC_i = \sum_{j=1}^{i} C_j \tag{8-4}$$

The value of $ACEI_i$ represents the cost efficiency index of performing all the repair work necessary to eliminate defect types 1 to i, and the value AC_i represents the respective cost. As the number of defect types considered increases, the $ACEI$ value decreases since the individual CEI values also decrease progressively. The aggregate of repair work with the highest $ACEI$ index value is the first to be performed and indicates the first bridge to be repaired. The accumulated cost of this repair work is deducted from the global budget available and the process continues with the second highest ACEI. Whenever a repair work aggregate for a certain bridge that contains n techniques is included in the list, the repair work aggregate

Table 8-7. Decision criteria of the decision modules

Module		Decision criteria
Maintenance		Human factor; economic analysis
Inspection	A	Human factor; structural reliability
Strategy	B	Structural reliability; human factor
Repair (work selection)		Economic analysis; human factor; structural reliability

in the same bridge containing n–1 techniques is eliminated from expenditures and the available budget value is corrected.

The description of the decision levels has been made for the situations in which the options are limited to repair (again achieving the initial after construction situation) or no action. The less common cases of capacity upgrading and replacement are particular situations that are described in Chapter 13. In the same chapter, the input, output and detailed flowchart of this decision submodule are also presented.

8.2.7.4. Summary

A summary of the several decision modules within DMM is presented in Tables 8-5 to 8-7 (de Brito 1992). For the strategy inspection submodule, two proposals are put forward in Chapter 13: one based on the defects rating (designated by A) and the other based on the determination of the structural reliability evolution with time (designated by B).

As a final note, the system proposed is of the fourth and most complex type of bridge management system according to the classification proposed by Vassie (1996): those with an inventory database, basic inspection scheduling, and recording and maintenance scheduling taking into account the rate of deterioration, minimizing life costs and prioritizing where the budget is restrained.

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PART 4

Organization of a Bridge Management System

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CHAPTER 9

The Database

9.1. Introduction

The efficiency of a bridge management system is dependent to a great extent on its data storage and post-treatment data. In fact, the system uses a huge volume of information simultaneously with the acquisition of data about the inspection, normal use, maintenance, and repair of each bridge. Before computers, the support basis for all this material was paper in the shape of dossiers, reports, forms, manuals, drawings, photos, schemes, and so forth. This made accessing the information difficult and made it difficult to store all this bulky material in accessible places. The computer revolution and the advent of database software allowed for the storage of great quantities of information on relatively small objects (magnetic disks).

This evolution did not eliminate the necessity for continuing to use traditional information storage environments. To start with, access to digitalized data demands equipment that is not always available (at the bridge site, the best one can hope for is to resort to a personal computer of limited capacity, i.e., a PC). Some specific information (e.g., drawings) is easier to access when it is on paper, mostly because of the limited dimensions of the computer screen. The storage capacity of existing magnetic disks and CD's has greatly increased, but there is still some difficulty in storing graphic information due to the huge size of the corresponding files. There are also some hazards associated with the use of data in a computer (software and hardware bugs, mechanical damage, and sensitivity to temperature and magnetic fields) that make it more vulnerable than the data registered on paper. Finally, it is not economically feasible to systematically resort to computers to store the endless amount of information collected during the day-to-day management of each bridge (particularly at the construction stage), especially because all of this data must be manually inserted into the right files in a very time-consuming process.

Therefore, we discuss the simultaneous use of a digital database and a traditional means for the collection and storage of information, which will be assembled in the so-called "bridge dossier," which is described in greater detail in Chapter 8. The bridge dossier will put together all the information concerning the bridge, with a low degree of selectivity, merely organizing it to facilitate access. The database contains only selected information, synthesized to an indispensable minimum and organized as files that are easy to read on the computer screen and can also be printed on paper.

The main characteristics desired from such a reference database are found in de Brito and Branco (1991) and de Brito (1992):

- user-friendly in terms of both reading existing data and storing new data (most of the time, the person actually feeding the data into the computer does not have any knowledge of bridge designing or building);
- easy access (operations involving the introduction, modification, or reading of information from the database must be quick and simple);
- thoroughness without complexity (all information likely to be used in the future must be stored but over-specialized data should be avoided because of inspector time limitations);
- capacity to create specific reports adapted to the user's needs through comprehensive menus;
- possibility of easily transferring part of the information to portable microcomputers capable of being used at the bridge site;
- availability of simplified executable versions capable of being installed in the same type of equipment;
- clear but economical internal organization (the data must be separated into blocks that constitute by themselves complete and independent pieces of information);
- capacity to perform its own maintenance (backups generation, passwords protection, protection against misuse or users mistakes, etc.);
- adaptability to the system requirements both at the bridge site and at headquarters.

9.2. General Organization

9.2.1. International Experience

In the bridge management systems presently being used or implemented, the database is, save for a few exceptions, one of the most important modules of the system. This is the situation in Austria (Straninger and Wicke 1993); Canada (Reel et al. 1988); Colombia, Croatia, Denmark, Honduras, Malaysia, Mexico, and Saudi Arabia (Lauridsen and Lassen 1999); Finland (Söderqvist 1999); Germany (Krieger and Haardt 2000); Hong Kong and Sri Lanka (Blakelock 1993); India (Cox and Matthews 2000); Italy (Camomilla and Romagnolo 1999); Japan (Yokoyama et al. 1996); the Netherlands (El Marasy 1990); Poland (Legosz and Wysokowski 1993); Portugal (Santiago 2000); South Africa and Taiwan (Nordengen et al. 2000); Sweden (Lindbladh 1990); Switzerland (Grob 1989); Thailand (Sørensen and Clausen 1989); the United Kingdom (Hayter and Allison 1999); and the United States (Thompson 1993). There are, of course, a number of differences between the several databases according to the specific needs of each system.

The United States National Bridge Inventory (NBI) registers the data recorded during the inspection of public bridges in accordance with the Federal Aid Highway Act of 1968, and is used to determine a sufficiency rating that allows or disallows federal funding for bridge rehabilitation and replacement (according to the Federal Coding Guide, Report no. FHWA-PD-96-001 "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges"), and deserves some special attention. Since the early 1980s, the biennial inspection program has been fully implemented, and all U.S. transportation agencies are required to supply detailed inspection information to the Federal Highway Administration (FHWA).

According to McClure (2002), 614,083 bridges are covered by the inventory whose results have been analyzed and reveal that 87,801 of the bridges are structurally deficient and 79,860 are functionally deficient (an additional 20,517 are scour critical according to federal guidelines). Based on analysis, it can be inferred that even in the most developed country of the world, with an efficient database and working BMS systems in all its states, funding below the critical level can lead to a huge backlog (the total improvement cost is more than $210 billion).

The structure of some of the databases/systems mentioned above is presented here in terms of simplified diagrams:

- the Austrian Road and Traffic Research Association database (Figure 9-1) (Straninger and Wicke 1993);

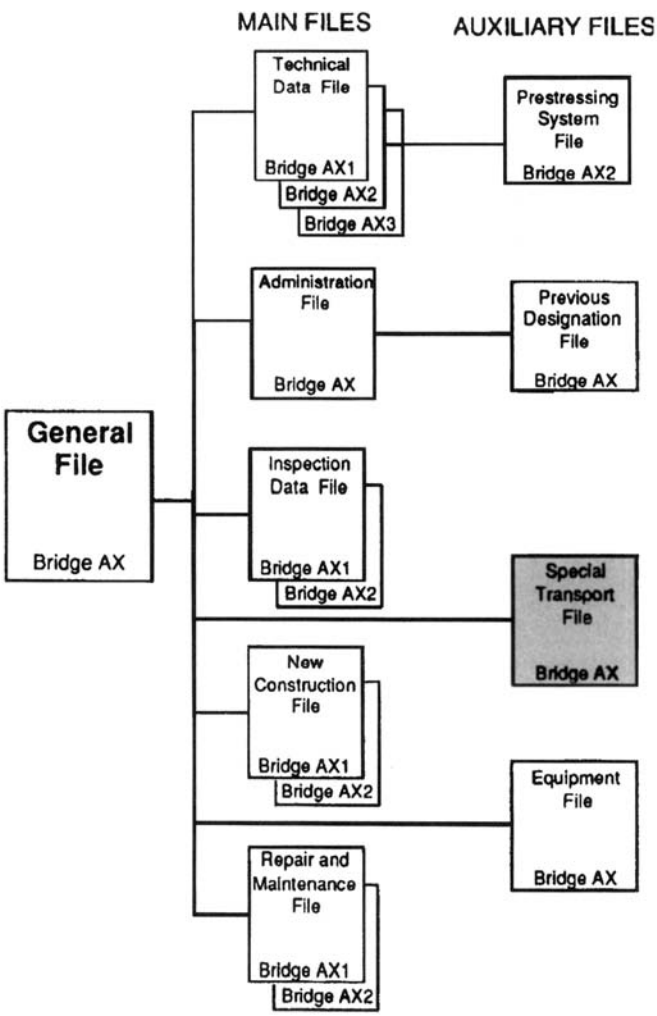

Figure 9-1. Subfile structure of the Austrian bridge database

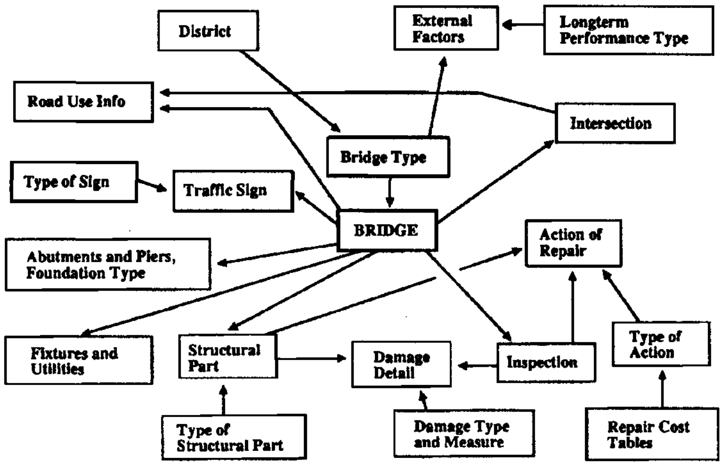

Figure 9-2. Entity attribute relationship of the Finnish bridge directory

- the Railways of Denmark bridge management and maintenance system (DANBRO) (Figure 7-16);
- the Finnish Road and Waterways Administration bridge directory (Figure 9-2) (Kähkönen and Marshall 1990);
- the Administration of the Province of Perugia (Italy) information system (Figure 9-3) (Gusella et al. 1996);

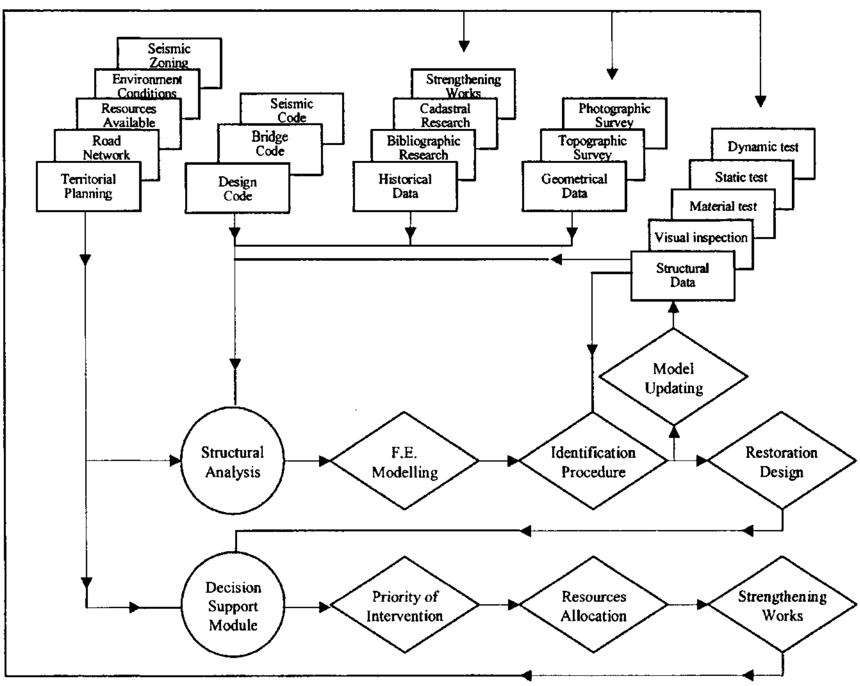

Figure 9-3. Block diagram of the Perugia information system

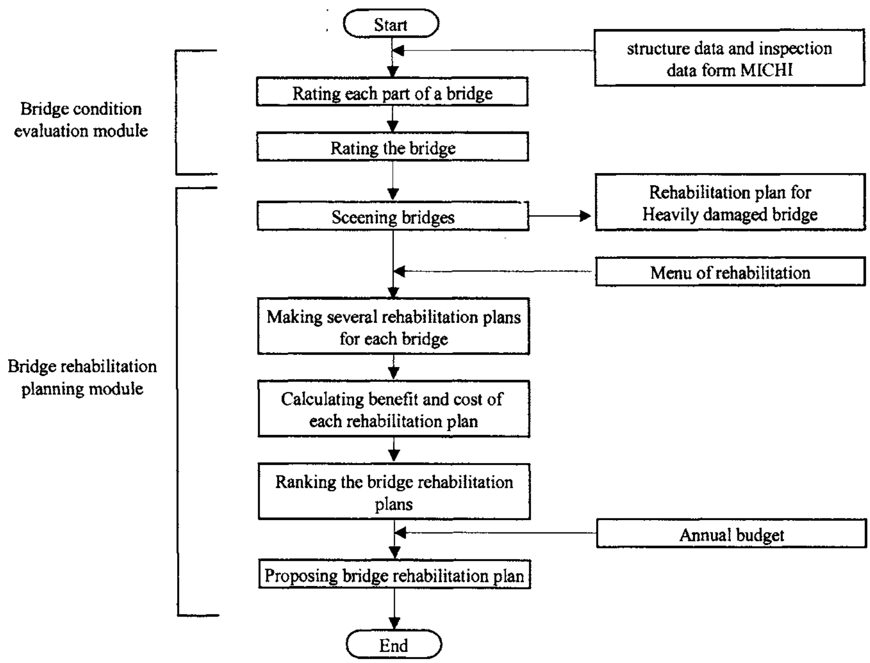

Figure 9-4. Japanese Public Works Research Institute bridge management system

- the Japanese Public Works Research Institute bridge management system (Figure 9-4) (Yokoyama et al. 1996);
- the Polish bridge management system (Figure 9-5) (Legosz and Wysokowski 1993);
- the United Kingdom Highways Agency Structures Management Information System (SMIS) (Figure 9-6) (Hayter and Allison 1999).

Generally, the aforementioned databases are organized into the following modules:

- bridge reference module in which all the information that undergoes very few changes throughout lifetime of the structure is stored;
- system reference module that contains all the main information catalogues used by the system during inspection, decision-making, and repair work bidding;
- inspection and maintenance/rehabilitation support module, of a fundamentally variable nature, in which all the relevant information collected during inspection and after the maintenance/rehabilitation work is stored.

9.2.2. A Reference Database

Based on a synthesis of the different databases analyzed, the following database is proposed, in which the information stored is classified according to its variation with time (El Marasy 1990):

- static information—reference files/forms that, after being created, do not require any changes;

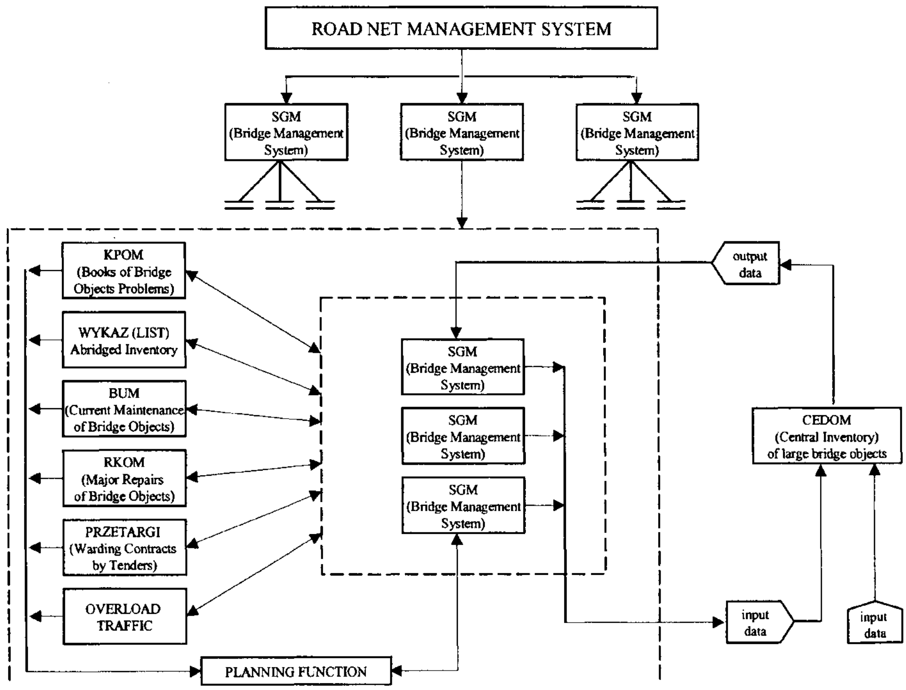

Figure 9-5. Operational diagram of the bridge management system in Poland

- semi-static information—files/forms concerning the bridge that, under normal circumstances, are not altered or concerning costs that are added/updated as they change;
- upgradeable information—files/forms in permanent evolution throughout the service life of the bridge.

9.2.2.1. Static Information

The static information comprises the following data (de Brito 1992):

- classification system (Chapter 10);
  - a list of all the defects liable to be detected in concrete (or other material) bridges;
  - a list of all the possible causes (direct or indirect) of the defects referred to above;
  - a list of all the repair techniques (maintenance and structural repair) for the defects referred to above;
  - a list of all the diagnostic methods that can be employed in the detection and analysis of defects during the bridge's inspections;
  - a list of all the criteria used to rate the diagnostic methods with the highest potential;
  - a rating of the diagnostic methods and its prioritization according to the criteria mentioned;

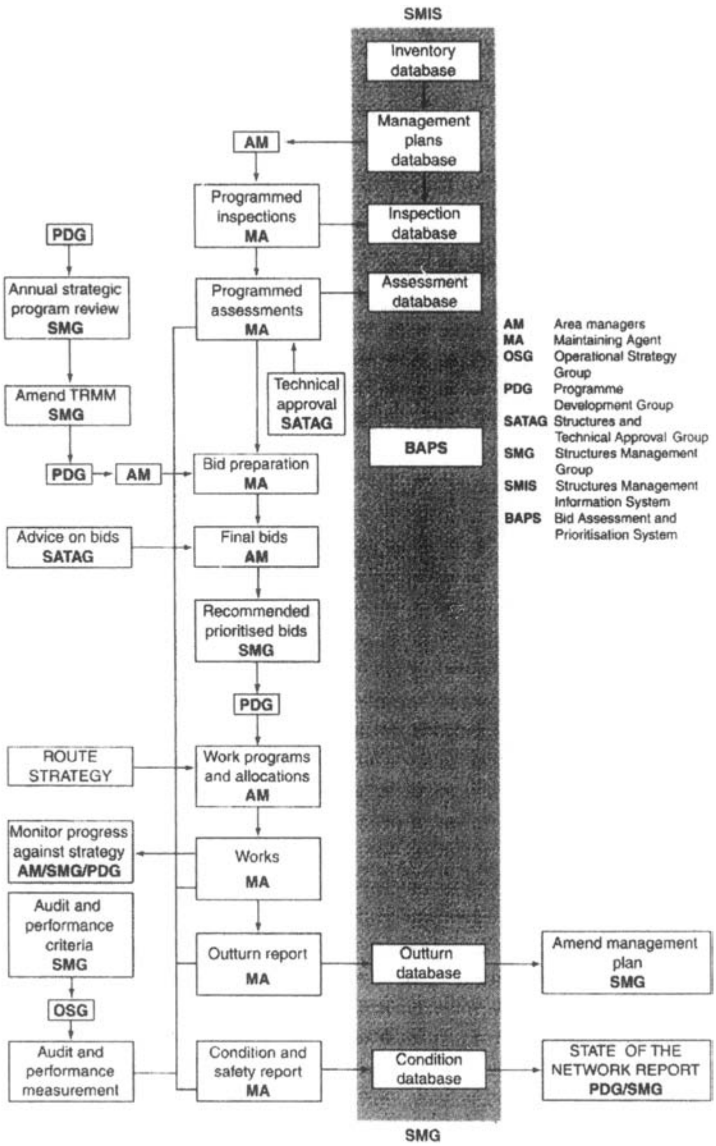

Figure 9-6. Interaction between the Structures Management Information System (SMIS) and structures management operational procedures within the United Kingdom Highways Agency

- correlation matrices (Chapter 10);
  - correlation matrix of probable cause versus defect;
  - correlation matrix of repair technique versus defect;
  - correlation matrix of diagnostic method versus defect;
- defect forms (Figure 8-9), according to the list referred above and including one photo/drawing, the defect type, the form number, the defect designation and description, its possible causes and consequences, further inspection factors, the inspection parameters, and the defect rating;

- repair forms (Chapter 10), according to the list referred to above and including the repair type, the form number, the repair designation, its field of application, the material characteristics, the description of the technique, the personnel and equipment required, the estimated mechanical efficiency, special problems, unit cost estimates, schematics/drawings, and secondary elementary forms;

- inspection manuals (Chapter 10), customized for each type of structure, in which the system inspection types, the terminology used, the symptomatology detected, the most common diagnostic methods, and all the information capable of aiding the inspector in its task, are defined;

- identification forms, described in detail in Section 9.4;

- graphic information about each bridge, also described in detail in Section 9.5;

- defect rating (Chapter 11);
  - self-explanatory list of the criteria used to pseudo-quantitatively rate the defects detected during inspection;
  - rating attributed for each category in the various criteria;
  - procedure to be followed in terms of the defect rating;
  - procedure to be followed in the choice of inspection strategy in terms of the defect rating;

- costs quantification (Chapter 12);
  - self-explanatory list of the various costs of design, construction, and normal use of the bridge throughout its service life;
  - description of how each item can be quantified and predicted in the medium term to the long-term;
  - basic notions about present value costs analysis;
  - decision criteria about the various options available for bridge rehabilitation/replacement depending on the economic analysis;

- structural reliability analysis backing files (Chapter 13);
  - definition of the most conditioned cross sections of each bridge, the ultimate limit states considered, and the plausible failure mechanisms;
  - definition of the parameters considered for each failure mechanism, its probabilistic modeling, the sensitivity of the model to each parameter, and the value updating procedure (common to all bridges);
  - definition of the data needed for the structural analysis (common to all bridges);
  - definition of how the selected parameters affect the changes in the reliability index with time (common to all bridges);

- computer programs;
  - program (COSTS) of quantification and prediction of long-term costs (Chapter 12);
  - programs to implement the decision system within the maintenance/small repair subsystem (Chapter 11) and the rehabilitation/replacement subsystem (Chapter 13);
  - programs to calculate, update, and predict the reliability index changes with time (Chapter 13);

- reliable mathematical models of deterioration mechanisms or, alternatively, simplified tables that predict whole or residual service lives for each repair technique and for the do-nothing option (Chapter 13);

- load-bearing definition files with a description of the most common vehicles, which may be based on codes, as well as special trucks capable of passing over the bridges of the network; identification of horizontal and vertical under-clearances and over-clearances;

- administrative data (bridge authority; departments responsible for design, inspection, maintenance, and management; identification and location of the bridge dossiers; roadways and railways over-passed and under-passed; water course crossed, etc.).

9.2.2.2. Semi-Static Information

The semi-static information consists of the following data (de Brito 1992):

- costs files of a general nature (Chapter 12), with a national scope or at least valid for all the bridges within the network (updated and enlarged every year with the information related to the previous year);
  - discount rates;
  - inflation rates;
  - construction costs;
  - inspection costs;
  - maintenance costs;
  - failure costs;

- individual cost files for each bridge (Chapter 12), concerning the existence of a file with a predetermined type of information for each bridge (also updated and expanded every year);
  - initial costs;
  - maintenance costs;
  - repair costs;
  - failure costs (this file also includes information about the registered traffic traveling over the bridge);

- annual budgets for maintenance and rehabilitation/replacement with an indication of the resources actually spent;
- load-bearing capacity of each bridge for each year (defined in terms of maximum axle load, a certain code vehicle or the actions code used);
- the reference state form for each bridge, described in detail in Section 9.6; under normal circumstances, the information contained in it is static; the exceptions to this rule are cited in Chapter 10.

9.2.2.3. Updateable Information

The updateable information consists of the following data (de Brito 1992):

- the inspection forms that will be described in detail in Section 9.7 and take into account all the information collected at the bridge site;

- rating of all the defects detected in both current and detailed inspections (Chapter 10);
- maintenance work needed according to the inspector, with estimates of the amount of work required (Chapter 11);
- results from the measurements of all parameters that may influence the determination of the reliability index at the most conditioned cross-sections;
- identification, severity, and extension of all structural defects detected during the structural assessment (Chapter 10);
- repair work needed according to the inspector with relatively precise estimates of the amount of work required (Chapter 13).

The information included in this database is shown in Figure 9-7.

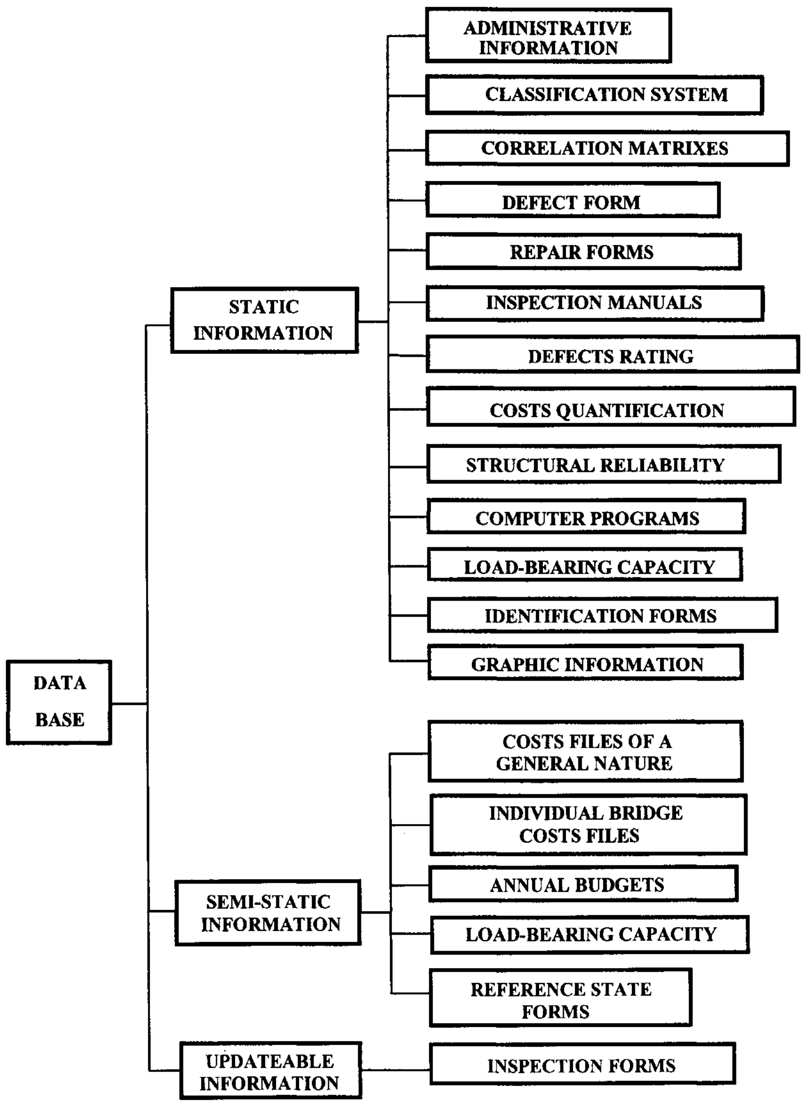

Figure 9-7. Information contained within the database

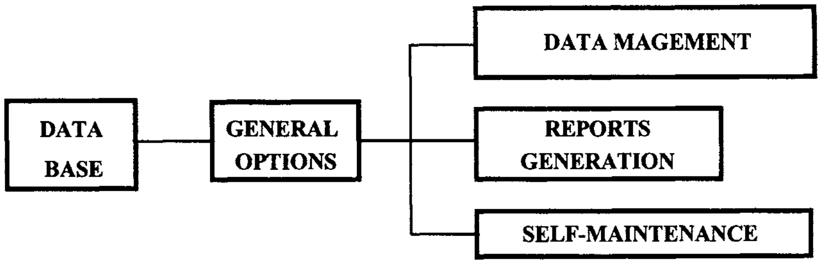

Figure 9-8. General options of the database

The majority of the preceding data are stored in the shape of sequential or text files that can be written on, read, added to, or printed without needing specific database software. Special attention should be paid to the elaboration of the identification, reference state, inspection forms, and the graphic information because they are of the most interest to the inspector (de Brito and Branco 1991).

9.3. Utilization Options

The database must consider the following general options (Reel et al. 1988) (Figure 9-8):

- data management;
- reports generation;
- self-maintenance.

The first option makes it possible to add, read, alter, or print the information contained in the database. This option is detailed later in this chapter.

The reports generation option allows the preparation of reports in which the user includes the information he needs for further development of his work and eliminates superfluous information. This option is detailed later in this chapter.

The self-maintenance database option (Reel et al. 1988) permits the recovery of lost data, backup generation to the hard disk or drive, the combination of data from different files, the deletion of screens, and so forth.

The general option of information management provides access to four types of information modules (Figure 9-9): (1) identification form, (2) graphic information, (3) reference state form, and (4) inspection forms.

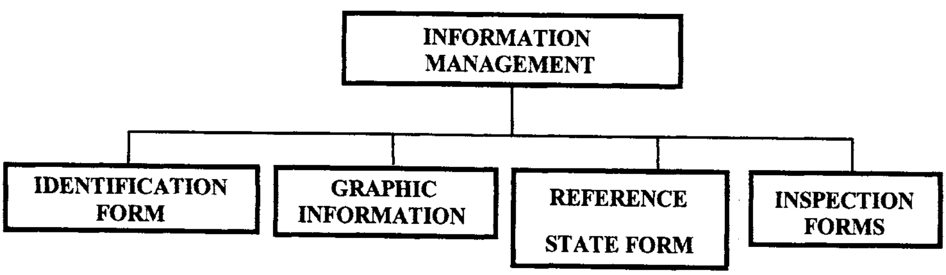

Figure 9-9. Information modules included in the information management general option of the database

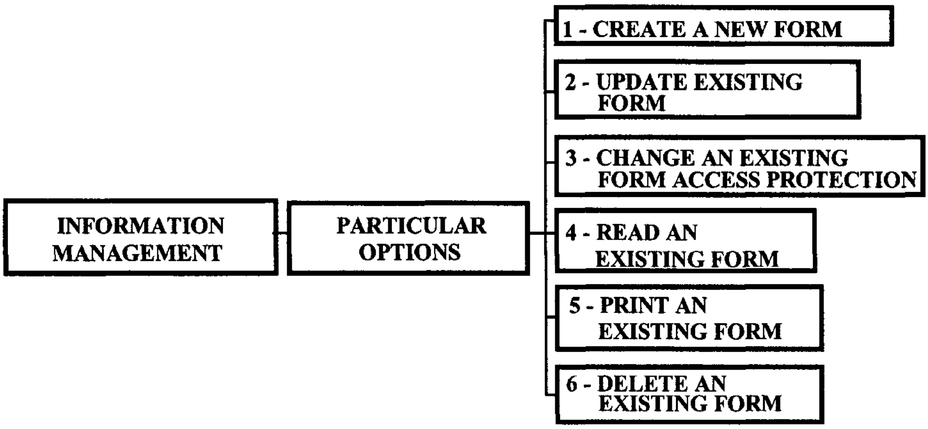

Figure 9-10. Particular options of managing the information contained in the standard forms

Within each form, the user can select a particular option from a set of predetermined options (Figure 9-10). These include (Reel et al. 1988) and (de Brito and Branco 1991):

1—create a new form;

2—update an existing form;

3—change the access protection of an existing form;

4—read an existing form;

5—print an existing form;

6—delete an existing form.

Option 1 is used when the form in question does not yet exist in the database. The system checks to make certain that such is the case in order to prevent the loss of existing information due to the overlapping of a new form. The user must then fill in all empty fields for which information is available, none of which is protected against alterations.

Option 2 allows alteration of the form as long as it has already been filled in, something that the system checks. Some of the fields on the form are protected but the user can alter others. It may happen that the form is protected, in which case it will be necessary to remove the protection, as shown in option 3, before making any alterations.

Option 3 permits changing the form access protection from protected to unprotected and vice versa. The only protection considered is against changing the contents of fields and deleting the form. Every form can be read if option 4 is chosen.

Option 4 permits reading the form in question as long as it has already been filled in, which is checked by the system. It is not possible to change any field because they are all protected against writing.

Option 5 complements the general option of reports generation by allowing the form in question to be printed in its entirety without any selection criterion. The form may be used as a backup document during inspection.

Option 6 permits the elimination of the form in question. If the file is protected against deletion, first it will be necessary to unprotect it by resorting to option 3. It is advisable that access to options 2 and 6 be protected by a password to guarantee the form's reliability.

The four modules of information that constitute the database general option of information management are described next, both individually and in detail.

9.4. Identification Form

The identification form for each bridge is the first form to be filled in the database. It is basically an "identification card" of each bridge and contains a range of important albeit synthesized information that allows unequivocal identification of the bridge.

The identification form is divided in three information submodules (Figure 9-11) (de Brito and Branco 1991):

- bridge site (Figure 9-12);
- design general information (Figure 9-13);
- construction general information (Figure 9-14).

In addition to this information, there are two more screens at the end of the identification form. In the first one, named "Form History," the user who created the form and the date on which it was created are identified. Changes to the form's content (using option 3 defined previously) are also documented with the same information. The screen "Form Protection" allows turning the form protection against changes and deletions on or off.

The first piece of data inserted in the identification form is the code number that identifies the bridge. It can be chosen from a menu that is automatically shown on the screen with the code numbers of every bridge part in the management system (or the district where the user is presently). The code number must be standardized (e.g., two letters connected by a hyphen to a four-digit number). To each code number there is a corresponding bridge name. This should not be overly long and should unequivocally identify the bridge in question.

The user is then able to access any of the three information submodules mentioned previously (Figure 9-11) or to return to the general menu.

9.4.1. Bridge Site

The site of the bridge is identified by the name of the spot, the parish, the council, and the district (Figure 9-12). The ways involved are also identified by name and reference kilometer according to the bridge authority. The names must, as much as possible, be stan-

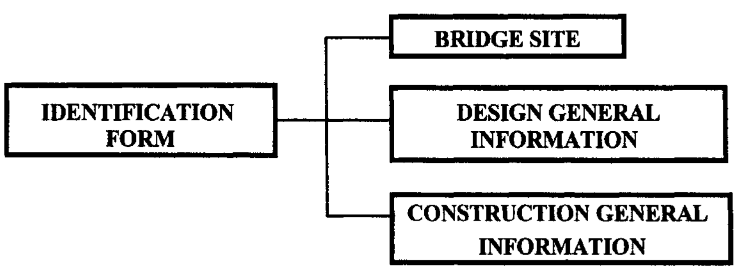

Figure 9-11. Information contained within the identification form

IDENTIFICATION FORM

CODE NUMBER: __________

BRIDGE SITE

1. SITE

1.1. SPOT: ____________________ ; 1.2. PARISH: ______________

1.3. COUNCIL: ______________ ; 1.4. DISTRICT: ______________

2. WAYS

2.1. UPPER WAY: __________ ; 2.2. KM: _____

2.3. LOWER WAY / OBSTACLE: __________ ; 2.4. KM: ______

INTERSECTION COORDINATES: 2.5. M: ______ km; 2.6.P: ______ km

INTERSECTION LEVEL (WEARING SURFACE):

2.7. UPPER WAY: _______ m; 2.8. LOWER WAY: ______ m

2.9. STRETCH: ____

Figure 9-12. Bridge site screen from the bridge identification file

dardized in the shape of code number or abbreviations. In the event that the bridge crosses a watercourse or other natural obstacle, this must also be identified. The coordinates and level (of the wearing surface) at the intersection point are also included in the form, as well as the identification of the road stretch where the bridge is inserted.

As happens with all the other information submodules that comprise the proposed database, this submodule can undergo additions, limitations, or corrections according to the specific needs of the bridge management authorities and the experience that is gained from implementing the management system.

9.4.2. Design General Information

This submodule contains the most important information concerning bridge (structural) design. The chief designer, the year of conclusion of the project, and the total estimated cost of the bridge and accessory structures are identified (Figure 9-13). The design record is defined by its title and the reference to the site at which the several volumes that contain the calculations, as well as the reference number, the title, and the location of the site where the original drawings have been deposited. If drawings have been microfilmed, their reference number is also given. The reference numbers of both the drawings and the microfilms must be standardized in a manner similar to the way code numbers were assigned to the bridges.

It must be stated that the drawings identified here were generated before construction and therefore are not as-built drawings.

IDENTIFICATION FORM

CODE NUMBER: __________

DESIGN GENERAL INFORMATION

3. STRUCTURAL FINAL DESIGN YEAR: _____

4. STRUCTURE DESIGNER: ______________________________

5. COST ESTIMATE: ______________ $

6. STRUCTURAL DESIGN RECORDS

6.1.WRITTEN RECORDS

6.1.1. VOLUME NUMBER ___

6.1.1.1. TITLE: ______________________________

6.1.1.2. SITE REFERENCE: ______________________________

6.1.2. ....

....

....

6.2.DRAWINGS

6.2.1. DRAWING NUMBER ___

6.2.1.1. REFERENCE NUMBER: __________

6.2.1.2. TITLE: ______________________________

6.2.1.3. SITE REFERENCE: ______________________________

6.2.1.4. MICRO-FILM REFERENCE: ______________________________

6.2.2. ....

....

....

....

....

Figure 9-13. Design general information screens from the bridge identification file

9.4.3. *Construction General Information*

In this submodule a description of the bridge construction stage is given. The starting year and final year are identified (Figure 9-14). The builder, the technical director, the surveyor, and the bridge owner are identified, as well as the type of traffic expected to use the bridge (road, railway, or pedestrian). The most important construction costs are described: the global cost shown in the contract and the costs at deliverance, the cost of the stretch, the

IDENTIFICATION FORM

CODE NUMBER: ___________

CONSTRUCTION GENERAL INFORMATION

7. STARTING YEAR: _____ ; 8. FINAL YEAR: _____

9. CONSTRUCTION

9.1. BUILDER: ______________________________

9.2. TECHNICAL DIRECTOR: ______________________________

9.3. SURVEYOR: ______________________________

9.4. BRIDGE OWNER: ______________________________

9.5. TYPE OF TRAFFIC: ____________________

10. COSTS

10.1. GLOBAL COST IN THE CONTRACT: ______________ $

10.2. TOTAL COSTS OF BUILDING THE STRETCH: ______________ $

10.3. % OF THE BRIDGE IN THE STRETCH: ______________ $

10.4. BRIDGE REAL COST: ______________ $

10.5. DECK AREA: ________ m^2; 10.6.COST / m^2: __________ $

11. AS-BUILT DRAWINGS

11.1. ARE THERE CHANGES FROM THE INITIAL DESIGN PLANS? ___

11.2. ARE THE AS-BUILT DRAWINGS IN RECORD? ___

Figure 9-14. Construction general information screen from the bridge identification file

percentage of the bridge cost in the stretch, and the cost per square meter of the deck. Also shown are changes from the initial design plans and the corresponding as-built drawings.

The data that are inserted in each writing field must be inasmuch as possible, subject to simple automatic tests that prevent storage of erroneous or senseless information. The fields that can be filled in automatically (based on the fields already filled in, decrease the data insertion time.

9.5. Graphic Information

This form contains all the available relevant graphic information concerning the bridge. The database does not need to be coupled with CAD software but must be capable of storing photos or digitized drawings imported from a scanner or drawings made using CAD (which is especially useful because it allows the use of reference grids such as those described in Chapter 10). Complementarily, the identification form stores the location of the drawings records and their microfilms, if they exist.

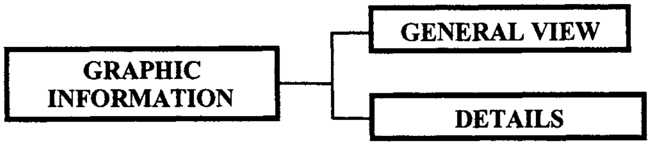

Figure 9-15. Information contained within the graphic information form

The form is divided into two information submodules (Figure 9-15) (de Brito and Branco 1991):

- general view;
- details.

In addition to this information, there are two screens at the end of the form, "Form History" and "Form Protection," which have functions identical to those described in the identification form.

Drawings and reference grids stored in this form can be printed on paper and taken to the bridge site in order to record the defects detected. If, in parallel with the database, CAD software is available, this type of information can be inserted digitally into the drawings. These drawings would then be stored in the form, thereby yielding a "graphic history" of the inspections and a much better visualization of the evolution of defects. The defects detected during the various inspections can then be inserted into a single drawing, thus producing a document that is very useful to the inspector (Figure 9-16).

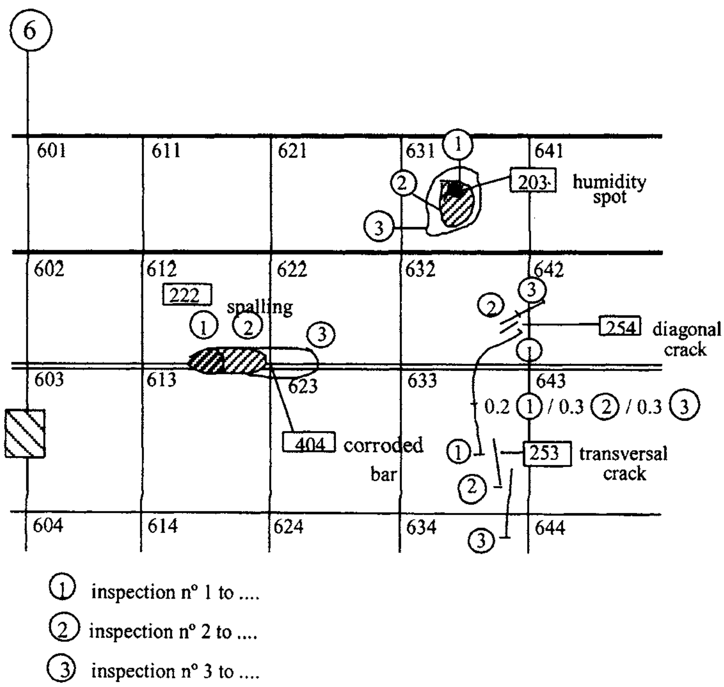

Figure 9-16. Graphic representation of the defects evolution

Another interesting future application of this form is proposed in the Danish bridge management and maintenance system (Sørensen and Berthelsen 1990), described in Chapter 7. It would allow for the possibility that the database will contain digitalized maps of the whole area managed by the management system at various scales. In the smaller scales, the user is able to choose the sector in which he is interested, which is automatically shown on the computer screen at a larger scale (Figure 7-17). By zooming in several times, the user obtains an on-screen map to an appropriate scale that contains the road or the crossroads that are included with the bridge in question, and all of the adjoining bridges are represented graphically. By using the "mouse," the user can select the bridge he wants. The bridge's graphic scheme is then shown on the screen (Figure 7-17). The whole process makes it appear that the user has a lens that he can progressively zoom "down" until he sees exactly what he wants in great detail. It is possible to implement this process without having to use a database with CAD software included.

The first piece of data inserted in the graphic information form is the bridge code number. The name of the bridge is inserted automatically (the system reads it from the identification form that is supposed to be filled in beforehand). The user then has the option of accessing either of the two information submodules mentioned previously (Figure 9-15) or returning to the general menu.

In the first submodule, the plans, views, and cutaways (in photos or drawings) for all bridges are inserted. The larger scales schematics and photos related to specific predetermined areas of the bridge are stored in the second submodule. The reference grids are loaded into the more appropriate of the two submodules. Alternatively, an independent submodule can be created solely for the reference grids and another for the schematic drawings containing the information collected during the inspections concerning the defects detected.

The graphic information stored in this form concerns the structure actually built (based on the as-built drawings) and not the initial design structure, subject to changes during construction.

9.6. Reference State Form

The reference state form contains all the information necessary to describe the bridge in detail both at the design stage and after its construction. At relevant points of bridge service life (generally immediately after construction ends or after the execution of significant rehabilitation work), the real situation must be characterized so that the inspections have something to refer to. This situation is designated by "reference state" and is described in detail in Chapter 10.

The reference state form is divided into two information submodules (Figure 9-17) (de Brito and Branco 1991):

- design stage (preliminary studies, characteristics of the ways to the bridge, structural description, and design traffic);
- post-construction stage (changes from the design stage, tests performed during and at the completion of construction and traffic on the bridge).

At the end of this form, there are two screens, "Form History" and "Form Protection," which have the same functions as those described for the identification form. In addition to the information described, the form includes reasons for alteration of the form, since only

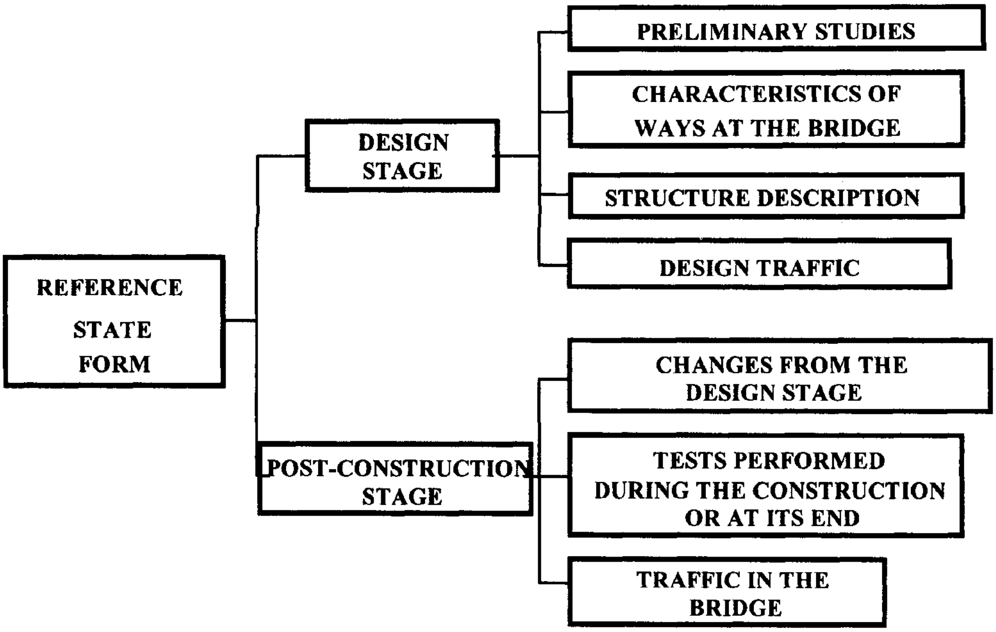

Figure 9-17. Information contained within the reference state form

under very special circumstances (see Chapter 10) is the reference state changed. Some general information about the valid reference state is also included, such as identification and role of all the members of the inspection team and equipment used.

It is inevitable that some of the information contained in the reference state form is a repetition of the information contained in the identification form, since it was designed as a kind of résumé of the first one.

The first piece of data inserted in the reference state information form is the bridge code number. The name of the bridge is then inserted automatically (the system reads it from the identification form that is supposed to be filled in before). The user then has the option of accessing either of the two information submodules mentioned previously (Figure 9-17) or returning to the general menu.

9.6.1. Design Stage

In this submodule, all the information necessary to describe the bridge reference state in the theoretical hypothesis that there are no changes from the design during construction and that the actual traffic on the bridge corresponds to the prediction made in the corresponding preliminary study. As mentioned previously, the information is divided into four sections (Figure 9-17) (de Brito and Branco 1991):

- preliminary studies (Figure 9-18);
- characteristics of the ways at the bridge (Figure 9-19);
- structure description (Figure 9-21);
- design traffic (Figure 9-22).

REFERENCE STATE FORM

CODE NUMBER: __________

PRELIMINARY STUDIES

1. TRAFFIC

1.1. PERSON IN CHARGE: ____________________ ; 1.2. YEAR: ______

1.3. MAIN CONCLUSIONS: ____

____

2. LOCATION

2.1. PERSON IN CHARGE: ____________________ ; 2.2. YEAR: ______

2.3. MAIN CONCLUSIONS: ____

____

3. HYDRAULICS

3.1. PERSON IN CHARGE: ____________________ ; 3.2. YEAR: ______

3.3. MAIN CONCLUSIONS: ____

____

4. ENVIRONMENTAL IMPACT

4.1. PERSON IN CHARGE: ____________________ : 4.2. YEAR: ______

4.3. MAIN CONCLUSIONS: ____

____

5. ECONOMIC FEASIBILITY

5.1. PERSON IN CHARGE: ____________________ ; 5.2. YEAR: ______

5.3. MAIN CONCLUSIONS: ____

____

6. STRUCTURAL PRELIMINARY STUDY

6.1. PERSON IN CHARGE: ____________________ ; 6.2. YEAR: ______

6.3. MAIN CONCLUSIONS: ____

____

6.4. CONSTRUCTION ESTIMATED COST: ______________________ $

Figure 9-18. Preliminary studies at the design stage screens from the reference state form

REFERENCE STATE FORM

CODE NUMBER: ____________

CHARACTERISTICS OF THE WAYS AT THE BRIDGE

7. UPPER WAY

7.1. TRANSVERSE SECTION

7.2. LONGITUDINAL SECTION

7.3. AXIS

7.4. WEARING SURFACE ____________________ ; 7.5. SKEW ______ gr

8. LOWER WAY

8.1. TRANSVERSE SECTION

8.2. LONGITUDINAL SECTION

8.3. AXIS

9. EMBANKMENTS

9.1. BACK FILLING TOTAL LENGTH: ______________ m

9.2. SLOPE: ________

9.3. EXCAVATION TOTAL LENGTH: ______________ m

9.4. SLOPE: ________

9.5. MATERIALS USED: ____________________________________

9.6. HAVE THE EMBANKMENTS BEEN STABILIZED? ___

9.7. RETAINING WALLS TOTAL LENGTH: ________ m

Figure 9-19. Characteristics of the ways at the bridge at the design stage screens from the reference state form

9.6.1.1. Preliminary Studies

The first section lists the person in charge, the date, and the main conclusions of several preliminary studies carried out before the bridge design itself: traffic limitations, location, hydraulic limitations, environmental impact, economic feasibility, and structural preliminary study (Figure 9-18). With respect of the last factor, the structural solutions prescribed and the budget estimate are also described.

9.6.1.2. Characteristics of the Ways to the Bridge

The second section defines the essential characteristics of the communication routes that the bridge serves or overpasses. Schematic representations of the transverse and longitudinal sections and of the axis are shown on the screen, for both the upper and the lower ways (Figure 9-19). These are chosen from standard sections and axes menus that are shown on special screens prepared for that effect. The user merely chooses the standard section or axis that corresponds to the way in question and fills in the distances, slopes, and curvature radii necessary for its complete definition (Figure 9-20). For the upper way, the wearing surface type and bridge skew also need to be provided. In this section, the access embankments to the bridge adjoining area are also characterized. Specifically, the following data are stored: total length in back fill and excavation and the respective slope, materials used, need for stabilization, and total length of retaining walls.

9.6.1.3. Structure Description

The third section's function is to describe the structural solution prescribed in the bridge structural plans. The various elements that constitute the bridge (including the non-structural elements) are described and reference numbers for the drawings in which they are defined are described (Figure 9-21) (de Brito and Branco 1991):

- longitudinal design (type, spans, and connection deck-columns);
- deck (structural design, materials used, and total width);
- transverse girders (number per span);
- road wearing surface (average thickness, materials used, waterproofing membrane, and cathodic protection);

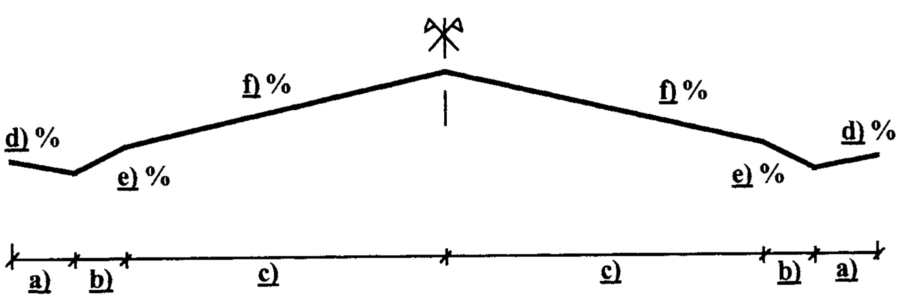

Figure 9-20. Standard road transverse section where the user fills in the several writing fields

REFERENCE STATE FORM

CODE NUMBER: ___________

STRUCTURE DESCRIPTION

10. LONGITUDINAL DESIGN

10.1. TYPE: ___________________________ m

10.2. SPANS (NUMBER _____): _____ m; _____ m; ....

10.3. CONNECTION DECK - COLUMNS: _________

11. DECK

11.1. STRUCTURAL DESCRIPTION: __________________________

11.2.MATERIALS USED: ___________________________________

11.3.TOTAL WIDTH: _________ m

11.4.DRAWINGS NUMBERS: _________________________________

12. TRANSVERSE GIRDERS:

12.1.NUMBER/SPAN: ______

12.2.DRAWINGS NUMBERS: _________________________________

13. WEARING SURFACE

13.1.AVERAGE THICKNESS: ____cm; 13.2. MATERIALS USED ________

13.3.WATERPROOFING MEMBRANE? ___

13.4.CATHODIC PROTECTION? ___

14.SECONDARY ELEMENTS

14.1.SIDEWALKS

14.1.1.AVERAGE THICKNESS: ____cm; 14.1.2. MATERIALS USED _____

14.1.3.WATERPROOFING MEMBRANE? ___

14.1.4.CATHODIC PROTECTION? ___

14.2.DRAINAGE SYSTEM

14.2.1.MINIMUM SLOPE: ___ %; 14.2.2. MATERIALS USED _________

14.2.3.DECK DRAINS LOCATION - DRAWING NUMBER: _____________

14.3.CURBS - DRAWING NUMBER: _____________

14.4.TRAFFIC BARRIER WALL - DRAWING NUMBER: _____________

14.5.HAND RAILING - DRAWING NUMBER: _____________

14.6.EDGE BEAMS - DRAWING NUMBER: _____________

14.7.ACROTERIA - DRAWING NUMBER: _____________

14.8 UTILITIES PIPING - DRAWING NUMBER: ______________________

14.9.TRAFFIC SIGNS - DRAWING NUMBER: ____________________

14.10.LIGHTING - DRAWING NUMBER: ______________________

Figure 9-21. Structure description at the design stage screens from the reference state form

15.JOINTS

15.1.LOCATION: ____________________ ; 15.2. TYPE: ________

15.3.DRAWING NUMBER: ______________________

16.BEARINGS

16.1.FIXED BEARINGS

16.1.1.LOCATION: ____________________ ; 16.1.2. TYPE: ______

16.1.3.DRAWING NUMBER: ___________________

16.2.BEARING PROVIDING TRANSLATION

16.1.2.LOCATION: ____________________ ; 16.2.2. TYPE: ______

16.2.3.DRAWING NUMBER: ____________________

17. COLUMNS

17.1.STRUCTURE DESCRIPTION: __________________________

17.2. MATERIALS USED: ________________________________

17.3.NUMBER: _______

17.4.DRAWINGS NUMBERS: ______________________________

18.ABUTMENTS

18.1.UPSTREAM ABUTMENT

18.1.1.STRUCTURE DESCRIPTION: _________________________

18.1.2. MATERIALS USED: ________________________________

18.1.3.DRAWINGS NUMBERS: ____________________________

18.2.DOWNSTREAM ABUTMENT

18.2.1.STRUCTURE DESCRIPTION: _________________________

18.2.2. MATERIALS USED: _______________________________

18.2.3.DRAWINGS NUMBERS: ____________________________

19. FOUNDATIONS

19.1.FOUNDATION SOIL

19.1.1.DESIGN ALLOWABLE TENSION (FOOTINGS/CAISSONS): ___ MPa

19.1.2.DESIGN STRENGTH AT THE POINT (END-BEARING PILES): c) kN

19.1.3.DESIGN FRICTION TENSION (FRICTION PILES): _____ MPa

19.1.4.SINGULARITIES: _________________________________

19.2.STRUCTURE DESCRIPTION

19.2.1.COLUMNS: ____________ ; 19.2.2. ABUTMENTS: _____

19.3. MATERIALS USED: _______________________________

19.4.DRAWINGS NUMBERS: ____________________________

20.APPROACH SLABS (DRAWING NUMBER): ________________

21.RETAINING WALLS (DRAWING NUMBER): ________________

Figure 9-21, Continued. Structure description at the design stage screens from the reference state form

- secondary elements (sidewalks—casing average thickness, materials used, waterproofing membrane, and cathodic protection; drainage system—drains minimum slope, materials used, and deck drains location; curbs; traffic barrier wall; hand railings; edge beams; acroteria; utilities piping; traffic signs; lighting);
- joints (location and type);
- bearings (fixed and providing translation bearings—structural solution and materials used);
- columns (structural solution, materials used and number);
- abutments (upstream and downstream—structural solution and materials used);
- foundations (soil characterization—design allowable tensions and loads and singularities; structure description—columns, abutments and materials used); approach slabs;
- retaining walls.

To facilitate the insertion of data in the form, taking into account that the person who does it is not necessarily a structural engineer, menus corresponding to all the writing fields in which the number of possible options is limited must be conceived. For example, when filling out the field correspondent regarding materials used for sidewalk casings, the user may access the following menu (de Brito and Branco 1991):

PREFABRICATED CONCRETE SLABS

PLAIN CONCRETE

STEEL SHEETS

MASONRY

BRICK

ASPHALT

If the actual material is not included in the menu, the database automatically adds it after the user writes it down.

9.6.1.4. Design Traffic

The fourth section describes the main parameters resulting from the preliminary studies concerning bridge traffic, which were used as basic data for the bridge planning: type of traffic, code used in the structural design, bridge loading class, number of lanes, type of standard vehicle considered, uniform live load, design traffic speed, maximum traffic volume during the bridge service life, special vehicles, and traffic restrictions (Figure 9-22).

9.6.2. Post-Construction Stage

In this submodule, all the information necessary to establish the actual reference state of the bridge, and specifically all the changes from the design stage made during or after the

REFERENCE STATE FORM

CODE NUMBER: ______

DESIGN TRAFFIC

22. TYPE OF TRAFFIC ______

23.CODE USED IN THE STRUCTURAL DESIGN: ______

24.BRIDGE LOADING CLASS: ______ ; 25. NUMBER OF LANES: ______

26.STANDARD VEHICLE: ______ kN/axle

27.UNIFORM LIVE LOAD: ______ kN/m^2

28.DESIGN TRAFFIC SPEED: ______ km/h

29.MAXIMUM TRAFFIC VOLUME DURING THE BRIDGE SERVICE LIFE:

______ vehicles / year OR ______ vehicles / day OR ______ vehicles / hour

30.SPECIAL VEHICLES CONSIDERED: ______

31.TRAFFIC RESTRICTIONS

31.1. MAX. LOAD/AXLE: ______ kN; 31.2. MAX. GLOBAL LOAD: ______ kN

31.3. MAX. HEIGHT: ______ m; 31.4. MAX. WIDTH: ______ m

31.5. MAX. SPEED: ______ km/h; 31.6. MAX. WIND SPEED: ______ km/h

Figure 9-22. Traffic at the design stage screen within the reference state form

construction, is included. As discussed previously, this information is divided into three sections (Figure 9-17) (de Brito and Branco 1991):

- changes from the design stage (Figure 9-23);
- tests performed during the construction or at its end (Figure 9-24);
- traffic on the bridge (Figure 9-25).

9.6.2.1. Changes from the Design Stage

The first section lists, for each construction element, the dimension or modified reinforcement and the reference numbers of the design stage and as-built drawings, in terms of both geometry and detailing (Figure 9-23). The defects detected at the handing-over of the bridge (theoretically taken care of at the expense of the builder before the bridge is put in service) are also described: element affected, defect form number (Table 10-1), short description, curative measures implemented, and an additional costs estimate. Finally, all the items that must be regularly monitored during future inspections are discussed.

REFERENCE STATE FORM

CODE NUMBER: ___________

CHANGES FROM THE DESIGN STAGE

32.GEOMETRY

32.1.STRUCTURAL ELEMENTS

32.1.1.ELEMENT N° ___________

32.1.1.1.ELEMENT: ___________

32.1.1.2.DIMENSION MODIFIED: ___________

32.1.1.3.INITIAL DRAWING: ________ ; 32.1.1.4.AS BUILT DRAWING ________

32.1.2. ....

....

....

....

32.2.2 SECONDARY ELEMENTS / APPROACHES

32.2.1.ELEMENT N° ___________

32.2.1.1.ELEMENT: ___________

32.2.1.2.DIMENSION MODIFIED: ___________

32.2.1.3.INITIAL DRAWING: ________ ; 32.1.1.4.AS BUILT DRAWING ________

32.2.2. ....

....

....

....

33.REINFORCEMENT/STRANDS DETAILING

33.1.ELEMENT N° ___________

33.1.1.ELEMENT: ___________

33.1.2.REINFORCEMENT MODIFIED: ___________

32.1.3.INITIAL DRAWING: ________ ; 33.1.4.AS BUILT DRAWING ________

33.2. ....

....

....

....

34.DETECTED ANOMALIES

34.1.ANOMALY N° ___________

34.1.1.ELEMENT: ___________

34.1.2.FILE REFERENCE: ________

Figure 9-23. Changes from the design stage at the post-construction stage screens within the reference state form

34.1.3.SHORT DESCRIPTION: ______________________________

34.1.4.MEASURES TAKEN: ______________________________

34.1.5.ADDITIONAL COSTS (ESTIMATE): ______________ $

34.2. ....

....

....

....

....

....

35. ASPECTS WORTH MONITORING CLOSELY IN THE INSPECTIONS:

____

____

____

____

____

____

Figure 9-23, Continued. Changes from the design stage at the post-construction stage screens within the reference state form

9.6.2.2. Tests Performed During or at the Completion of Construction

The second section describes the main results of the tests performed during and at the completion of bridge construction. The first results include the materials tests (steel and concrete) and the second results include static and dynamic global load tests and other tests (Figure 9-24). For each test, a comparative analysis of the results obtained against those expected is made and the eventual discrepancies are discussed. For the global load tests, the person in charge, the loading system, and the anomalies detected are identified. A future service load for the bridge, not necessarily the same as in the initial design, is prescribed as a function of the load tests results.

9.6.2.3. Traffic on the Bridge

In the third section, the main data and results from the traffic census that must be implemented after the bridge is put in service are described: person in charge, year, period of time, measured traffic volume, percentages and maximum values, traffic restriction breaches detected, and so forth. (Figure 9-25). A list of all the anomalies detected in the normal traffic flow over/under the bridge is also prepared.

9.7. Inspection Forms

The inspections forms contain all the relevant information about the periodic inspections (current and detailed) and the structural assessments (defined in Chapter 10). This infor-

REFERENCE STATE FORM

CODE NUMBER: ___________

TESTS PERFORMED DURING THE CONSTRUCTION OR AT ITS END

36.MATERIALS

36.1.STEEL

36.1.1.TEST N°: ________

36.1.1.1.MEASURED PARAMETER: ________________________

36.1.1.2.TEST VALUE: ________ ; 33.1.1.3.DESIGN VALUE: ________

36.1.2. ....

....

....

36.2.CONCRETE

36.2.1.TEST N°: ________

36.2.1.1.MEASURED PARAMETER: ________________________

36.2.1.2.TEST VALUE: ________ ; 33.1.1.3.DESIGN VALUE: ________

36.2.2. ....

....

....

37.LOAD TEST

37.1.PERSON IN CHARGE: _________________ ; 37.2. YEAR: ______

37.3.LOADING SYSTEM: ____

37.4.LOAD IMPOSED: ____

37.5.LOAD FACTOR AS COMPARED TO THE DESIGN LOAD: ________ %

37.6.MAIN RESULTS: ____

____

37.7.ANOMALIES DETECTED: ____

____

37.8.FUTURE RECOMMENDED SERVICE LOAD: __________________________

37.9.REMARKS: ____

____

38.GLOBAL VIBRATION TEST

38.1.PERSON IN CHARGE: _________________; 38.2. YEAR: ______

38.3.EXCITATION METHOD: ____

38.4.MAIN RESULTS ____

38.4.1.FUNDAMENTAL FREQUENCY

38.4.1.1 IN SITU VALUE: _____ Hz; 38.4.1.2.DESIGN VALUE: _____ Hz

Figure 9-24. Tests performed during the construction or at its end screens within the reference state form

38.4.2.RESPONSE AMPLITUDE

38.4.2.1 IN SITU VALUE: ___ mm; 38.4.2.2.DESIGN VALUE: _____ mm

38.4.3.DAMPING COEFFICIENT

38.4.3.1.IN SITU VALUE: ________ ; 38.4.3.2.DESIGN VALUE: ________

38.4.4.ANOMALIES DETECTED: ____

____

38.4.5.REMARKS: ____

____

39.OTHER TESTS

39.1.TEST N°: _________

39.1.1.MEASURED PARAMETER: __________________________

39.1.2.IN-SITU VALUE: ___________ ; 39.1.3.DESIGN VALUE: ____________

39.2. ....

....

....

Figure 9-24, Continued. Tests performed during the construction or at its end screens within the reference state form

mation is fundamental for future diagnosis and allows the inspector to follow the evolution of the defects detected and to plan the maintenance and repair work. The information is also an indispensable tool for parametric studies and for the development of mathematical degradation mechanisms.

Each inspection form is divided in three information modules (Figure 9-26) (de Brito and Branco 1991):

- inspection characteristics;
- detected defects;
- inspections history.

At the end of this form, there are two screens, "Form History" and "Form Protection", which have the same functions as those described for the identification form. In addition to the information described then, the form history contains a list of visas and of copies to be made at headquarters in order to follow administrative procedures for the inspection process.

The first piece of data inserted into the reference state information form is the bridge code number described previously. The inspection form number relating to the bridge chosen is provided next. When creating a new form, the database automatically supplies this number. Conversely, a menu with the numbers of already existing forms is shown on the screen, from which the user picks the one he wants. The name of the bridge is inserted automatically, based on the previously filled in identification form, and the user has the option of accessing any of the three information submodules described previously (Figure 9-26) or returning to the general menu.

REFERENCE STATE FORM

CODE NUMBER: __________

TRAFFIC ON THE BRIDGE

40.TRAFFIC CENSUS

40.1.PERSON IN CHARGE: ________________ ; 40.2. YEAR: ______

40.3.PERIOD OF TIME: ___ DAYS

40.4 MAX. DAILY TRAFFIC VOLUME MEASURED: ______ vehicles

40.5.% NORMAL TRAFFIC: _____ ; 40.6.% HEAVY TRAFFIC: _____

40.7.% 2 WHEELS TRAFFIC: _____

40.8.MAX. LOAD/AXLE MEASURED: _____ kN

40.9.MAX. GLOBAL LOAD MEASURED: _____ kN

40.10.MAX.HEIGHT MEASURED: _____ m

40.11.MAX. WIDTH MEASURED: _____ m

40.12.MAX.SPEED MEASURED: _____ km/h

40.13.TRAFFIC RESTRICTIONS BREACHES (NUMBER / %)

40.13.1.LOAD / AXLE: __________ ; 40.13.2.GLOBAL LOAD: __________

40.13.3.HEIGHT: __________ ; 40.13.4.WIDTH: __________

40.13.5.SPEED: __________

41. DETECTED ANOMALIES

41.1.BOTTLE-NECKING AT THE BRIDGE: ___

41.2.EXCESSIVE VIBRATION FROM TRAFFIC: ___

41.3.EXCESSIVE CENTRIFUGAL FORCE: ___

41.4.EXCESSIVE SLOPE AT THE APPROACHES: ___

41.5.PAVEMENT IRREGULARITIES: ___

41.6.DISCOMFORT / UNSAFETY FEELING: ___

41.7.INADEQUATE / INEXISTENT TRAFFIC SIGNS: ___

41.8.INADEQUATE / INEXISTENT LIGHTING: ___

41.9.DANGER TO THE PEDESTRIANS: ___

41.10.OTHERS: ____

____

41.11.REMARKS: ____

____

Figure 9-25. Traffic in the bridge at the post-construction stage screens within the reference state form

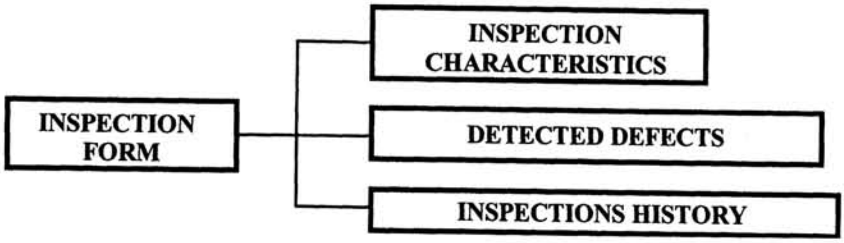

Figure 9-26. Information contained within an inspection form

9.7.1. Inspection Characteristics

In this submodule the main general characteristics of the inspection are described. The inspection type must be either current or detailed since this form is not prepared to describe structural assessments. As discussed in Chapter 8, structural assessments are variable in nature and are a function of the motive for its implementation and of the tests to be performed, thus making it almost impossible to standardize the corresponding report. The main results of an eventual future structural assessment (caused by the findings of a periodic inspection) are provided in the next information submodule (detected defects). The inspection is dated and the team that performed it is described by name and by the role of each member (Figure 9-27). The means of access and the equipment used are also shortly described in the inspection form. Finally, the weather conditions and relevant singularities during the inspection, which may be meaningful for future inspections, are described. All the information contained in this submodule is fundamental in the planning of the next inspections. Whenever possible, the user resorts to preestablished menus to fill in the various fields of the form in a standardized manner.

9.7.2. Detected Defects

This submodule is extremely important for the bridge management system as a whole. The most important findings of the inspection system (Chapter 10) are stored in the detected defects module. The items "Maintenance Work Needed" and "Repair Work Needed" function as databases for the decision subsystems relating, respectively, to maintenance/small repair (Chapter 11) and rehabilitation/replacement (Chapter 13). All defects (structural or otherwise) detected are described: the element affected, the form number (Table 10-1), the graphic location (according to grid references such as the one exemplified in Figure 9-28), the parameter measured and the corresponding value, classification attribution (according to the criteria defined in Chapter 11), and pertinent remarks (about probable causes, possible consequences, evolution potential, etc.) (Figure 9-29). By comparing various inspection forms for the same bridge, it is possible to follow the evolution of the defects with time. This short description almost eliminates the need to prepare an inspection report.

The item "Maintenance Work Needed" contains a list of repair techniques defined as maintenance (Table 10-3) proposed by the inspector in order to eliminate the most important nonstructural defects detected during periodic inspections. There is a relationship between the techniques and the defects, which is achieved based on the inspector's experience and with help from the correlation matrix defects—repair techniques (Chapter 10) and, at further stages of development of the system, based on the type 4 parameters (defined in Chapter 11). Also included in the form are the estimated quantities of work for

INSPECTION FORM

CODE NUMBER: __________

FORM NUMBER: ______

INSPECTION CHARACTERISTICS

1.TYPE: ______________________________ ; 2.DATE: ____________

3.INSPECTION TEAM:

3.1.MEMBER N°: _____

3.1.1.NAME: ____

3.1.2.ROLE: ____

3.2. ....

....

....

4.EQUIPMENT USED

4.1.MEANS OF ACCESS: ____________________________________

4.2.EQUIPMENT: ____

5.WEATHER CONDITIONS: __________________________________

____

6.RELEVANT SINGULARITIES: _______________________________

____

Figure 9-27. Inspection characteristics screens within the inspection form

each technique that, together with a unit price list, allow preparation of a rough maintenance budget. Any additional remarks that the inspector considers relevant are also added.

After a periodic inspection, a decision must be made concerning the need to request a structural assessment (Chapter 13). If that is the option, information needed to plan the inspection is inserted into this submodule: maximum notice, fields to investigate, special equipment needs (including special access means), and elements to investigate.

The item "Repair Work Needed" contains a list of structural repair techniques proposed by the inspector in order to eliminate the most important structural defects detected, both during the periodic inspections and in the structural assessments. As often happens with maintenance, there is a relationship between these techniques and the defects, which is obtained in a similar way. In addition to the estimated amount of work for each technique, other elements necessary to the decision system (Chapter 13) are provided: estimated time for repair, average number of lanes closed to traffic during repair, and the estimated useful life of the repair. The functional failure costs associated with the repair can be substantially higher than the repair costs themselves and must therefore be included in the economic analysis (Chapter 12) in order to choose the best repair option. Finally, the most relevant remarks are also added.

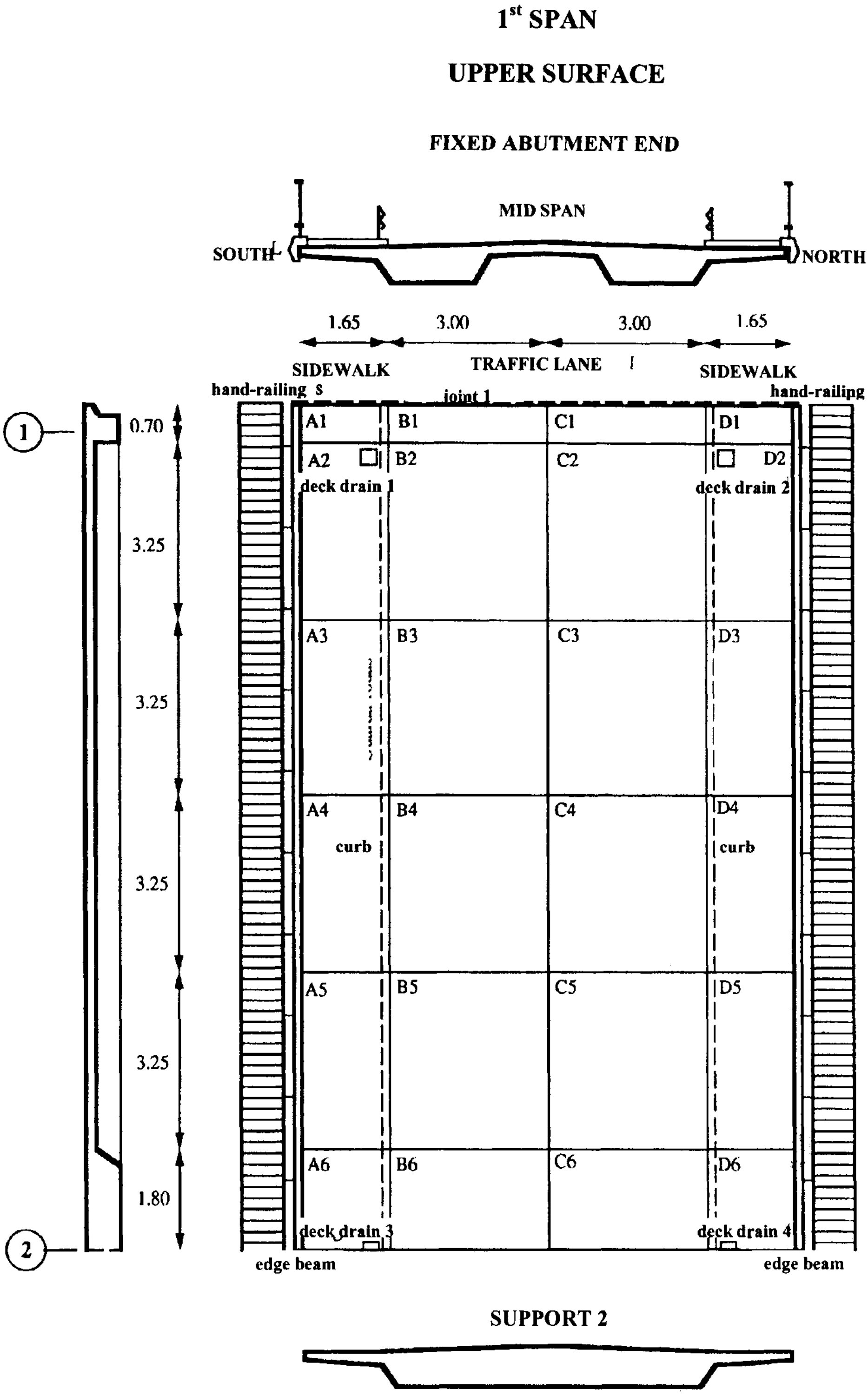

Figure 9-28. Example of a graphic representation of a bridge with a superimposed reference grid

INSPECTION FORM

CODE NUMBER: __________

FORM NUMBER: _____

DETECTED DEFECTS

7.DEFECTS DESCRIPTION

7.1.DEFECT N°: _____

7.1.1.ELEMENT: ______________________________

7.1.2.FORM REFERENCE: __________ ; 7.1.3.GRID REFERENCE: ________

7.1.4. PARAMETER MEASURED: ____________ ; 7.1.5.VALUE: __________

7.1.6.CLASSIFICATION: ____________

7.1.7.REMARKS (CAUSES, CONSEQUENCES, ETC.): ______________________

____

7.2. ....

....

....

....

....

....

....

8.MAINTENANCE WORK NEEDED

8.1.REPAIR TECHNIQUE N°: _____

8.1.1.FORM REFERENCE: __________

8.1.2.DEFECTS ELIMINATED: ______________________________

8.1.3.ESTIMATED QUANTITY: ______________

8.1.4.REMARKS: ______________________________

____

8.2. ....

....

....

....

....

....

9.STRUCTURAL ASSESSMENT

9.1.NECESSARY? ________ ; 9.2.IF SO, AT WHICH NOTICE? ____________

9.3.FIELDS TO INVESTIGATE: ______________________________

Figure 9-29. Detected defects screens within the inspection form

9.4.SPECIAL EQUIPMENT NEEDED: ____________________

9.5.ELEMENTS TO INVESTIGATE: ____________________

10.REPAIR WORK NEEDED

10.1.REPAIR TECHNIQUE N°: _____

8.1.1.FORM REFERENCE: ________

10.1.2.DEFECTS ELIMINATED: ____________________

10.1.3.ESTIMATED QUANTITY: __________

10.1.4.ESTIMATED TIME OF REPAIR: _____ days

10.1.5.AVERAGE NUMBER OF LANES CUT TO TRAFFIC DURING REPAIR: _____

10.1.6.ESTIMATED USEFUL LIFE OF THE REPAIR: ____ years

10.1.7.REMARKS: ____________________

10.2. ....

Figure 9-29, Continued. Detected defects screens within the inspection form

9.7.3. *Inspections History*

As the title suggests, this submodule portrays the bridge inspection history and allows for the verification of the regularity with which inspections have been performed. Past periodic inspections are described by type, date, bridge general condition, and relevant remarks (Figure 9-30). Interventions outside the maintenance scope are also related: nature, date, and elements affected. Finally, the date of the valid reference state is given.

9.8. Reports Generation

The reports generation option is extremely important, because it permits the user to select only the information that he actually needs in each specific circumstance. The report is shown on the computer screen so that the user can verify that the information selected is in fact what he wants. The report can then be printed in accordance with a standard template.

INSPECTION FORM

CODE NUMBER: __________

FORM NUMBER: _____

INSPECTIONS HISTORY

11.CURRENT / DETAILED INSPECTIONS

11.1.INSPECTION N° _____

11.1.1.TYPE: ____________________ ; 11.1.2.DATE: ____________

11.1.3.BRIDGE GENERAL CONDITION: ______________

11.1.4.REMARKS: ____

____

11.2. ....

....

....

....

....

12.INTERVENTIONS OUTSIDE MAINTENANCE

12.1.INTERVENTION N° _____

12.1.1.NATURE: ____

12.1.2.DATE: __________

12.1.3.ELEMENTS AFFECTED: ______________________________________

12.2. ....

....

....

....

13.REFERENCE STATE DATE: ___________

Figure 9-30. Inspections history screens within the inspection form

The system must be prepared to generate all the reports, which will be useful to the inspector and contain the following options (de Brito 1992):

- a list of all bridges programmed for inspection in the following *x* months with an indication of the date planned; the type if inspection and length of time needed; the personnel, equipment, tests, and means of access that probably will be necessary (based on previous inspections forms); restrictions to take into account; administrative information; and other aspects necessary for planning. The list can also be limited according to the location, age, structural type, or other characteristics of each bridge; the inspection type; the personnel, equipment, tests, or means of access necessary; or any other parameter that may influence the inspection planning;

- identification, graphic information (with simplified schematic drawings of the structure), reference state or inspection forms of various bridges according to any of the parameters mentioned above;
- a simplified version of the inspection manual according to the structural type of the bridge(s) to be inspected;
- a simplified list of all the system bridges or of a selection of those in terms of age, structural type, location, or other characteristic, with an indication of the entity or agent responsible for the inspection and maintenance;
- a list of the maintenance work performed on a selection of bridges in a selected period of time (or since their construction), with an indication of the global costs for each year;
- an identical list for repair work;
- a list of the bridges with an estimated load-bearing capacity that does not comply with the present or future codes (which is equivalent to the bridges posted);
- a list of the bridges that may carry certain standard classes of exceptionally heavy vehicles in accordance with the load-bearing capacity of the bridge (in order to define their itinerary);
- a list of availability in terms of personnel, equipment, and special means of access during a certain period of time according to the district;
- budget estimates for the present year of maintenance and repair work, necessity planning, and amounts already spent;
- results from the economic analysis made with specific software of long-term costs quantification and prediction (Chapter 12);
  - tables of yearly partial costs broken down into various elements for each bridge and the results of the sensibility analysis for partial costs;
  - tables that summarize the yearly partial costs for each bridge and results of the sensibility analysis only for the global costs;
  - a table that summarizes the yearly global costs for each bridge;
- results from the maintenance/small repair decision subsystem (Chapter 11);
  - a list of the maintenance work to be executed during the present year based on priority;
  - a list of the maintenance work needed that will be executed during the present year due to budget limitations;
  - the deficit or surplus of the maintenance budget;
- results from the inspection strategy submodule based on a reliability analysis within the rehabilitation/replacement decision subsystem (Chapter 13);
  - a prediction of the evolution of the reliability index in the most conditioning cross-sections and corresponding ultimate limit states in a time frame that goes from the last periodic inspection to the next one;

  - a prediction of the evolution of the reliability index that models the global failure of the bridge during the same period;
  - a proposal for or against the implementation of a structural assessment for the bridge during the same period;
  - a list of the defects that must be analyzed during the structural assessment with their location and extension;

- results from the repair work selection submodule within the rehabilitation/replacement decision subsystem (Chapter 13);
  - a list of the repair work to be executed during the present year according to priority;
  - a list of the repair work needed that will be executed during the present year due to budget limitations;
  - the deficit or surplus of the repair budget;

This options list can be increased, limited, or corrected in accordance with the experience that is gained as the management system is implemented.

9.9. Database Users' Manual

Simultaneously with the implementation of the database, it is necessary to prepare a users' manual. The manual content is in great part common to this chapter and must include the following elements (de Brito 1992):

- a description of the database general architecture, its objectives, the type of information that it stores, and the way it is taken into account in the various bridge management system modules;
- a description of the database general utilization options;
- a description of the particular utilization options within the various forms that the database comprises;
- a summary description of the information contained in each of those forms.

Following the example of the Database User's Manual of the Ontario Structure Inspection Management System (OSIMS) (Reel et al. 1988), it is proposed that after those elements there must exist a descriptive section in which the user gets to know how to use the database in practice. Specifically, the procedures for entering the system, changing from screen to screen, inserting new data, updating the existing data and all the other general options, are explained. The manual reproduces the computer screens that are displayed when entering the program and proceeding with it. For each screen, the fields that cannot be altered (according to the particular option chosen) are identified and their meaning is explained. For the writing fields that the user is supposed to fill in or alter, the type of entry (with or without a menu) and data that the program accepts are pointed out. The preestablished menus must be included in the description of the corresponding writing field. For each particular utilization option of the database, a complete example must be presented, in which all the computer screens are displayed before and after the user alters them. In particular, it must show how to create identification, reference state, and graphic information forms for a specific bridge and an inspection form for the same bridge, as well as the various options of reports generation.

The entire error messages specific to the database must be listed and exemplified in an appendix, as well as the appropriate format for each writing field and any other information that may be useful to the user (Reel et al. 1988).

The manual is very meticulous, thus leading to repetition of rules in several places. It, therefore, becomes bulkier, but it also is easier to comprehend, particularly for the non-technical personnel who use it.

CHAPTER 10

Inspection Strategies

10.1. Procedures Standardization

Historically, the inspection of existing bridges has been treated as a secondary task of a semi-random nature. The inspections were usually performed as a result of warnings received from sources very often outside the bridge network organization; or as a result of human or material losses resulting from a major inadequacy of the bridge that does not allow it to fulfill the function for which it was designed. Except for the most developed countries, the greatest priority in terms of public investment is still to widen existing bridges and road networks rather than maintaining existing infrastructures. For these countries, as infrastructure saturation is achieved in progressively wider areas and the existing network continues to grow, priority shifts in the budget toward maintenance rather than building new bridges, as shown in (Figure 10-1) (Schneider 1988).

Consequently, bridge inspection in all but the most developed countries is still performed somewhat haphazardly, without planning or with planning only in the short term, and it is common to find bridges that have never been inspected since they were put into service. Even when that is not the case, several years may pass without a bridge being inspected if there is no major reason to do so. Consequently, there is no regular flow of information about the general condition of each bridge that would allow analysis and prediction of its degradation with time, which is only possible if a regular, periodic, and standardized inspection program is operational.

In this chapter, the general organization of a typical inspection system is described: types of inspection with their timing and objectives. A description is included of a standard classification system devised to reduce the subjectivity of reports and conclusions drawn from an inspection: defects, their possible causes, current repair techniques, and diagnostic and surveillance methods. The correlation between the defects themselves and the remaining items is presented with the use of matrices. To further support in situ inspections, some instruments, also described here, must be devised and implemented: an inspection manual, a computer-based inspection support module (ISM), and a bridge dossier.

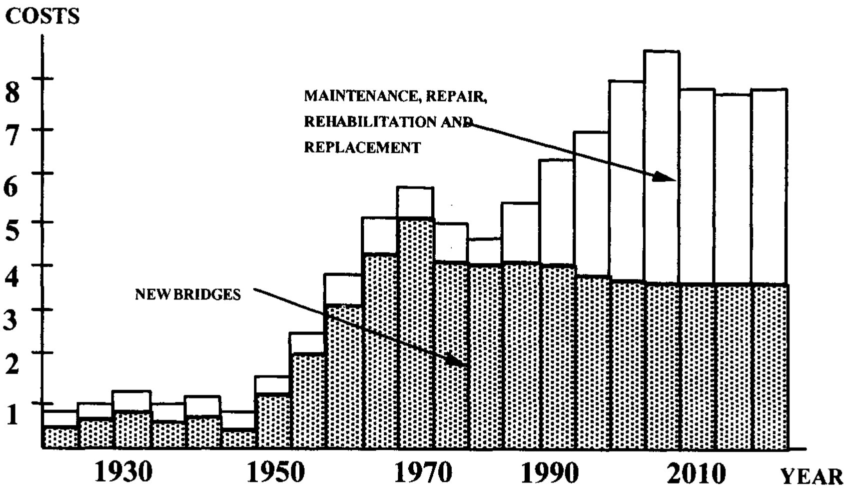

Figure 10-1. Qualitative evolution of the global bridge budgets in Switzerland

10.2. General Organization of a Knowledge-Based Inspection System

An inspection system must be devised at the bridge network level and not at the single bridge level. Therefore, the inspection of each bridge must be performed at the same time as the adjoining bridges, if possible, to dilute fixed costs and increase efficiency. The use of the most expensive and difficult to transport equipment must be maximized and is an important element in inspection planning. The routine inspection calendar should not be changed frequently, however, and, when changed, notice should be given in advance.

Routine inspections must not only be planned but also must be performed at fixed periods of time. The quality of the inspection is directly related to the knowledge of the individual who is performing the inspection and compliance with certain detailed procedures. The inspection is more efficient from a cost standpoint when it is conducted based on a specific routine. An inspector who is less familiar with the procedures, as a result of frequent absences, will be less efficient and there is a greater probability that he will fail to detect the problems (Little 1990).

In Chapters 6 and 7, several management systems, either in use or being implemented, were presented. Not all of the management systems presented put special emphasis on the inspection system. The following references, which explicitly describe the inspection system, are cited:

- Belgium—(de Buck 1987)
- Canada (Ontario)—(Reel and Conte 1989)
- Cyprus—(May and Vrahimis 1990)
- Denmark—(Sørensen and Berthelsen 1990)
- Finland—(Kähkönen and Marshall 1990)
- France—(MTRD 1979)

- Germany—(Zichner 1987)
- Italy—(Malisardi and Nebbia)
- Japan—(Miyamoto 1989)
- Portugal—(Santiago 1990)
- Sweden—(Lindblath 1990)
- Switzerland—(Andrey 1987)
- United Kingdom—(Holland and Dowe 1990)
- United States of America (Florida)—(Little 1990)

The inspection general organization presented here as a reference follows very closely the system used in France (MTRD 1979) and the proposal made by Andrey (Andrey 1987), as can be seen in Table 7-8. The functionality of the management system is based on a standardized inspection strategy. It consists of a periodic set of inspections based on a fixed timetable, in which some flexibility is allowed to take into account a plausible global allocation of inspection resources, complemented by special inspections when something serious is detected or suspected. Three types of inspection are considered: (1) routine inspection, (2) detailed inspection, and (3) structural assessment.

10.2.1. Routine Inspection

The routine inspection is based almost exclusively on direct visual observation that, as will be shown later in this chapter, seems to be the diagnostic method with the greatest potential. During an inspection, no important structural defect is anticipated and the work recommended falls within the range of maintenance.

A period of 15 months between routine inspections is recommended so that the influence of the weather on the general condition and degradation of the bridge can be evaluated (Andrey 1987). However, the possibility of other unplanned visits to the bridge must not be ruled out, as long as they do not replace periodic inspections. These unplanned visits may occur right after a periodic inspection to clarify any points about which there is some doubt or to more precisely quantify the maintenance work needed. If that is considered useful, the inspection may be split into several partial inspections (MTRD 1979).

A routine inspection must be planned in advance to make the best of certain circumstances (e.g., traffic, weather conditions) that may facilitate detection of defects. The necessary means of access for each bridge and its distance from other bridges must also be taken into account in the planning (MTRD 1979).

The personnel required for routine inspection do not have to be overspecialized, but some field experience is advisable. The inspection team typically consists of two individuals unless the size of the deck justifies using an additional person(s). One of the individuals must know the inspection manual in detail and, if possible, he should be familiar with the bridge being inspected, even though he does not have to be an engineer.

The equipment to be taken to the site should be portable, independent of an exterior energy source, and simple to work with (Figure 7-19). The equipment might include: pencils, pens, crayon and markers, thermometer, rulers, tape-measure, cracks ruler, clinometer, plumb-bob, hammer, screw driver, sclerometer (Figure 10-2), chisel, portable lantern, photo and video cameras, binoculars, and so forth (MTRD 1979 and Reel and

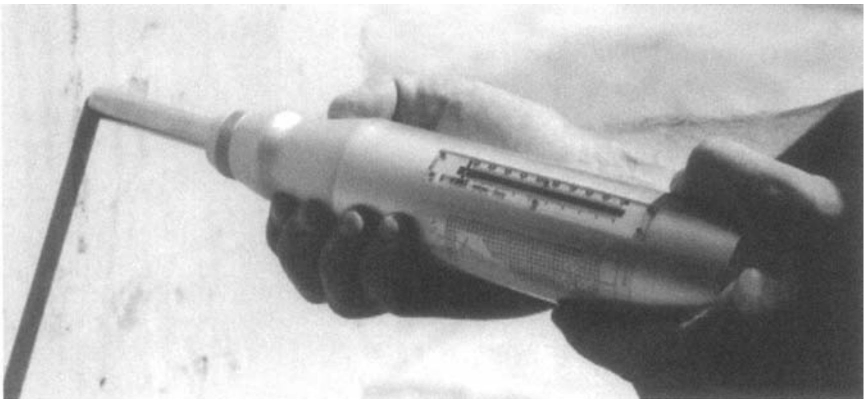

Figure 10-2. Schmidt hammer/sclerometer

Conte 1989) The possibility of using a galvanic half-cell in this type of inspection should be considered. In principle, there is no need for special means of access (Figure 10-3) (Santiago 2000).

Routine inspections should allow for the detection of quickly evolving defects and to monitor defects detected during earlier inspections. The inspections are limited to direct visual observations of the bridge's most exposed areas and to the detection of surface de-

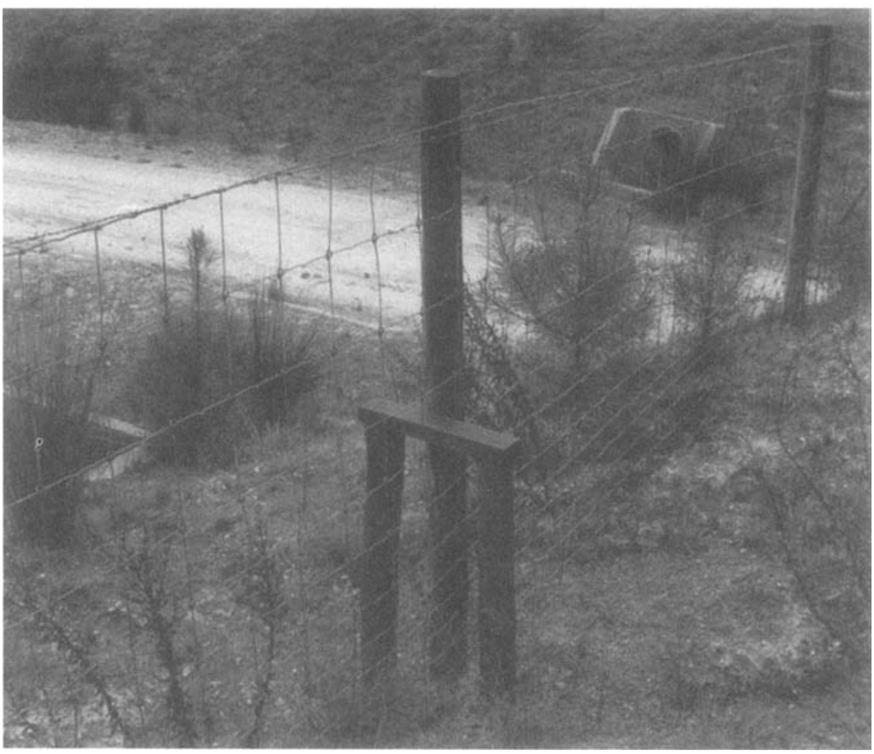

Figure 10-3. Simple device to clear fences

fects. A list of points to be analyzed must be included in the inspection manual and should be adapted to take into account previous inspections and the bridge type. If surveillance equipment has already been installed, its readings must be obtained. The weather conditions and temperature at the site must be registered because some defects (cracks, deflections, etc.) result from them. It should be noted if unexpectedly heavy lorries are found to be using the bridge. Even though the objective of this inspection is to observe and not to analyze, it is always useful to try to understand and compare the observations made to avoid having to come back later (MTRD 1979).

The inspection report consists fundamentally of the inspection form described in Chapter 9. The defects are recorded and referenced (based on the classification system described later in this chapter) and rated (in accordance with the criteria described in Chapter 11). They must also be registered graphically in simplified schematic diagrams of the structure (prepared in advance if possible). It is always useful to include photographs, as long as they are properly referenced and scaled (Figure 10-4) (Santiago 2000). At headquarters, the item "Maintenance Work Needed" from the inspection form is filled in and a decision is made concerning the need to implement a structural assessment. The structural defects detected must be registered even though "Repair Work Needed" is filled in only after a structural assessment is performed. All the data collected are dated and appended to the bridge dossier.

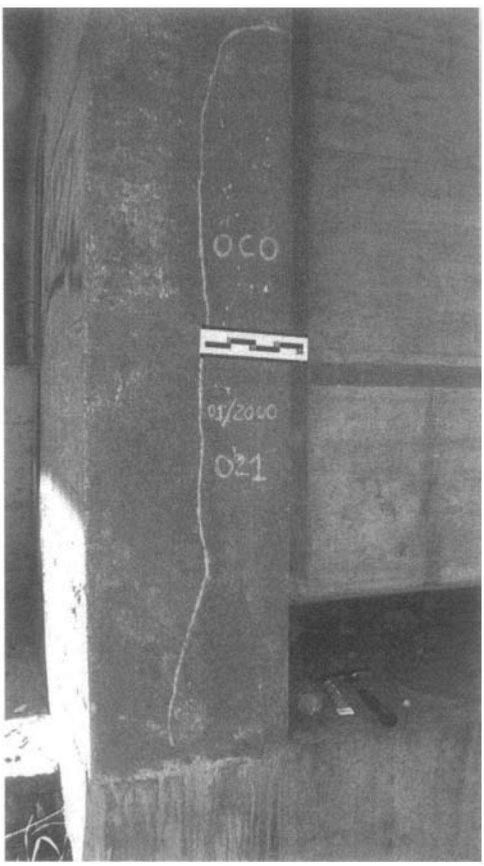

Figure 10-4. Example of a defect properly referenced and scaled

Figure 10-5. Routine maintenance during an inspection: vegetation removal

A proposal has been made and tested with good results (Santiago 2000) to implement a certain number of small tasks concerning maintenance during periodic inspections: vegetation removal (Figure 10-5) (Santiago 2000), cleaning bearings, and nails and formwork pieces removal, and so forth.

10.2.2. Detailed Inspection

In the detailed inspection, easy and fast nondestructive in situ tests are performed (Figure 10-6) (Cânovàs) in addition to direct visual observation as a means of investigating every detail that may potentially lead to future problems. There is a possibility that special means of access may be used if such is considered indispensable. If any main structural defect is detected, a structural assessment *must* be recommended. A structural assessment is outside the scope of the detailed inspection, which focuses mainly on general maintenance. The period recommended for a detailed inspection is 5 years and replaces a routine inspection if their calendars coincide (Andrey 1987).

Even though it is highly recommended that a fixed period between detailed inspections is maintained, it is also possible to pay a visit to a bridge outside the dates planned for the long term, particularly under the same circumstances as those described for impromptu routine inspections. A preliminary visit to the bridge site may be useful to evaluate existing conditions. When there is a need to follow up the evolution of certain defects with greater frequency, however, the period between visits may be reduced to 1 year, especially for localized areas of the bridge (MTRD 1979). Detailed inspections should also consider the design inspection plan, which is defined in accordance with the estimated service life of structural components.

Planning a detailed inspection includes a careful study of the bridge dossier to get to know the causes and evolution of the defects detected in the previous inspections and the specific points to be analyzed closely. The following documents deserve special attention: the reference state (described later in this chapter) and the associated form (described in Chapter 9), the as-built drawings, the last periodic inspection form (also described in Chap-

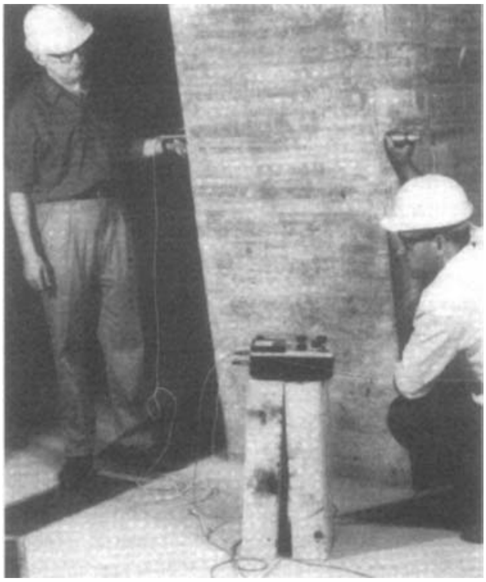

Figure 10-6. Ultrasonic pulse velocity test performed in situ

ter 9), and reports of all the structural assessments performed and of the maintenance and repair work implemented. Based on previous inspection forms and a preliminary visit to the site, the eventual special means of access needed are planned for. Total or partial closing of bridge lanes is also taken into account. The following documents must be brought to the site and/or prepared beforehand: a list of all the single points to be checked, schematics with reference grids of the most relevant elements, and the last periodic inspection form and the inspection manual.

Detailed inspections must be planned and led by an expert (graduate engineer) in the performance of inspections and have a detailed knowledge of the bridge's structural type. The other members of the personnel must have the correct level of specialization. The number of people required depends on the type and number of in situ tests planned and also on the deck area of the bridge.

In addition to the portable equipment listed previously for routine inspections, the following equipment is required for in situ testing and site measurements and control: galvanic half-cell (Figure 10-7, **a**), magnetometer (Figure 10-29), ultrasound (Figure 10-7, **b**) (Cánovàs), core extractor (Figure 10-30) (Cánovàs), displacement transducer, cracks microscope, strain gauges, and so forth. In most cases, there is no need for very high-tech equipment, since no specific serious problem is anticipated. The use of laboratory tests must be restricted to situations in which they are indispensable. The entire bridge, from infrastructure to superstructure, must be surveyed, based on the premise that eventually there will be a need for access that requires special equipment such as an inspection vehicle with bascule basket (Figure 10-8) (MTRD 1979), sliding or fixed scaffolding, diving equipment, or a rubber boat.

During a detailed inspection, there generally is no suspicion about any specific major defect, other than those that may have been detected during previous inspections. Even though the inspection range includes the entire bridge, the examination does not need to go very deep. During the inspection or after studying the reports, the existence of a serious

a.


b.

Figure 10-7. Half-cell of copper equipment based on **a.**, copper sulphate; **b.**, ultrasonic equipment

defect might be acknowledged or serious doubts about bridge safety may arise. The areas in question should be thoroughly inspected before any decisions are made about how to proceed. The objective of a detailed inspection is to gain knowledge of surface defects, material deterioration, deflection, and displacement of the structure, the drainage system, and surveillance equipment condition (Andrey 1987).

The inspection form described in Chapter 9 is the main document used for describing an inspection. In it, all the detected defects with their respective placement and ratings are

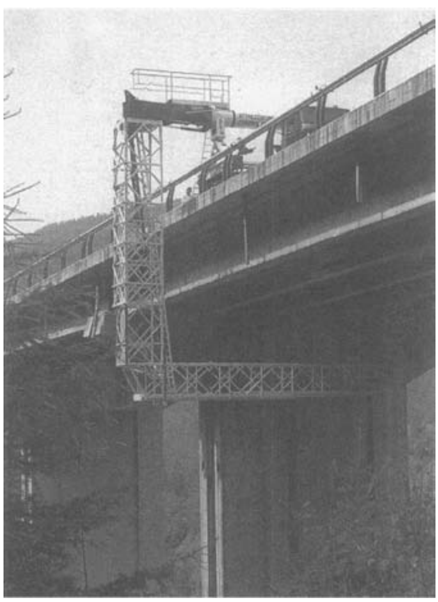

Figure 10-8. Special bridge inspection vehicle

recorded. A separate report must be prepared with the results of the in situ tests: sclerometer, electric potentials, ultrasound, bar cover measurements, cores, and so on. The results from the fixed surveillance equipment are summarized in another report. The item "Maintenance Work Needed" from the inspection form gives all the indications necessary for periodic maintenance (replacement of railings, bituminous pavement repair, cornice reconstruction, shrubbery removal, etc.). To facilitate the task of the person in charge of the next inspection, it is recommended that a list of the points that deserve special attention in the future or even continuous surveillance be prepared. All the data collected are dated and appended to the bridge dossier. According to the results obtained, the inspection may have one of the following consequences (Andrey 1987): the organization of a structural assessment or of complementary surveillance measurements; the preparation of a list with particular aspects to follow especially carefully in the next inspection; the organization of maintenance work needed; and the establishment of a medium-term maintenance plan.

10.2.3. Structural Assessment

A structural assessment is generally the result of the detection of a major structural or functional defect during a routine or detailed inspection. This type of inspection may also be necessary if strengthening the structure or widening the deck are under consideration. The results expected from this inspection are: the characterization of the structural defects, an estimation of the bridge's remaining service life (by using degradation mathematical models), and an estimate of its present load-bearing capacity. This inspection is very thorough but also relatively circumscribed. The means used are not easy to predict because a wide range of situations can lead to a structural assessment. However, it can be said that, potentially, all in situ diagnostic methods may be needed, even though careful budget control must be exercised.

According to its definition, this inspection is nonperiodic and cannot be planned in the long term. It is precipitated by the detection of defects that may jeopardize the structural safety of the bridge (Figure 10-9) (Santiago 2000) or bridge capacity fulfillment of the role for which it was designed. The inspection may also be brought about by nonstructural problems such as the ruinous condition of the asphalt surface, which is potentially capable of jeopardizing the safety or welfare of users. Finally, a structural assessment can be the means to control the global behavior of the structure after a rare event (passage of a heavy truck over the bridge, floods, earthquakes, traffic accident, etc.).

Even though it can be planned only in the short-term, the structural assessment is the inspection that must be prepared for most thoroughly to avoid the assignment of specialized personnel and very expensive equipment at the bridge for a period of time longer than necessary. Curtailing bridge traffic, even if partially and temporarily, however, is almost inevitable, and one reason for very careful planning, which can limit the resulting functional failure costs (Chapter 12). Safety measures for inspection personnel (MTRD 1979) and the allocation of special means of access are other functions of planning. One or more preliminary visits to the site are almost always necessary.

The inspection team is to be led by a graduate engineer who is an expert with a great deal of experience on bridges of the structural type to be inspected and the materials and construction techniques used. The remaining personnel are very specialized, depending on the tests predicted as necessary, and their number depends on the bridge dimensions and the depth of the required tests.

As discussed previously, the equipment necessary for a structural assessment may be nonportable, expensive, and difficult to move. For each situation, the range of recommended equipment is quite wide, which is why no specific list can be presented. Laboratory

Figure 10-9. Scour of a bridge foundation caused by soil running water

tests are also indispensable as a complement to the information collected in situ. In situations in which global structural behavior is being questioned, the static and dynamic load tests (Figure 10-10) are a valuable tool (Chapter 3). However, they must be used with some parsimony because, besides being expensive, they force the total interruption of traffic over the bridge for indeterminate periods of time. The special means of access that eventually will be necessary are discussed in the description of detailed inspections.

The procedures to be followed at the site are similar to those described for the detailed inspections. However, considering the great variety of situations that may arise (with many different causes and effects), it is not possible to standardize the sequence of actions to be

Figure 10-10. Bridge load test made with several lorries filled with sand

followed. The points to investigate are obviously dependent on previous periodic inspections. Contrary to what happened earlier, there are now hints and even certainties about the location and source of problems, which generally facilitate the survey.

The inspection form described in Chapter 9 applies fundamentally to periodic inspections and reserves very limited space for the structural assessment. Once again, this is due to the great variety of situations and measurements possible during a structural assessment, which preclude the existence of a universal standard form. In the Rehabilitation Manual (OMT 1988) of the Ontario Management System, several standard forms are presented for the most current tests [delamination, bar cover, corrosion potential, concrete core drilling, asphalt surface sample collection, laboratory tests with the in situ specimens, radar and thermography (DART)], likely to be adapted to any other management system. According to the same document, the structural assessment final report must be presented in the following order: index; structure identification standard form; simplified schematic drawing of the bridge; summary of the most significant results; structure general condition standard form; equipment used and calibration sheets; photos and schematic representations of the cores; identification and description of the cores; identification and description of the asphalt surface samples; photos of the samples; photos from the bridge location; and drawings. In the inspection form presented in Chapter 9, the most important item related to the structural assessment goes by the name "Repair Work Needed" and gives all the indications necessary to make a decision about whether to implement structural repair work (Chapter 13). All the data collected are dated and appended to the bridge dossier.

10.2.4. Bridge Initial Characterization

When a periodic inspection, either routine or detailed, is performed on a bridge, it is fundamental to compare the conclusions and measurements obtained with the respective values expected. The combination of structural and functional characteristics that define a bridge make up what has been defined as the reference state (MTRD 1979 and Andrey 1987), to which all inspections refer. It is not impossible to implement a regular inspection system without this initial characterization of each bridge. Even if the bridge is too small or insignificant to be subject to a periodic surveillance, its reference state must be created.

According to this definition, the inspection that defines the reference state is nonperiodic. In principle, the inspection should be performed when the bridge is turned over to the owner and after any minor repairs, which are builder's responsibility, are performed. This bridge characterization does not necessarily occur right after it is built. In situations in which the bridge has never been subjected to any inspection/maintenance plan, it becomes necessary to create a "reference state," very often without resorting to the structural design plans. Under these circumstances, it is necessary to survey the structure's dimensions and use in situ tests to identify and locate the existing reinforcement and its eventual active corrosion. It is also very useful to reach a conclusion concerning its load-bearing capacity, sometimes based on the construction date and valid codes. Characterization of the bridge may also be performed after strengthening/widening work of an existing structure has been completed. Under these circumstances, the surveyor's job generally becomes easier as a result of an efficient quality control for the work performed. However, it is not generally possible to dispense with the load tests in order to confirm unequivocally the rehabilitation efficiency.

Planning for this type of inspection for new bridges is made easier (when it coincides with the handing-over load tests) because there is no traffic. In the remaining cases, the traffic hindrance, which is inevitable, must be minimized, avoiding, whenever possible, the simultaneous blockage of all lanes. The final project drawings must be taken to the bridge site to register the detected changes and to prepare the as-built drawings. It is usually necessary to plan for the use of special means of access.

The inspection team composition is similar to that described for the structural assessment.

This inspection is based almost exclusively on direct visual observation of the structure outer surface. Nondestructive in situ tests (magnetometer, sclerometer, ultra-sounds, galvanic half-cell, structure leveling, etc.) are used. The object of these tests is to determine whether design/construction errors exist (e.g., geometric defects, imperfections due to unskilled workmanship, damage due to the load tests, etc.), visible or invisible, that require immediate action. Global structural capacity is investigated using static and dynamic load tests. Special means of access, which will eventually become necessary, are as described previously.

This inspection is a combination of the detailed inspection and the structural assessment. It is more thorough than the detailed inspection but it has in common with it its general nature, since the entire bridge is inspected, not just isolated areas. However, it resorts to tests that are used only in structural assessments. Therefore, the procedures at the site have some of the characteristics of a periodic inspection and other characteristics of a structural assessment.

The most important result of this inspection is the creation of the reference state, which is described in the reference state form described in Chapter 9. The basic design stage situation after the bridge has been completed and the minor repair work has been done by the contractor before the handing-over is described. The situation is defined by preliminary studies, the characterization of access roads to the bridge, the structural solution, and the design detail regarding traffic flow. In the post-construction situation, the changes from the design stage, the tests performed during construction or at its completion, and the real traffic on the bridge are described. Additionally, the as-built drawings and schematic drawings of all the exposed surfaces are prepared, using a scale that makes the drawings easy to handle and yet large enough so that information about the defects detected during future inspections can be inserted. All of the schematics must have a reference grid (discussed later in this chapter).

10.2.5. General Flowchart of the Inspection System

Figure 10-11 (de Brito 1992) presents the general flowchart of an inspection system. On the left side, the basic input elements for each type of inspection are presented, the locations at which inspection results are used and/or stored are shown on the right side. The decision system described in Chapters 11 and 13 is also represented to clarify the influence of the inspection system in it.

10.3. Classification System

In order to standardize inspection reports and forms, as well as procedures both at the bridge site and at headquarters, it is fundamental to create a classification system of all defects that may be detected in concrete bridges. This classification system is the only way to avoid having the same event described in different ways by different inspectors or a situation in which identical classifications describe defects that are not identical. Computerized inspection systems are based on information entered into the inspection forms, which therefore must be authentic, precise, and unequivocal.

The need to create a coherent classification extends to the possible causes of defects, repair techniques, and diagnostic methods.

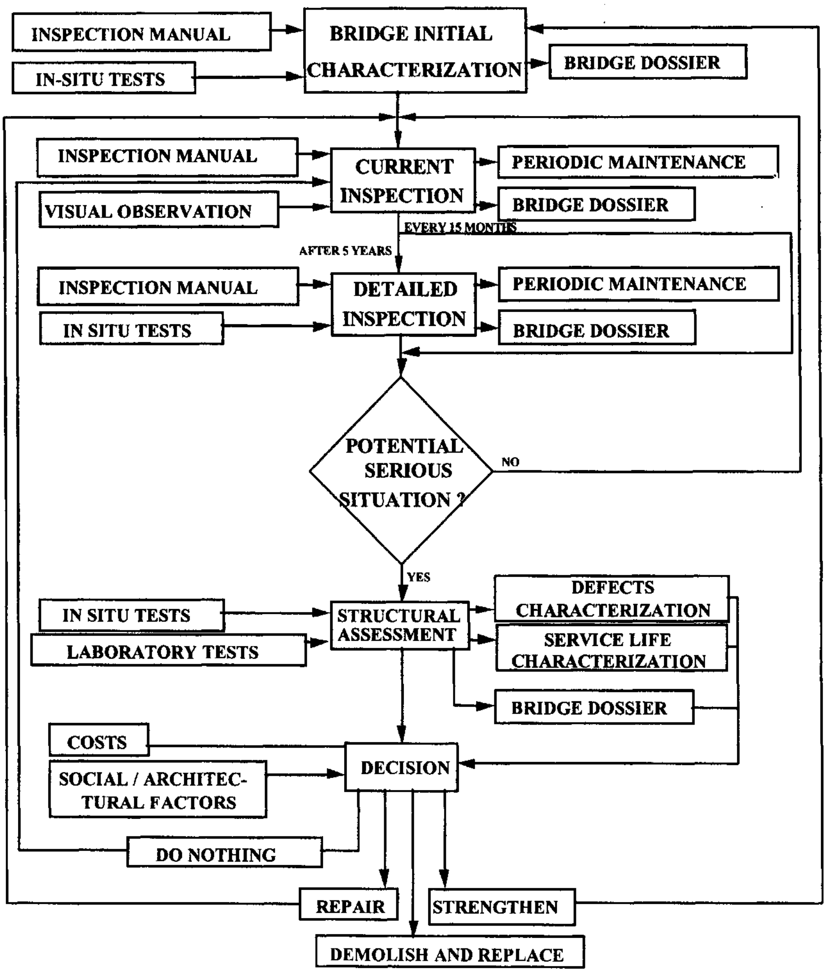

Figure 10-11. Inspection system

Defects in concrete structures have already been classified according to very different criteria: as a function of the location in the structure (infrastructure, superstructure, etc.); as a function of the importance of the structural elements in which they occur (main elements, secondary elements, etc.); as a function of the materials type (concrete, steel, asphalt, etc.); and as a function of probable causes (defects related to corrosion, others related to earthquakes, etc.).

10.3.1. Defects

All defects likely to be found in concrete structures (totaling 94 entries) were classified into nine different groups (de Brito et al. 1994).

The classification system proposed here (Table 10-1) (de Brito 1992) is based on two other classification systems presented previously (Andrey 1987 and de Brito 1987). The criterion used was basically of a location and functional nature (de Brito et al. 1994): the foundations (Figure 10-9) (Santiago 2000)/abutments/embankments are referred to as

Table 10-1. Defects of concrete bridges

	A-A. Superstructure Global Behavior		
A-A1	Permanent deformation (Figure 10-12)	A-A3	Columns tilting
A-A2	Relative displacement	A-A4	Vibration
	A-B. Foundations/Abutments/Embankments		
A-B1	Scour (Figure 10-9)	A-B6	Embankment slippage
A-B2	Settlement	A-B7	Heavy vegetation growth/burrows
A-B3	Rotation	A-B8	Obstruction of the waterway by debris
A-B4	Settlement/failure of the approach slab	A-B9	Silting
A-B5	Embankment erosion		
	A-C. Concrete Elements		
A-C1	Corrosion stain	A-C7	Delamination/spalling (Figure 10-13)
A-C2	Efflorescence/moisture stain	A-C8	Concrete crushing
A-C3	Concretion/swelling	A-C9	Map cracking
A-C4	Wear/scaling/disintegration	A-C10	Longitudinal crack
A-C5	Voids/porous area/honeycombing/	A-C11	Transverse crack
	aggregates nest	A-C12	Diagonal crack
A-C6	Stratification/segregation	A-C13	Crack over/under a bar
	A-D. Reinforcement/Cables		
A-D1	Exposed bar (loss of cover) (Figure 10-14)	A-D6	Broken bar
A-D2	Exposed duct (loss of cover)	A-D7	Broken strand/wire
A-D3	Exposed strand/wire (loss of cover)	A-D8	Deficiently grouted duct
A-D4	Corroded bar	A-D9	Faulty sealing of prestress anchorage
A-D5	Bar with reduced cross-section	A-D10	Corroded anchorage
	A-E. Bearings		
A-E1	Obstruction due to debris/	A-E8	Detachment/failure of anchor bolts/pins
	vegetation growth	A-E9	Lead crushing
A-E2	Obstruction due to rust	A-E10	Elastometer creep
A-E3	Broken retainer-bars	A-E11	Elastometer crushing
A-E4	Cracked roller	A-E12	Bearing displacement (Figure 10-16)
A-E5	Roller failure	A-E13	Failure of the bearing seat
A-E6	Corrosion	A-E14	Moisture/trapped water
A-E7	Deteriorated base plate/pot		
	A-F. Joints		
A-F1	Vertical misalignment	A-F7	Detachment/failure of anchorages
	(traffic shock action) (Figure 10-15)	A-F8	Loosening/failure of bolts/pins
A-F2	Loss of parallelism	A-F9	Cracking of the metallic components
A-F3	Transverse shear cut	A-F10	Displacement/failure/deterioration of the
A-F4	Obstruction due to debris/		vegetation growth
A-F5	Obstruction due to rust		Filler/sealant
A-F6	Corrosion	A-F11	Moisture/trapped water
	A-G. Wearing Surface (Asphalt)/Watertightness		
A-G1	Map/alligator cracking	A-G7	Wheel track rutting
A-G2	Crack along a repaired patch edge	A-G8	Rippling
A-G3	Other cracks	A-G9	Loss of bond/delamination
A-G4	Gravel pocket	A-G10	Flushing
A-G5	Pothole	A-G11	Missing/torn waterproof membrane
A-G6	Ravelling		
	A-H. Water Drainage		
A-H1	Deposit of water (Figure 10-17)	A-H5	Obstructed deck drain
A-H2	Obstructed curb gutter	A-H6	Drain discharging directly on structural
A-H3	Gutter joint leakage		elements
A-H4	Gutter cross-section reduction	A-H7	Lack of drainage in void cross-sections
	A-I. Secondary Elements		
A-I1	Inadequate/inexistent traffic signs	A-I9	Broken welding
A-I2	Deteriorated traffic signs	A-I10	Damaged/eroded sidewalks
A-I3	Inexistent curbs/traffic barrier wall	A-I11	Damaged utilities
A-I4	Damaged curbs/traffic barrier wall	A-I12	Inadequate/inexistent lighting
A-I5	Damaged hand railing (Figure 10-18)	A-I13	Lighting out of order
A-I6	Defective coating	A-I14	Deteriorated edge beams
A-I7	Corrosion	A-I15	Damaged acroterium
A-I8	Loosening/failure of bolts/pins		

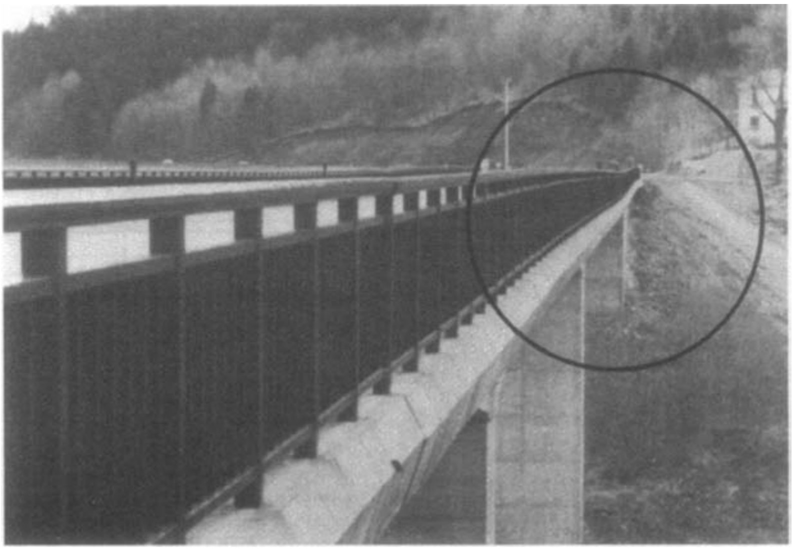

Figure 10-12. Defect A-A1: Permanent deformation

one group, the joints in another group, the bearings in yet another group, and so on. It appeared from the beginning that there was a need to create a separate group, A-A, superstructure global behavior (Figure 10-12) (MTRD 1979), in order to include defects that influence that global behavior.

Groups A-C, concrete elements (Figure 10-13) (Santiago 2000) and A-D, reinforcement/cables (Figure 10-14) (Santiago 2000) have a much wider scope: they allow the rating of each defect, in concrete and ordinary or prestressed steel, respectively, independent of

Figure 10-13. Defect A-C7: Delamination/spalling

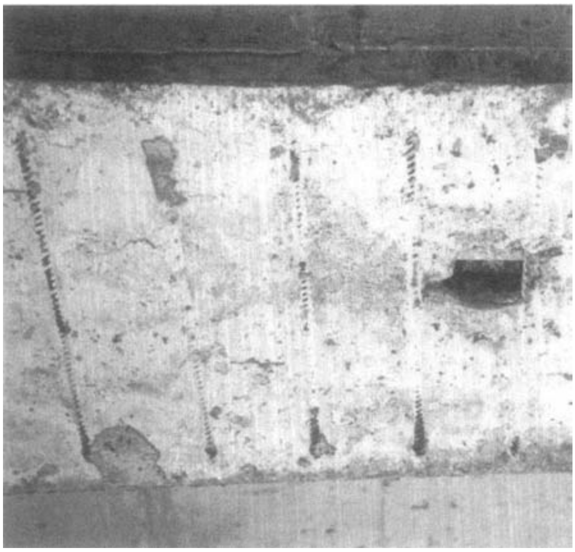

Figure 10-14. Defect A-D1: Exposed bar (loss of cover)

location. It is thus possible to avoid repeating these defects for all of the specific elements included in the other groups (sidewalks, foundations, edge beams, etc.). If, for example, a case of spalling of an edge beam is detected, this defect must be classified as A-C7 (delamination/spalling) and not as A-I14 (edge beams deterioration).

This list of defects endeavors to cover any defect susceptible to detection in a bridge with a structure made entirely of reinforced and prestressed concrete. To achieve that goal, special groups were created for joints (Figure 10-15) (Reel and Conte 1989) and bearings (Figure 10-16) (Reel and Conte 1989), secondary elements, and so forth, which are not specific to concrete bridges but are essential to its normal functioning. These groups may be

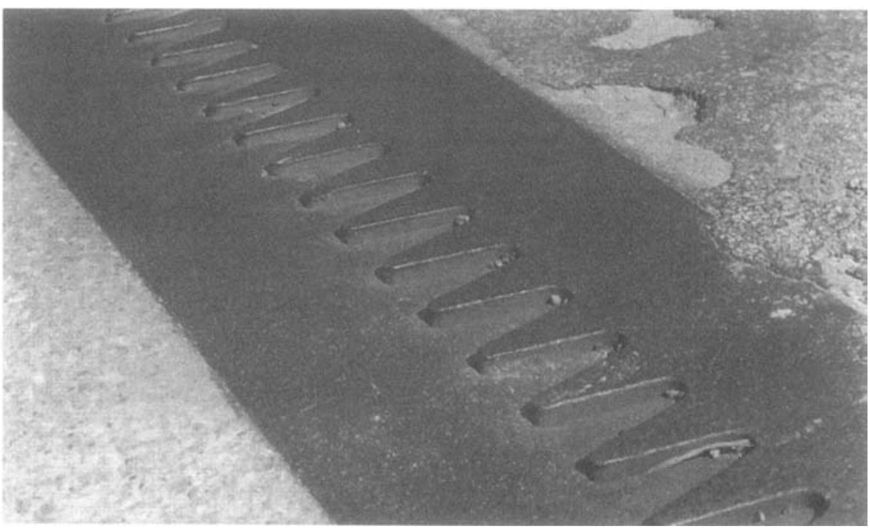

Figure 10-15. Defect A-F1: Vertical misalignment (traffic shock action)

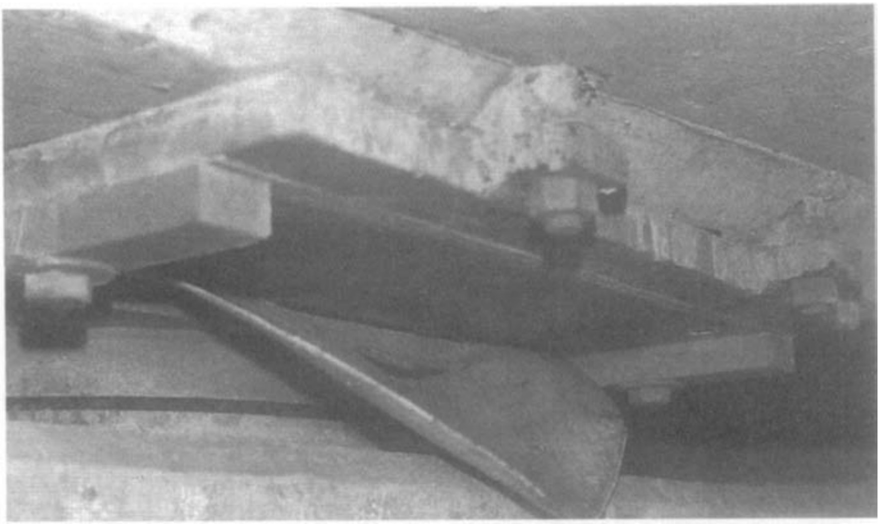

Figure 10-16. Defect A-E12: Bearing displacement

used directly in any classification system that may be devised for steel or composite steel and concrete bridges.

The classification system also endeavors to avoid redundancy as well as defects that cannot easily be included in any category. In some cases, it is necessary to resort to the defect form (discussed later in this chapter) in order to clarify the differences between similar defects (e.g., A-D4 corroded bar and A-D5 bar with reduced cross section). In that form, the rating criteria, concerning the seriousness of a defect in terms of its extension and degree of evolution, must be given.

Within each group, the defects were aligned approximately in terms of their similitude, common cause, and/or proximity, in order to facilitate the use of the list presented in Table 10-1 (de Brito 1992).

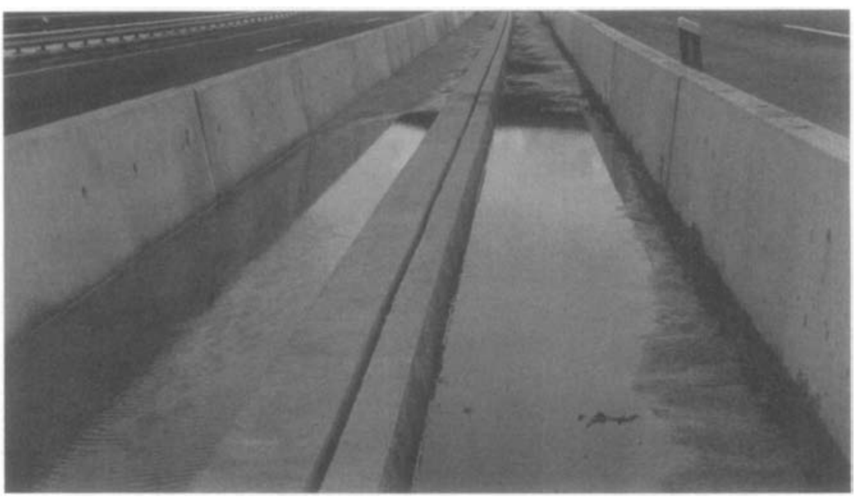

Figure 10-17. Defect A-H1: Deposit of water

Figure 10-18. Defect A-I5: Damaged hand railing

10.3.2. Possible Causes of the Defects

All possible causes (direct or indirect) of these defects (117 entries) were then classified according to chronological criteria and classified into nine different groups (de Brito et al. 1994).

It is common to find in technical references (CEB 1985 and Cánovàs) classification lists of causes of defects in concrete bridges. In most cases, the causes are classified in terms of their relation to design, construction, service, aggressive factors, and accidental actions. However, most classification systems do not list every error and circumstance that lead to a defect in a concrete structure and are often not specific enough about some of the subjects. It is also difficult to find a classification system for the cause of defects (specifically for bridges), either in concrete or in other structural materials.

The classification proposed here (Table 10-2) (de Brito 1992) is based on two classification systems presented previously (Reel and Conte 1989 and de Brito 1987). The criterion used was basically chronological: design errors precede construction errors, which themselves precede in-service accidental/environmental actions as well as aggressive factors.

Special emphasis was given to groups C-A, design errors (Figure 10-19) (Santiago 2000), and C-B, construction errors, because these are the main causes of defects in concrete structures, both bridges and buildings. Accidental actions, some of which are quite improbable but which are capable of causing significant and widespread damage, were divided into natural actions (group C-C) (Figure 10-20) (Reel and Conte 1989) and human-caused accidents (group C-D) (Figure 10-21) (Santiago 2000).

A group (C-E) (Figure 10-22) (Reel and Conte 1989) was dedicated to environmental actions that seemingly were quite harmless as isolated causes of deterioration. However, it should be noted that, in the vast majority of the defects detected in concrete, several causes can be singled out and only rarely does one of them assume major importance. Aggressive factors were also divided into natural factors (group C-F) (Figure 10-23) (Reel and Conte 1989) and human-caused factors (group C-G) (Figure 10-24) (MTRD 1979).

Table 10-2. Possible causes of defects of concrete bridges

	C-A. Design Errors		
C-A1	Deficient layout of the bridge or its approaches	C-A17	Deficient metallic connections design/detailing
C-A2	Deficient hydraulic design	C-A18	Deficient bearings design/positioning
C-A3	Wrong choice of materials	C-A19	Deficient joints design/positioning
C-A4	Wrong/missing design loads	C-A20	Excessive exposed areas in structural elements/faulty geometric design
C-A5	Over-simplified structural modelling	C-A21	Inability to predict the replacement of heavily deteriorated elements
C-A6	Missing temperature effects on long or skewed decks	C-A22	Difficulty/impossibility of inspection of parts of the structure
C-A7	Non-consideration of long-term effects in the design of vertical elements	C-A23	Non-prevision of a minimum slope in quasi-horizontal surfaces
C-A8	Non-consideration of instability effects in the design of vertical elements	C-A24	Drainage directly over concrete, a joint, a bearing or an anchorage (Figure 10-19)
C-A9	Non-consideration of the building process	C-A25	Other drainage design faults
C-A10	Wrong seismic/horizontal loads design	C-A26	Lack of waterproofing membrane
C-A11	Non-detected computer analysis mistakes	C-A27	Deficient design specifications
C-A12	Deficient foundation modelling	C-A28	Incomplete/contradictory/over-compact drawings
C-A13	Deficient scour design/protection		
C-A14	Insufficient reinforcement/prestress design cover		
C-A15	Inadequate reinforcement/prestress spacing		
C-A16	Other reinforcement/prestress detailing errors		
	C-B. Construction Errors		
C-B1	Wrong interpretation of the drawings	C-B13	Deficient grouting of prestress cables ducts
C-B2	Inexperienced personnel	C-B14	Early/faulty demoulding
C-B3	Deficient soil compaction/stabilization	C-B15	Premature loading
C-B4	Deficient materials transport/storing	C-B16	Faulty patching
C-B5	Changes in prescribed materials mixing proportions	C-B17	Faulty placing of waterproofing membrane
C-B6	Use of inappropriate materials (contaminated water, over-reactive aggregates)	C-B18	Deficient asphalt paving/repaving of the deck
C-B7	Faulty casting	C-B19	Faulty asphalt patching
C-B8	Overuse of formwork/faulty formwork	C-B20	Obstruction of drains with asphalt
C-B9	Deficient concrete compaction/curing	C-B21	Faulty bolt/pin tightening
C-B10	Cold joint	C-B22	Defective welding
C-B11	Inaccurate reinforcement/prestress positioning/detailing	C-B23	Faulty coating
C-B12	Inadequate prestressing	C-B24	Faulty construction/placing of joints
		C-B25	Deficient placing of bearings
		C-B26	Insufficient/inexistent quality control
	C-C. Natural Accidental Actions		
C-C1	Earthquake	C-C6	Snow avalanche
C-C2	Fire	C-C7	Tornado/cyclone
C-C3	Downpour	C-C8	Tsunami
C-C4	Flood (Figure 10-20)	C-C9	Thunderbolt
C-C5	Earth sliding	C-C10	Volcano eruption
	C-D. Man-Caused Accidental Actions		
C-D1	Fire	C-D4	Overload
C-D2	Collision/traffic accident (Figure 10-21)	C-D5	Heavy objects dropped
C-D3	Explosion/bombing	C-D6	Vandalism

(Continued)

Table 10-2, Continued. Possible causes of defects of concrete bridges

	C-E. Environmental Actions		
C-E1	Temperature	C-E5	Ice (freeze/thaw cycles) (Figure 10-22)
C-E2	Humidity (wet/dry cycles)	C-E6	Wind
C-E3	Rain	C-E7	Direct solar radiation
C-E4	Snow		
	C-F. Natural Aggressive Factors		
C-F1	Water (wet/dry cycles)	C-F7	Alkali-aggregate reaction (Figure 10-23)
C-F2	Carbon dioxide	C-F8	Abrasion (wind, sand, heavy objects suspended in a stream)
C-F3	Salt/salt water (chlorides)		
C-F4	Acids/soft water	C-F9	Cavitation
C-F5	Ammonium/magnesium salts	C-F10	Biological action (algae, lichen, roots)
C-F6	Sulphates	C-F11	Evaporation of volatile components
	C-G. Man-Caused Aggressive Factors		
C-G1	Water	C-G6	Abrasion (traffic, transport of materials)
C-G2	Carbon dioxide	C-G7	Cavitation (Figure 10-24)
C-G3	De-icing salts	C-G8	Biological action (sewers) properly still in service
C-G4	Pollution		
C-G5	Organic compounds (sugar, oils)		
	C-H. Lack of Maintenance		
C-H1	Accumulation of rust/debris in the bearings	C-H5	Gutter/drains obstructed by debris
		C-H6	Lack/loosening of pins/bolts
C-H2	Bearings (or components of) not functioning properly still in service	C-H7	Defective metallic coatings
		C-H8	Heavy vegetation growth/burrows (Figure 10-25)
C-H3	Accumulation of rust/debris in the joints		
C-H4	Joints (or components of) not functioning properly still in service		
	C-I. Changes from Initially Planned Normal Use		
C-I1	Changes upstream/downstream in the channel/stream layout	C-I7	Inappropriate/missing lighting
		C-I8	Foundations settlement
C-I2	Heavy increase in traffic flow	C-I9	Closing of joints
C-I3	Increase in maximum allowed load (Figure 10-32)	C-I10	Changes in the span distribution
		C-I11	Abnormal functioning of the bearings
C-I4	Increase of the dead load due to repeated repaving	C-I12	Strengthening works of certain elements but not all the necessary
C-I5	Excessive traffic speed	C-I13	Change in codes (live loads, seismic action)
C-I6	Inappropriate/missing signs		

A special group (C-H) is based on a lack of maintenance as a cause of defects. The list of causes in this group followed the criterion that they must by themselves cause the defects either directly or indirectly. Some of the causes listed are defects as well, but they do have further consequences (Figure 10-25) (MTRD 1979).

Finally, group C-I (Figure 10-26) (Santiago 2000) concerns situations in which changes from the initial planning for normal use of a concrete bridge are likely to create functional problems in the future. In most classification systems, these situations are incorrectly ignored, taking into account, for example, the rate at which traffic demand changes.

Within each group and to facilitate an understanding of Table 10-2 (de Brito 1992), causes with similar characteristics are listed sequentially.

Figure 10-19. Drainage directly over concrete due to a design error in the drainage system (cause C-A24)

10.3.3. Repair Techniques

Repair techniques used to eliminate or prevent the defects listed previously (69 entries) were classified in the same groups as the defects (de Brito et al. 1994).

There are many publications concerning the repair/rehabilitation of concrete structures (de Brito 1987, CEB 1983 and de Brito 1988). However, it is not as easy to find proposals of classification systems for such repairs. When available, they do not apply specifi-

Figure 10-20. Stream obstructed by debris and siltation after a flood (cause C-C4)

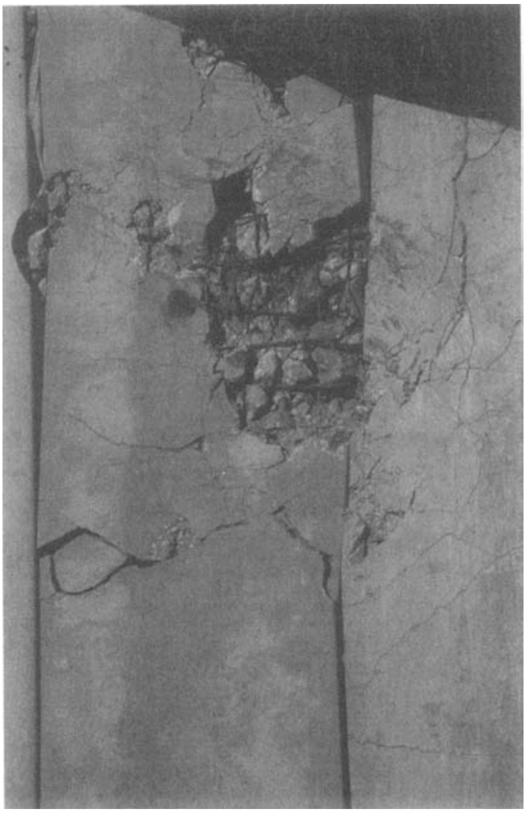

Figure 10-21. Broken concrete element due to a lorry collision (cause C-D2)

Figure 10-22. Severe scaling of concrete surface due to freeze/thaw cycles (cause C-E5)

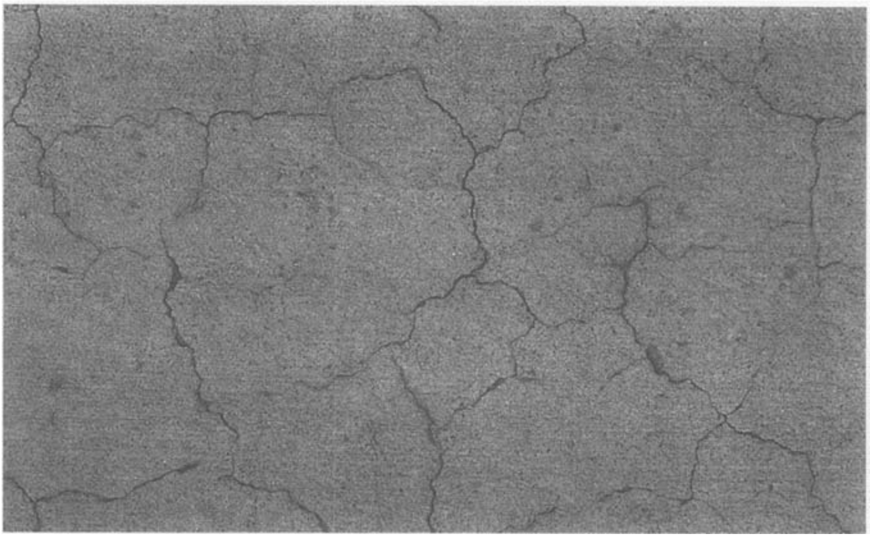

Figure 10-23. Map cracking due to severe alkali-aggregate reaction (cause C-F7)

cally to bridges and do not take into account the multitude of works that must be undertaken to keep the bridges both functional and structurally safe. In fact, most of the work done on a bridge after it has been built concerns maintenance, not rehabilitation. Also, the work predominantly concerns functional aspects, not any structural repairs.

Taking these facts into consideration, the classification presented here (Table 10-3) (de Brito 1992) includes, in addition to repair techniques, maintenance work. To clarify this division between maintenance and repair when used in the decision system (Chapters 11 and 13), each technique listed relates to only one management module. Therefore, a repair technique whose classification is followed in the global list by (*m*) must be included in maintenance and the associated costs must be included in maintenance costs, C_M (see Chapter 12), independent of other circumstances. The reverse happens for techniques with (*r*) after its classification,

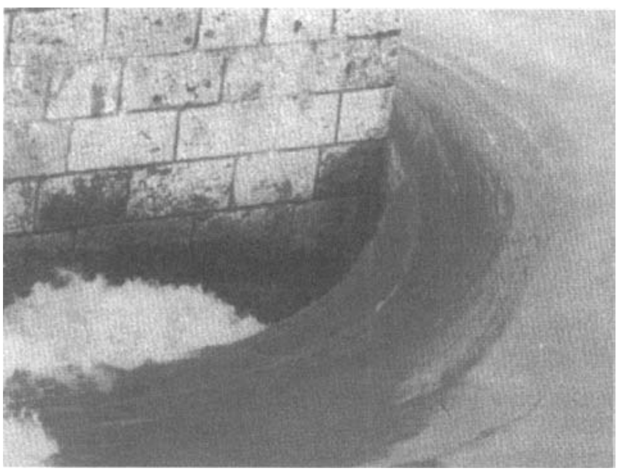

Figure 10-24. Foundation scour due to man-caused cavitation (cause C-G7)

Figure 10-25. Noxious vegetation in the sidewalks borders due to lack of maintenance (cause C-H8)

Figure 10-26. Settlement of the approach slab due to an increase in the maximum allowed load (cause C-I3)

Table 10-3. Repair (**r**) and maintenance (**m**) techniques of defects of concrete bridges

	R-A. Superstructure Global Behavior		
R-A1	Release of internal/external connection (**r**)	R-A3	Building a span support (new column) (**r**)
R-A2	Restraint of internal/external connection (**r**)	R-A4	Additional exterior prestress (**r**)
	R-B. Foundations/Abutments/Embankments		
R-B1	Scour repair (wedge foundations using calibrated material) (**r**)	R-B5	Replacement of the approach slab (**r**)
R-B2	Scour prevention (hydrodynamic protections, islet construction (**r**)	R-B6	Embankment consolidation (**r**)
R-B3	Foundation consolidation (jacking up and compaction) (**r**)	R-B7	Removal of accumulated debris/vegetation growth/burrows (**m**) (Figure 10-5)
R-B4	Soil compaction under approach slab (**r**)	R-B8	Removal of silting (**m**)
	R-C. Concrete Elements		
R-C1	Cosmetic repair (**m**)	R-C5	Crack sealing (**r**)
R-C2	Concrete patching (with deteriorated concrete removal) (**r**)	R-C6	Crack stapling (**r**)
R-C3	Crack injection (**r**)	R-C7	Concrete refacing/encasing (**r**)
R-C4	Crack grouting (**r**)	R-C8	Partial/global replacement (**r**)
	R-D. Reinforcement/Cables		
R-D1	Concrete patching (with reinforcement/prestress cleaning) (**r**) (Figure 10-27)	R-D6	Additional/replacement of prestress (**r**)
R-D2	Concrete patching (with reinforcement splicing/replacement) (**r**)	R-D7	Grouting of void ducts (**r**)
R-D3	Concrete encasing (with reinforcement splicing/replacement) (**r**)	R-D8	Corrosion treatment and sealing of anchorage (**m**)
R-D4	Glued steel plates (**r**)	R-D9	Anchorage repair with transverse reinforcement (**r**)
R-D5	Incorporated steel profiles (**r**)	R-D10	Replacement of anchorage(**r**)
	R-E. Bearings		
R-E1	Removal of debris/moisture/trapped water/vegetation growth (**m**) (Figure 10-6)	R-E6	Replacement of the anchor bolts/pins (**r**)
R-E2	Replacement of the retainer-bars (**r**)	R-E7	Replacement of the lead (**r**)
R-E3	Replacement of the roller (**r**)	R-E8	Replacement of the elastometer (**r**)
R-E4	Blast cleaning/coating (**m**)	R-E9	Concrete patching of the bearing seat (**r**)
R-E5	Replacement of the base plate/pot (**r**)	R-E10	Repositioning of the bearing (**r**)
		R-E11	Replacement of the bearing (**r**)
	R-F. Joints		
R-F1	Removal of debris/moisture/trapped water/vegetation growth (**m**)	R-F4	Replacement/tightening of bolts/pins (**r**)
R-F2	Blast cleaning/coating (**m**)	R-F5	Replacement of the filler/sealant (**r**) (Figure 10-28)
R-F3	Replacement of the anchorages (**r**)	R-F6	Replacement of the joint (**r**)
	R-G. Wearing Surface (Asphalt)/Watertightness		
R-G1	Localized patching (**m**)	R-G4	Latex modified concrete overlay (**m**)
R-G2	Waterproofing and asphalt repaving (**m**)	R-G5	Concrete overlay, waterproofing and asphalt repaving (**m**)
R-G3	Patching, waterproofing and asphalt repaving (**m**)	R-G6	Cathodic protection (**m**)

(*Continued*)

Table 10-3, Continued. Repair (**r**) and maintenance (**m**) techniques of defects of concrete bridges

	R-H. Water Drainage		
R-H1	Removal of debris/obstructing asphalt from deck drain or gutter (**m**)	R-H5	Installation of new deck drains/ void tubes (**m**)
R-H2	Gutter joint repair (**m**)	R-H6	Replacement of drain/gutter/void tubes (**m**)
R-H3	Deck drain extension downwards/ upwards (**m**)		
R-H4	Diversion of point of discharge of deck drain (**m**)		
	R-I. Secondary Elements		
R-I1	Installation/replacement of traffic signs (**m**)	R-I7	Replacement of sidewalks (**m**)
R-I2	Installation/replacement of curbs/ traffic barrier wall (**m**)	R-I8	Replacement of utilities (**m**)
		R-I9	Installation/replacement of lighting (**m**)
R-I3	Replacement of hand railing (**m**)	R-I10	Replacement of edge beams (**m**)
R-I4	Blast cleaning/coating (**m**)	R-I11	Replacement of acroterium (**m**)
R-I5	Replacement/tightening of bolts/pins (**m**)	R-I12	Removal of vegetation growth (**m**)
R-I6	Welding repair (**m**)		

whose costs are included in the repair costs, C_R. The fundamental concept here is that the techniques included in maintenance do not concern structural aspects of the bridge.

When preparing this list, consideration was given to the fact that repairs have more to do with the defects themselves than with what caused them. Although very often the repair has to take into account the cause of the defect, it is the actual defect that concerns bridge management. Thus it was decided that there should be a close parallel relationship between

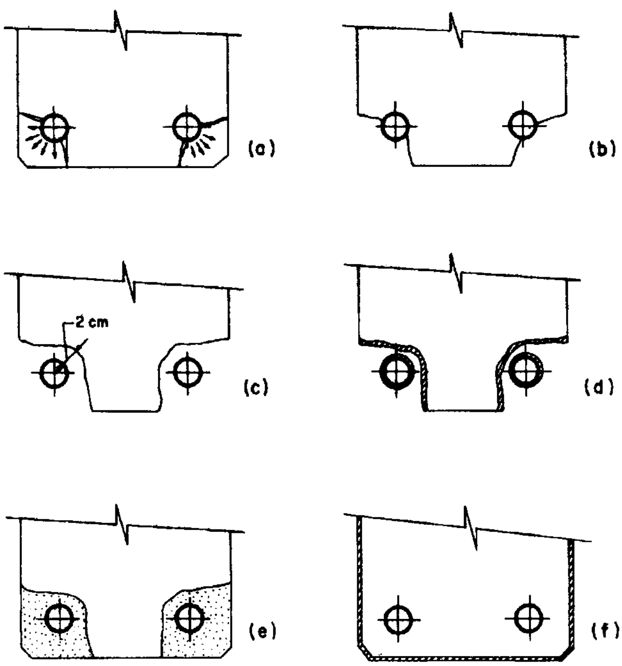

Figure 10-27. Repair steps of a beam damaged by reinforcement corrosion (repair technique R-D1)

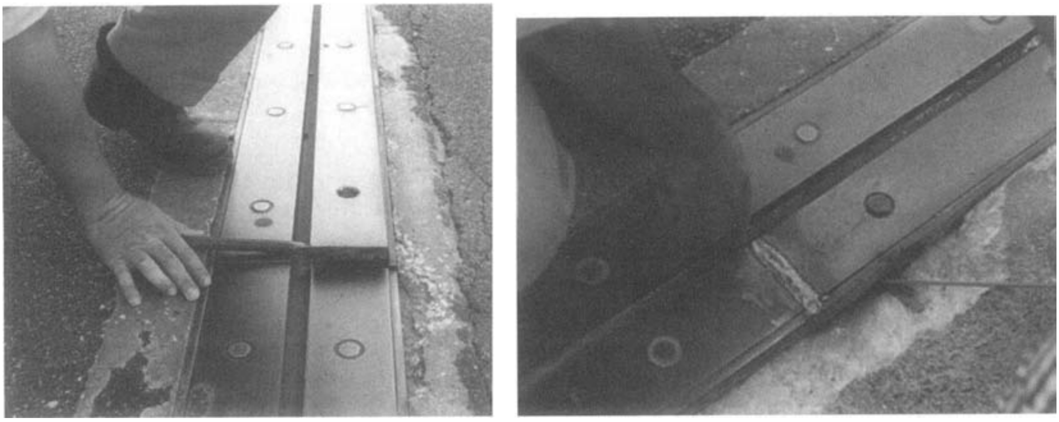

Figure 10-28. Replacement of the filler/sealant of a joint (repair technique R-F5)

classification of repairs and classification of defects described before. The criteria used in the elaboration of the defect classification also apply to repair classification and are not repeated here. The specific criteria concerning the repairs are discussed next.

For each entry on the repairs list, a repair form must be created in which the first step is, whenever possible, the elimination of the cause(s) of the defect. The repair is thus dependent on the cause of the defect. In each form, a list of the work to be done as well as the need for materials, equipment, and workmanship are given. A list of unit costs is also fundamental to estimate the cost of possible future repairs and to choose from different solutions. The repair form is described in more detail later in this chapter.

Within each group and in order to facilitate the understanding of Table 10-3 (de Brito 1992), repair techniques with similar characteristics were listed sequentially.

10.3.4. Diagnostic Methods

There are a huge number of available testing methods, even if they are restricted to the nondestructive tests (Chapter 3). In Iffland and Birnstiel (1993), four tables are provided, in which the application, limitations, and references concerning each method are outlined:

- methods for determining physical conditions and properties of steel;
- methods for determining chemical and physical properties of concrete;
- methods for determining physical conditions of concrete;
- methods for physical conditions of reinforcing steel, pre-tensioning steel, and post-tensioning steel.

The in situ diagnostic methods used to detect or analyze the defects (81 entries) were also classified in 14 different groups (de Brito et al. 1994).

Before starting any diagnostic work on a bridge where defects have been detected, it is necessary to define the problems clearly. The symptomatology gives an indication of the route to follow. It is a waste of time and money to start diagnosis without knowing what information one needs.

The experimental methods used for rating defects in concrete bridges vary significantly in cost, equipment used, information collected, and necessary know-how and workmanship.

The knowledge of what needs to be investigated usually reduces the choice to a handful of methods.

When the method(s) is selected, the necessary know-how, equipment, manpower, and facilities must be gathered. Accurate information about the procedure is required, and the necessary information must be recorded to avoid multiple visits to the site. Diagnostic work is usually disruptive for the normal functioning of the bridge and therefore must be limited as much as possible in time and space. Interpreting the results can be a frustrating task, as very often experimental results are confusing, appear contradictory, and do not follow highly mathematical and theoretical patterns. Faced with such results, the person in charge has to root out the dubious results (and try to find an explanation for them), confirm the logical results, and, if necessary, select the information to be collected during the next visit. Laboratory work, although useful to clarify some points, must be avoided because it is generally both time-consuming and expensive.

The limitations of each method must be known: sometimes the accuracy of some methods does not permit anything but a qualitative diagnosis. At the present level of knowledge in the field of diagnostics, very few (and usually expensive) methods can return quantitative information that is reliable to any acceptable degree.

Faced with the great variety of existing methods and information collected by each method, it is difficult to define a practical global classification. The most current classification is characterized in terms of the amount of damage the methods produce in the element/structure analyzed (nondestructive, semidestructive and destructive tests). Some authors choose to classify the tests based on the main principle used (electrical, acoustic, magnetic, mechanical, etc.) or on the results obtained (geometric measurements, resistances, deformations, etc.).

In Table 10-4 (de Brito 1992), a classification proposal, based on the technique employed and the type of results provided, is presented. Most laboratory tests have been excluded from the list. Diagnosis must depend fundamentally on in situ testing and, whenever possible, an in situ interpretation of the results. The list is as thorough as possible and includes the latest developments in this field (i.e., tests with an interesting potential but have not yet proved worthwhile in situ).

This list is the result of several years of research and consultation, including access to many references, but only Andrey (1987), Malhotra (1976), and Jávor (1990) were included in the reference list.

Faced with a defect detected, it is necessary to select the method (or methods) that is more adequate for diagnosis. The list presented previously may be more confusing than useful because of its thoroughness and, therefore, it is necessary to rate the methods in terms of usefulness at the actual site. The characteristics that should be sought in a diagnostic method are (Andrey 1987 and de Brito and Branco 1990):

A low cost;

B easy and fast in situ performance;

C great amount of useful information;

D easy interpretation of results;

E nondestructiveness;

F portability of equipment;

G no source of energy necessary (or energy easily accessible in situ);

H no overspecialized workmanship or know-how;

Table 10-4. Diagnosis methods for concrete bridges

	M-A. Direct Visual Observation		
M-A1	Unaided (except for rulers, calibrated wedges, scales, callipers, a watch and other day-to-day equipment)	M-A3	Using endoscope
		M-A4	Using special means of aerial access
		M-A5	Underwater/on water
M-A2	Using telescopes, binoculars, micrometer, camera or video equipment (Figure 7-19)		
	M-B. Mechanical Techniques		
M-B1	Surface hammering/chain dragging	M-B4.3	Internal fracture method (or BRE test)
M-B2	Sclerometer (Figure 10-2)	M-B4.4	Expanding sleeve concrete test (ESCOT)
M-B2.1	Schmidt/rebound/impact/Swiss hammer test	M-B4.5	Anchorage system pull-out
		M-B4.6	Pull-off test
M-B2.2	Williams testing pistol	M-B4.7	Pull-out after penetration
M-B2.3	Frank spring hammer	M-B5	Core test (Figure 10-30)
M-B2.4	Einbeck pendulum hammer	M-B5.1	In compression and tension
M-B3	Penetration test	M-B5.2	For abrasion and freeze-thaw resistance
M-B3.1	Windsor probe test		
M-B3.2	Simbi hammer	M-B5.3	For microscopic and photographic means
M-B3.3	Spit pins	M-B5.4	For density and water absorption measurements
M-B3.4	Nasser pins		
M-B3.5	Drill penetration time	M-B5.5	For chemical studies
M-B4	Pullout test	M-B5.6	For special durability studies
M-B4.1	Conventional	M-B5.7	Break-off test
M-B4.2	Capo test (or Lok test)	M-B5.8	Cast-in-place cylinder test
	M-C. Potential Differences Measurement		
M-C1	Galvanic cell test (Figure 10-7, left)	M-C1.2	Multi-cell equipment
M-C1.1	Copper-copper sulphate half-cell		
	M-D. Magnetic Techniques		
M-D1	Magnetometer/covermeter/pachometer (Figure 10-29)	M-D2	Microwave absorption method
		M-D3	Disturbance of a magnetic field
	M-E. Electrical Methods		
M-E1	Conductance method	M-E3	Relative electrical resistivity method
M-E2	Absolute electrical resistivity method		
	M-F. Ultrasonic and Electromagnetic Techniques		
M-F1	Ultrasonic pulse velocity test (Figure 10-6)	M-F4	Mechanical sonic pulse velocity method
M-F2	Resonance method	M-F5	Radar
M-F3	Pulse echo method		
	M-G. Radioactive Methods		
M-G1	X-Ray propagation	M-G3	Radiation attenuation method (tomographic system)
M-G2	Gamma ray propagation	M-G4	Neutron emission
	M-H. Acoustic Techniques		
M-H1	Acoustic signs emission by loading	M-H2	Acoustic signs emissions during corrosion

(Continued)

Table 10-4, Continued. Diagnosis methods for concrete bridges

	M-I. Thermic Methods		
M-I1	Concrete maturity measurement	M-I3	Thermoelastic stress analysis
M-I2	Infrared thermography		
	M-J. Force/Deformation Techniques		
M-J1	Dynamometer	M-J9	Vibro-wire gauges
M-J2	Mechanical extensometers	M-J10	Stressmeters
M-J3	Electric extensometers	M-J11	Slot cutting
M-J4	Load cells	M-J12	Photoelastic coating technique
M-J5	Displacement transducers	M-J13	Moiré photography
M-J6	Clinometers	M-J14	Holography
M-J7	Infrared/laser rays	M-J15	Photogrammetry
M-J8	Water levelling		
	M-K. Chemical Indicators		
M-K1	Phenolphthalein (Figure 10-31)	M-K3	Rapid chloride test
M-K2	Silver nitrate	M-K4	Rapid alkali test
	M-L. Fluorescence Methods		
M-L1	Microscopic fluorescence		
	M-M. Load Tests		
M-M1	Deflection/stress measurements (Figure 10-10)	M-M1.2	Dynamic loading
		M-M1.3	Short-term
M-M1.1	Static loading	M-M1.4	Long-term
	M-N. Vibration Tests		
M-N1	Free vibration tests	M-N2.2	Variable frequency sinusoidal excitations
M-N2	Forced vibration tests	M-N2.3	Transient excitations
M-N2.1	Steady-state sinusoidal excitations	M-N3	Measurement of oscillation frequency

I reliability of results;

J (whenever possible) no laboratory work needed;

K no (or small) disruption of the bridge's normal use.

The main methods referred in Table 10-4 (de Brito 1992) were analyzed according to these criteria. Table 10-5 (de Brito and Branco 1990) shows the rating obtained based on the following point assignment:

2—the method complies with the criterion;

1—the method does not fully meet the requirement;

0—the method does not comply with the criterion.

The results of this analysis are debatable (because some criteria are more important than others), but they seem to contribute to an easier choice of the most promising diagnostic methods used on a day-to-day basis.

Methods with low ratings are generally limited to very specialized studies (frequently with laboratory support), entail high costs, and require a great deal of time. They must be used only when the "best" methods do not supply the required results.

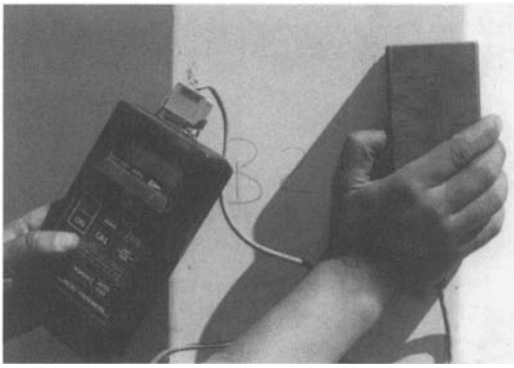

Figure 10-29. Magnetometer (diagnosis method M-D1)

Taking into account the rating obtained, the diagnostic methods that should constitute the backbone of concrete bridge inspections are the following: direct visual observation (with no equipment or assistance), surface hammering/chain dragging, extensometers (mechanical or electrical), deflection transducers, sclerometers, clinometers, chemical indicators, stressmeters, magnetometers, water leveling, vibro-wire gauges, dynamometers, load cells, galvanic cells, ultrasonography, penetration, pulse echo, and global dynamic load tests. Other methods (such as pull-out, core extraction, radar, maturity measurement, thermography, and static load tests) have some potential but are a second choice in relation to the first methods and should be used only when their output is specifically needed.

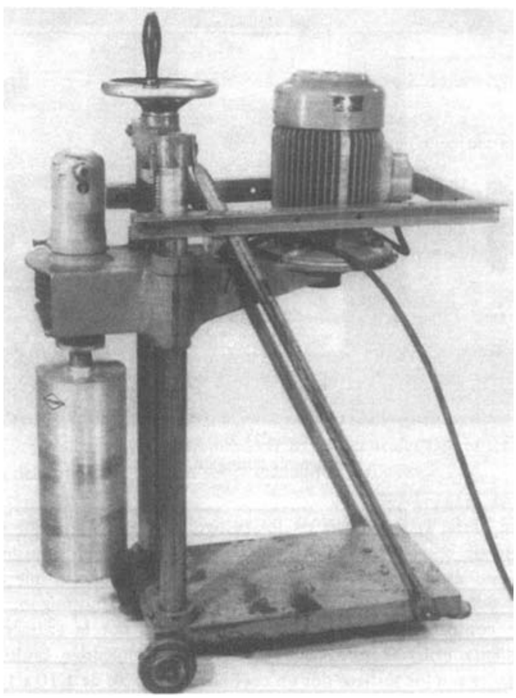

Figure 10-30. Core extractor (diagnosis method M-B5)

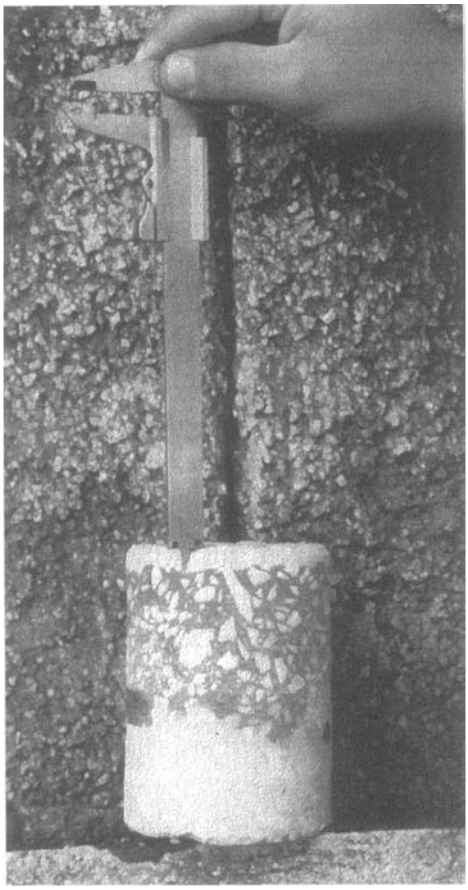

Figure 10-31. Measuring the carbonation front depth using a phenolphthalein indicator (diagnosis method M-K1)

10.4. Correlation Matrices

After creating a classification system that allows the standardization of inspection reports and forms, a tool is needed to relate the defects detected at the bridge site with their probable causes in order to facilitate in situ diagnosis. The preparation of the items "Maintenance Work Needed" and "Repair Work Needed" will be much easier if a first analysis correlation between the defects reported and their possible repair techniques is provided to the inspector. Finally, when planning the inspections and, in particular, those that require in situ tests, it is fundamental to choose the diagnostic methods, equipment, and workmanship that better fit the defects that are expected to be found or those defects detected during previous inspections and to be investigated in greater detail.

To achieve these objectives, correlation matrices (defects x probable causes, defects x repair techniques, and defects x diagnostic methods) have been prepared. Through a simple code defined later in this chapter, degrees of correlation have been defined (no, low, or high correlation) between each entry of the various entities in accordance with the classification system presented previously. The matrices are bulky and elaboration is a lengthy process during which errors may have been introduced. They do, however, rep-

Table 10-5. Diagnosis methods rating proposal

Diagnosis Method	Criterion A	B	C	D	E	F	G	H	I	J	K	Total
M-A1	2	2	2	2	2	2	2	2	0	2	2	20
M-A2	2	2	2	2	2	2	2	2	0	2	2	20
M-A3	2	1	0	1	2	2	2	1	0	2	2	15
M-A4	0	1	2	2	2	1	2	2	0	2	1	15
M-A5	1	0	1	1	2	1	2	1	0	2	2	13
M-B1	2	2	1	2	2	2	2	2	0	2	2	19
M-B2	2	1	1	2	2	2	2	2	0	2	2	18
M-B3	0	1	0	1	1	2	2	1	1	2	1	12
M-B4	1	0	0	1	1	1	2	1	1	2	1	11
M-B5	1	0	2	1	0	1	1	1	2	0	0	9
M-C1	1	1	2	1	1	1	2	1	1	2	0	13
M-D1	1	1	1	1	2	2	2	1	1	2	1	15
M-D2	0	0	0	0	2	1	1	0	0	0	1	6
M-D3	1	0	1	0	2	0	0	0	0	2	1	7
M-E1	0	0	0	0	2	0	1	0	1	0	1	5
M-E2	1	0	0	0	2	1	1	0	1	0	1	7
M-E3	1	0	0	0	2	1	1	0	1	0	1	7
M-F1	1	0	1	1	2	1	2	1	1	2	1	13
M-F2	0	0	0	0	1	0	1	0	1	0	1	4
M-F3	0	1	0	1	2	1	2	1	1	2	1	12
M-F4	0	0	0	0	2	1	1	0	0	2	1	7
M-F5	0	1	1	0	2	1	0	0	1	2	0	8
M-G1	0	0	1	1	2	0	0	0	0	1	1	6
M-G2	0	0	1	1	2	0	0	0	0	1	1	6
M-G3	0	0	0	0	2	0	0	0	0	0	1	3
M-G4	0	0	0	0	1	0	0	0	0	0	1	2
M-H1	0	0	0	0	2	0	0	0	0	0	1	3
M-H2	0	0	0	0	2	0	0	0	0	0	1	3
M-I1	0	1	0	1	2	2	2	1	0	1	1	11
M-I2	0	0	1	1	2	1	0	0	1	2	0	8
M-I3	0	0	0	0	1	1	1	0	0	0	1	4
M-J1	1	1	1	1	2	0	2	1	1	2	2	14
M-J2	2	1	1	1	2	2	2	2	2	2	2	19
M-J3	1	1	1	1	2	2	1	1	2	2	2	16
M-J4	1	0	1	2	2	1	2	1	2	2	0	14
M-J5	1	2	1	2	2	2	2	2	1	2	2	19
M-J6	1	2	1	1	2	2	2	2	1	2	2	18
M-J7	0	0	1	0	2	0	0	0	2	2	2	9
M-J8	1	0	1	2	2	0	2	1	2	2	2	15
M-J9	1	1	1	1	2	1	1	1	2	2	1	14
M-J10	1	1	1	1	2	2	1	1	2	2	2	16
M-J11	0	1	1	1	1	1	2	1	1	2	1	12
M-J12	0	0	0	0	2	1	1	0	0	1	1	6
M-J13	0	0	0	0	2	1	2	0	0	0	1	6
M-J14	0	0	0	0	2	1	1	0	0	0	1	5
M-J15	0	0	1	0	2	0	0	0	1	0	2	6
M-K1	2	1	1	2	1	2	2	2	1	2	1	17
M-K2	2	1	1	2	1	2	2	2	1	2	1	16
M-K3	1	1	1	2	1	2	2	2	1	2	1	16
M-K4	1	1	1	2	1	2	2	2	1	2	1	17
M-L1	0	0	0	0	1	0	2	1	1	0	1	6
M-M1	0	0	2	1	0	0	1	1	2	2	0	9
M-N1	1	2	2	0	1	0	1	1	1	2	0	11
M-N2	1	2	2	0	2	0	1	1	1	2	0	12
M-N3	0	0	1	1	2	0	0	0	2	2	0	8

2—method complies with the criterion; 1—method does not fully meet the requirement; 0—method does not comply with the criterion resent a first step and must be tested and corrected through its practical application in bridge networks.

These matrices have another important function. They are the basis for the computer-based in situ inspection support module (the ISM, which is discussed later in this chapter). The ISM is a very useful tool for the inspector because it not only provides bridge data and characteristics, but also gives indications about how to proceed when a certain defect is detected.

The criteria that shaped the making of the correlation matrices are presented next.

10.4.1. Defects versus Probable Causes

A first attempt at relating defects to their respective causes was made within the Brite 3091 Project (de Brito et al. 1994), in which only corrosion-related defects were included. The possible causes were divided into near causes and primary causes.

Near causes are those that immediately preceded the defect's becoming visible. Generally, they are not the root of the problem and have been preceded by the primary causes, which were actually responsible for triggering the problem.

Primary causes can be quite distant from the defect and their relationship is sometimes very indirect. For example, a faulty patch of asphalt pavement can be responsible for cracking along the edge of the repaired area. Water can then infiltrate the asphalt and, without a proper waterproofing membrane, reach the reinforcement bars. If this water contains chlorides from the sea or from deicing salts, corrosion can start and, at an advanced stage, can cause spalling of the concrete cover. Thus, faulty patching of the asphalt pavement can be a primary cause of spalling. As is obvious from this example, a primary cause by itself is relatively harmless. It is the existence of several primary causes and the passing of time that allows deterioration to reach such proportions that the defect becomes visible. Thus, the list of primary causes for each defect must be regarded as a group of factors that contribute synergistically to the development of a defect.

Although elimination of possible causes with a very low level of probability was attempted, some such causes may have remained. In some cases, in which the possible causes, albeit quite improbable, can cause significant damage, it has been decided to add them to the list (e.g., earth sliding or explosion/bombing).

The list thus prepared was then turned into a correlation matrix that included only corrosion-related defects (de Brito et al. 1994). In this matrix, in the intersection of each line (representing a defect) and each column (representing a possible cause), a coefficient representing the knowledge-based correlation degree between one and the other has been introduced. The criteria adopted for that coefficient is described here (de Brito et al. 1994):

- **0—NO CORRELATION:** no relation whatsoever (direct or indirect) between the defect and the cause;
- **1—LOW CORRELATION**: indirect cause of the defect, connected only with the early stages of the deterioration process; secondary cause of the deterioration process and not necessary for its development;
- **2—HIGH CORRELATION**: direct cause of the defect, associated with the final stages of the deterioration process; one of the main causes of the deterioration process and essential to its development.

The existence of the matrix does not invalidate the correlation list, which may contain more detailed and specific data that cannot be included in the matrix.

The correlation matrix has been further developed to include all defects in Table 10-1 (de Brito 1992). A short excerpt of the matrix is shown only as an example in Table 10-6 (de Brito 1992). The complete matrix is presented in de Brito (1992).

10.4.2. Defects versus Repair Techniques

A first attempt to relate the defects with their respective repair techniques was made within the Brite 3091 Project (de Brito et al. 1994), in which only corrosion-related defects were included. Possible repair techniques were divided into preventive repair techniques and defect repair techniques.

Preventive repair techniques, although they do not deal directly with the defect, may be necessary to eliminate its cause. For example, if the failure of retainer bars in a bearing is caused by steel corrosion, which in turn was caused by drainage of water directly over the bearing, which was itself caused by the failure of the sealant of a joint next to the bearing, then the replacement of that sealant will eliminate the cause but not the defect.

Defect repair techniques are those that deal directly with the defect. In the example given previously, the replacement of the retainer–bars, or even the whole bearing, would be necessary. If the defect affects a surrounding area, the repair of that area is also considered as part of the defect repair technique. For example, the repair of a corroded bar also includes concrete removal around the bar and patching of the cavity.

The list of preventive repair techniques is also closely related to the possible causes of the defect and, therefore, to the correlation list between defects and their causes.

The list thus prepared was turned into a correlation matrix, but it included only the corrosion-related defects (de Brito et al. 1994). In this matrix, in the intersection of each line (representing a defect) and each column (representing a repair technique) a coefficient representing the knowledge-based correlation degree between defect and technique has been introduced. The criteria adopted for that coefficient are (de Brito et al. 1994):

- **0—NO CORRELATION**: no relation whatsoever (direct or indirect) between the defect repair technique and the repair technique;
- **1—LOW CORRELATION**: preventive repair technique aimed at eliminating the cause or causes of the defects but not the defect itself;
- **2—HIGH CORRELATION**: defect repair technique aimed at eliminating the deterioration of the area in which the defect was detected but not necessarily its cause.

The existence of the matrix does not invalidate the correlation list, which may contain more detailed and specific data that cannot be included in the matrix.

The correlation matrix has been further developed to include all the defects in Table 10-1 (de Brito 1992). The complete matrix is presented in de Brito (1992).

10.4.3. Defects versus Diagnostic Methods

A first attempt to relate defects to their respective diagnostic methods was made within the Brite 3091 Project (de Brito et al. 1994), in which only corrosion-related defects were included. In this matrix, in the intersection of each line (representing a defect) and each column (representing a diagnostic method), a coefficient representing the knowledge-based

Table 10-6. Excerpt of the concrete bridge defects versus their possible causes correlation matrix

	C-A1	C-A2	C-A3	C-A4	C-A5	C-A6	C-A7	C-A8	C-A9	C-A10	C-A11	C-A12	C-A13	C-A14	C-A15	C-A16
A-A1	0	1	1	1	0	0	1	1	1	1	1	1	0	0	0	0
A-A2	0	1	1	1	1	0	1	1	1	1	1	1	0	0	0	0
A-A3	0	1	1	1	1	1	1	1	1	1	1	1	0	0	0	0
A-A4	1	0	0	1	0	0	1	0	1	0	1	1	0	0	0	0
A-B1	0	2	2	0	0	0	0	0	0	0	0	0	0	0	0	0
A-B2	0	1	1	1	0	0	1	1	0	0	1	2	0	0	0	0
A-B3	0	1	1	1	0	0	1	1	0	0	1	2	0	0	0	0
A-B4	1	0	0	1	0	0	0	0	0	0	0	1	0	0	0	0
A-B5	0	0	1	0	0	0	0	0	0	0	0	1	0	0	0	0
A-B6	0	0	1	0	0	0	0	0	0	0	0	1	0	0	0	0
A-B7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
A-B8	0	1	0	0	0	0	0	0	0	0	0	0	1	0	0	0
A-B9	0	2	0	0	0	0	0	0	0	0	0	0	2	0	0	0
A-C1	0	0	0	0	0	0	0	0	0	0	0	0	0	2	0	0
A-C2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
A-C3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
A-C4	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
A-C5	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
A-C6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0
A-C7	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0
A-C8	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	1
A-C9	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
A-C10	0	0	0	1	0	0	0	0	0	0	1	1	0	0	0	1
A-C11	0	0	0	1	0	1	0	0	0	0	1	1	0	0	1	1
A-C12	0	0	0	1	0	0	0	0	0	0	1	1	0	0	0	1
A-C13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	0
A-D1	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0
A-D2	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0
A-D3	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0
A-D4	0	0	0	0	0	0	0	0	0	0	0	0	0	2	0	0
A-D5	0	0	0	0	0	0	0	0	0	0	0	0	0	2	0	0
A-D6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
A-D7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
A-D8	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	1
A-D9	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	1
A-D10	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	1
A-E1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
A-E2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
A-E3	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0
A-E4	1	0	1	1	0	0	1	0	0	0	1	1	0	0	0	0
A-E5	1	0	1	1	0	0	1	0	0	0	1	1	0	0	0	0
A-E6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
A-E7	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
A-E8	0	0	0	1	0	1	0	0	0	1	0	0	0	1	0	1
A-E9	1	0	0	1	0	0	1	0	0	0	1	1	0	0	0	0
A-E10	0	0	0	1	0	0	1	0	0	0	1	1	0	0	0	0
A-E11	1	0	0	1	0	0	1	0	0	0	1	1	0	0	0	0
A-E12	1	0	0	1	0	1	1	0	0	1	0	0	0	0	0	0
A-E13	1	0	0	0	0	1	1	0	0	1	0	0	0	0	0	1
A-E14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

0—no correlation; 1—low correlation; 2—high correlation

correlation degree between defect and technique has been introduced. The criteria adopted for that coefficient are (de Brito et al. 1994):

- **0—NO CORRELATION**: no relation whatsoever (direct or indirect) between the defect and the diagnostic method;
- **1—LOW CORRELATION**: the diagnostic method may be useful as a second choice to a high correlation method when the latter cannot be performed or yields inconclusive results; it may also be useful to obtain secondary information on the extent or cause of the defect;
- **2—HIGH CORRELATION**: the diagnostic method is, in principle, essential to the inspection of the defect; it provides necessary information on the extent, degree, and cause of the defect; it may be replaced by a low correlation method if, for some reason (lack of equipment, workmanship, time, etc.), it cannot be performed; its use does not invalidate the use of other methods if more detailed information is thought to be necessary.

This matrix is also closely related to those presented previously because the diagnostic method is a function of the cause of the defect and of the repair technique that is supposed to be used.

In some cases, certain diagnostic methods have been classified as having no relationship to the defect, even though that is not strictly true. The criterion followed is to eliminate diagnostic methods that should be performed in the laboratory and can be replaced by other diagnostic methods already used in situ. Some diagnostic methods (e.g., the force/deformation techniques), which have been rated as very promising according to Table 10-5, may appear to be not very useful in practice. This is due to the fact that their use is restricted to specific problems (structural behavior analysis) that nevertheless occur very frequently or are necessary to keep the bridge in service.

The correlation matrix has been further developed to include all the defects in Table 10-1 (de Brito 1992). The complete matrix is presented in de Brito (1992).

10.5. Inspection Manual

Inspection procedures must be standardized at each stage in order to obtain judicious data that are not biased by subjectivity that is necessarily associated with such activities as the visual observation of bridges. It is thus fundamental to create a manual that outlines the various inspection levels (initial characterization, routine and detailed inspections, and structural assessment), follows a predetermined methodology, and makes inspections as independent as possible of the people who perform them. The manual must not be excessively technical because it targets a wide range of people with different qualifications.

The inspection manual that covers the type of the bridge to be inspected must accompany the inspector on his visit to the site. It is also advisable to study the manual in order to conceive a better plan for the inspection. The inspection team leader, the personnel in charge of the in situ tests, and the people in charge of the safety of the inspection should all have read the manual.

A general road or railway management system consists of more than bridges, and of the bridges involved, not all are made of concrete. The manual must therefore take into account the particular aspects of each type of structure to be inspected. This problem was solved in different ways in different countries: in France (MTRD 1979), a set of separate sections for each type of structure (and even certain elements such as the bearings) was created; in Canada (Reel and Conte 1989), a single rather bulky manual takes care of all structures in terms of their structural or nonstructural elements (joints, superstructure, embankments, secondary elements, etc.). The authors believe that it is best to have independent manuals for each structure type. The intent is to have the inspector bring the appropriate manual with him to the bridge to be visited. The manual should be light and portable but must contain all the data needed to proceed with the inspection. The one inconvenience is that there is a set of manuals, each with part of the information being common and repeated, while the rest of the manual is specific to the structural type involved. Therefore, inspection manuals for the following structure types must be prepared (MTRD 1979):

- (reinforced and prestressed) concrete bridges;
- masonry bridges;
- metallic bridges;
- timber bridges;
- suspension bridges;
- cable-stayed bridges;
- bascule bridges;
- tunnels;
- retaining walls (away from bridges).

Other inspection/maintenance manuals in different countries include:

- Finland (FNRA 1989);
- Germany (Wicke 1988);
- Japan (JPCCA 1994);
- Portugal (Santos 1997);
- South Africa (NPA 1990);
- United Kingdom (Bessant 1993) and (Narasimhan and Wallbank 1999);
- United States of America (AASHTO 1978), (Lichtenstein and Minervino 1990), (ACI 1984) and (FHA 1971).

Since this book relates specifically to concrete bridges, an inspection manual project adapted for them is presented next. As is shown, an important part of the text is common to other manuals and requires only minor adaptations. Naturally, the manual index proposed shows a clear parallelism with the book's present chapter index.

10.5.1. General Principles

In this chapter of the manual, a general introduction of the inspection system is made. The scope of the manual and the structural type covered by it are defined, also referring the other structural types within the system. The general guidelines of the management system are presented as well as the inspection role within it. The notions of inspection standardization and periodicity are proposed. The inspection administrative tree, the hierarchy levels (in terms of personnel and branches), and the data collection procedures are described. A short presentation of the computerized database and of its organization is made.

10.5.2. Inspection Types

The inspection organization in terms of initial characterization, routine inspection, detailed inspection and structural assessment, is described. The differences between them are clearly defined, as well as the circumstances under which each must be implemented, including their periodicity. For each inspection type, the following points are described in detail:

- periodicity (or absence of it);
- planning (preliminary visit, special means of access, personnel safety measures, traffic blockage necessity, tests to be performed, checklist, documents and drawings needed at the bridge site);
- personnel needed (team leader and auxiliary personnel);
- equipment needed (specific lists according to the tests to be made and to points under investigation, with second options should certain equipment be unavailable);
- site procedures (operations sequence as standardized as possible, locations for visual inspection, observation frequency, general elements to collect, specific data as a function of the points under investigation, the most current structural assessment procedures, and general information for the others);
- reports (inspection forms and maintenance/repair work needed lists filling out procedures, tests results, traffic information, specific details, recommendations for the next inspection, inspection and/or repair costs estimates).

10.5.3. Terminology

For each bridge element (embankment, infrastructure, superstructure, deck, bearings, joints, secondary elements, etc.), schematic drawings or photos are presented that show how to standardize the terminology used and to classify the various types that exist in the market (Figure 10-32) (Reel and Conte 1989). The same is done concerning the design actions used in the past in bridge projects (Reel and Conte 1989).

10.5.4. Symptomatology

Lists of all the defects found in concrete bridges (Table 10-1) (de Brito 1992) and of their respective possible causes (Table 10-2) (de Brito 1992) are given. The most current symptomatology (cracks, deformations, etc.) is the object of a detailed description in which the various phenomena are classified in terms of their visual aspect at the site, their respective causes are identified as well as how a site diagnosis is performed. Attention must be focused on the difference between material defects and functional disorders (Reel and Conte 1989).

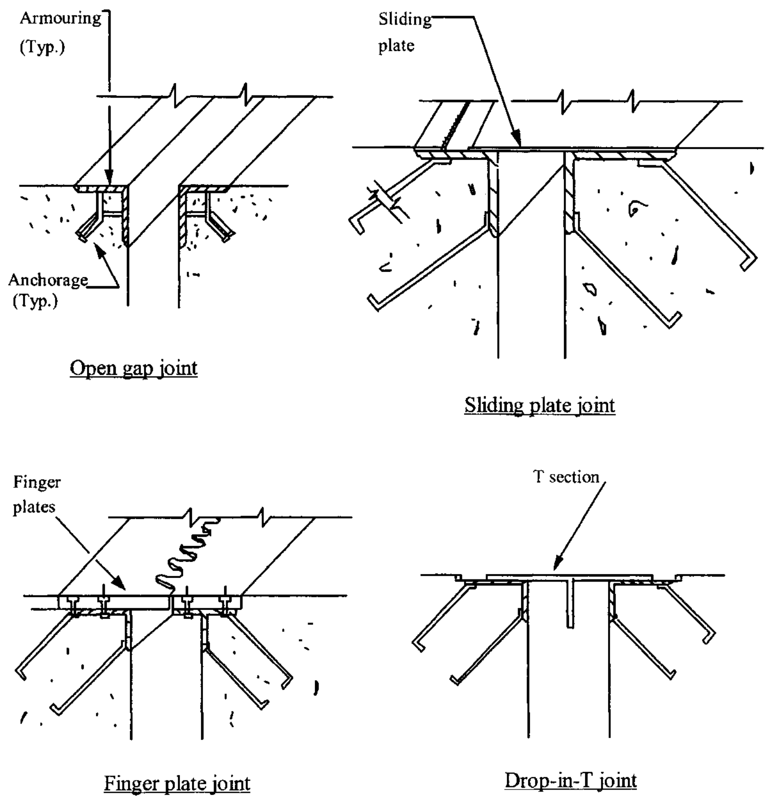

Figure 10-32. Selection of open joints used in Canadian bridges

10.5.5. Diagnostic Methods

The list of diagnostic methods for concrete bridges (Table 10-4) (de Brito 1992) is provided. Next, a very short description of the in situ tests currently available with an indication of its scope is given (it is highly advantageous to separate the tests that are easily integrated in a routine or detailed inspection from those that are specific to structural assessments). Laboratory tests with an effective and practical interest to complement the information collected in situ (from an inspection/maintenance perspective and not one based on scientific research) must be the object of a similar description. The criteria used to rate the diagnostic methods, summarized in Table 10-5 (de Brito and Branco 1990), are discussed. The tests with the greatest potential and the highest frequency of use are described in greater detail and include the following aspects: test description (principle on which it is based), factors that affect it, its scope, reliability of the results, numerical criterion (when it exists), and its advantages and disadvantages. Direct visual observation must be the object of a thorough description that defines the locations to visualize and the specific points to register.

10.5.6. Inspection Support Module

This chapter consists of a user's manual for the computer-based inspection support module (described in greater detail later in this chapter). The information that can be obtained in situ through this module and the criteria used (correlation matrices) are described. The files that it is necessary to load into the portable computer from the headquarters server, based on the bridges that are to be inspected are listed.

10.5.7. Defect Rating System

In this chapter, the defect rating system presented in Chapter 11 is described. Specific rules on how to rate each type of defect, illustrated with photos or diagrams, are given.

10.5.8 Maintenance

The list of repair techniques in concrete bridges (Table 10-3) (de Brito 1992) associated with maintenance is given. The criteria that allow the selection and rating of the defects, in terms of priority and which to eliminate, are described and an explanation of how to fill in the "Maintenance Work Needed" section of the inspection form. The levels of deterioration for various elements, based on which is repair and which is maintenance, must be recommended and defined. The repair techniques more frequently used within maintenance are the object of a detailed description (notwithstanding the repair form described later in this chapter). By resorting to tables and flowcharts (see Figure 6-7), rules must be provided on how to select a repair technique as a function of the defect detected in order to select the most effective and economic method of repair.

10.5.9. Repair/Rehabilitation

The list of repair techniques associated with structural repair for concrete bridges (Table 10-3) (de Brito 1992) is given. The criteria that allow decision making concerning the implementation of a structural assessment (Chapter 13) are described as well as the general notions about the present value economic analysis (Chapter 13). The item "Repair Work Needed" of the inspection form is also discussed. The most current structural repair techniques must be the object of a detailed description. The procedure for technique selection to be used in each case must take into account the information gathered at the structural assessment, the costs estimates, and experience in the use of rehabilitation methods and work (OMT 1988). Whenever possible, the method and material selection rules must be condensed into tables (see Table 6-13) and flowcharts, used by both the inspector and the knowledge-based decision system.

10.5.10. Bridge Dossier

In this chapter, the organization of the bridge dossier into the five subdossiers discussed later in this chapter is described and its objectives and management procedures are defined. For each subdossier, the documents that must be filed and their location in the archive are listed.

10.5.11. Defects Forms

The defects list (Table 10-1) (de Brito 1992) gives rise to a form for each defect. In it, the defect is described and is rated pseudo-quantitatively. A table of cause/consequence/future danger is created in which the symptoms susceptible to detection in a visual observation are related to their respective possible causes and probable future consequences. This set of forms, an example of which is presented in Figure 8-9, constitutes a very important appendix of the inspection manual.

A defect form contains the following elements (de Brito et al. 1994):

- type (in accordance with the list in Table 10-1) (de Brito 1992);
- code number (in accordance with the list in Table 10-1) (de Brito 1992);
- designation (in accordance with the list in Table 10-1) (de Brito 1992);
- short description (of the defect);
- possible causes (in accordance with correlation matrix defects—possible causes (de Brito 1992); causes are identified by a short description and by its code number in accordance with Table 10-2 (de Brito 1992); near causes are underlined);
- possible consequences (near or far in time);
- further inspection factors (questions related to the defect detected that may be of interest in its diagnosis or be new defects);
- inspection parameters (parameters that allow the rating of the defects in accordance with the system proposed in Chapter 11—Type 1);
- defect rating (specific rating criteria for each defect in accordance with the system mentioned previously and based on inspection parameters and other factors).

The defect form must be complemented, whenever possible, with an exemplifying photo or drawing.

The complete set of defect forms is presented in de Brito (1992).

10.5.12. Repair Forms

The repair technique list (Table 5-3) gives rise to a form for each technique. In it, the technique identification and its scope, the description of the materials used, the works sequence, the personnel and equipment needed, the estimated efficiency of the technique, specific notes, and a cost estimate, all must be included. Two secondary forms (appendices) may be included with schematic drawings and a detailed description of minor work that make up the technique. The typical repair form presented next aims merely at exemplifying what is intended and may undergo some changes according to the repair technique.

The group of repair forms thus prepared constitutes an appendix to the inspection manual, which is particularly useful after the inspection when planning the maintenance/ repair works.

TYPICAL REPAIR FORM (de Brito 1992)

1—**TYPE** (in accordance with the list in Table 10-3) (de Brito 1992)
2—**CODE NUMBER** (in accordance with the list in Table 10-3) (de Brito 1992)
3—**DESIGNATION** (in accordance with the list in Table 10-3) (de Brito 1992)
4—**SCOPE** (fields of application of the technique as a function of the defects causes, its degree, and any other pertinent factors)
5—**MATERIALS CHARACTERISTICS** (the materials list is divided into paragraphs that follow the sequence of the repair actions. Because most of the materials are used in diverse

repair techniques, material descriptions may be created and grouped in the secondary form to avoid repetition; a suitable language must be used ("if . . . then; else, . . .") so that the materials selection is made in terms of the objectives to be reached)
5.1—Surface treatment
5.2—Bonding
5.3—Reinforcement
5.4—Patching
5.5—Crack injection
5.6—Sealing/waterproofing
5.7—Repaving
5.8—Fire protection
5.9—Finishing
5.10—Others
6—WORKS DESCRIPTION (repairs are divided into paragraphs according to its logical sequence; the parameters that influence the decision to perform each action are the defect causes, the extent of the defect, the occurrence of simultaneous defects, etc.)
6.1—Removal of deteriorated material
6.2—Reinforcement emplacement
6.3—Patching
6.4—Crack injection
6.5—Sealing/waterproofing
6.6—Repaving
6.7—Fire protection
6.8—Finishing
6.9—Debris disposal
6.10—Others
7—PERSONNEL NEEDED (and their specialization)
8—EQUIPMENT NEEDED
9—ESTIMATED EFFICIENCY
9.1—Serviceability (service repair life estimate)
9.2—Mechanical (estimate of the percentage of initial strength that is restored by the repair technique)
10—SPECIAL PROBLEMS
10.1—Elimination of the defect causes
10.2—Counterindications
10.3—Special cautions
10.4—Advantages and disadvantages
10.5—Other comments
11—COST ESTIMATE (a unit cost must be presented in terms of the most adequate unit of measurement for the repair technique; if this is not possible to calculate, the unit costs must be defined for the work that the technique constitutes; these values are fundamental for the decision system, both at the maintenance level and at the repair level)
ANNEX 1—SCHEMES (includes schemes, photos, tables, and any other information useful for the identification of, defects and their causes, the work and materials needed, etc.)
ANNEX 2—SECONDARY FORMS (secondary forms include actions that are repeated for the same repair form or for different forms; this annex is common to all repair forms; the secondary forms must have a descriptive designation and a down-to-earth logically worded description)

10.5.13. Schematic Diagrams

The objective of this appendix is to explain the concept and to define criteria for the creation of a reference grid and to exemplify its design with case studies. In this appendix of the inspection manual, data collection systems are proposed based on graphic support and the use of a customized grid for each bridge that is superimposed on a simplified schematic drawing of the structure (Figure 10-33) (de Brito 1992). With this graphic aid, the inspec-

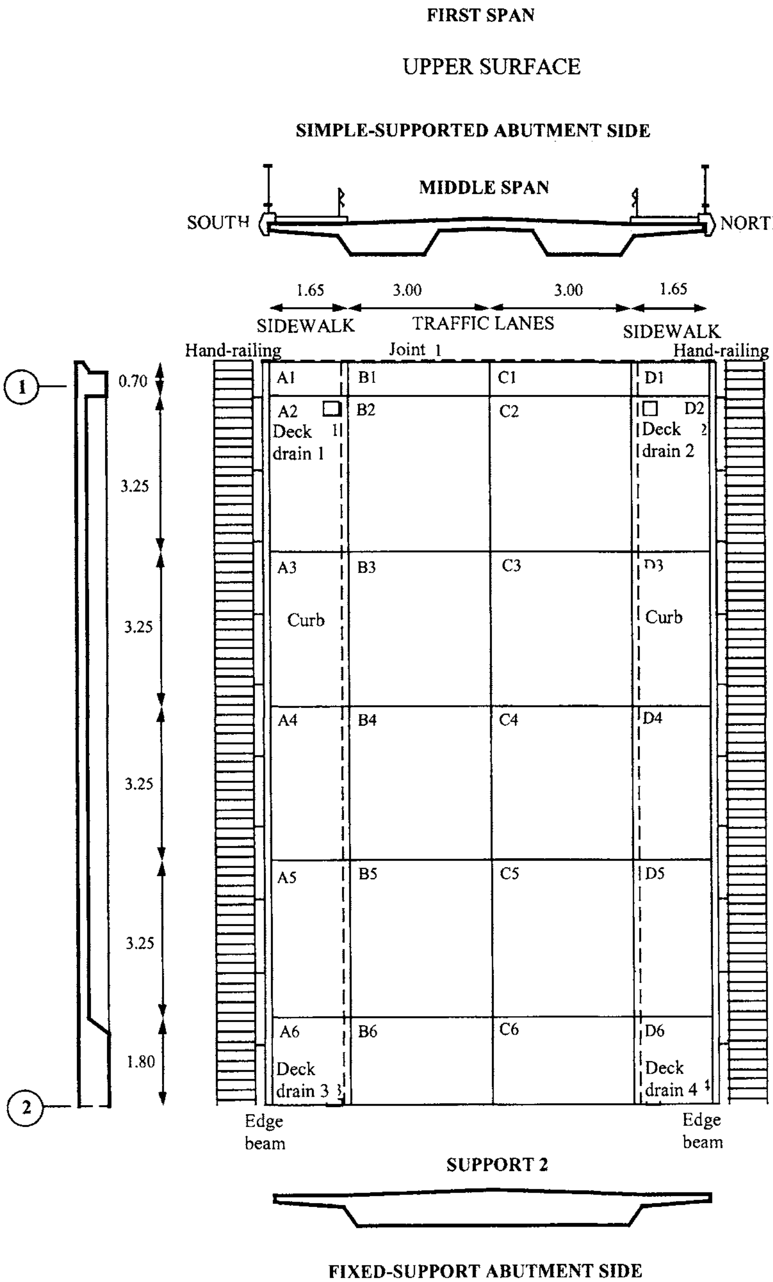

Figure 10-33. Example of a graphical scheme of a bridge with a reference grid

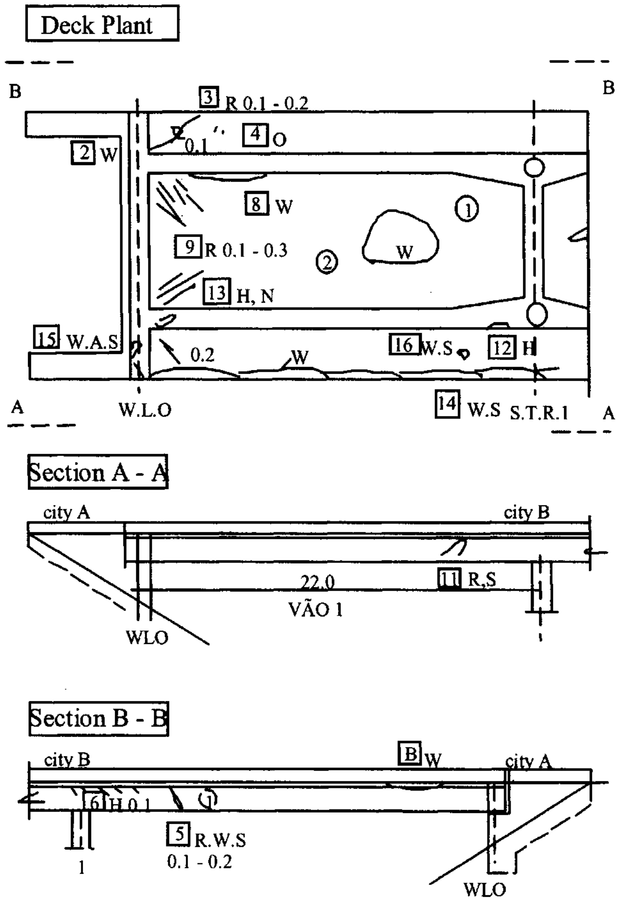

Figure 10-34. Example of a bridge graphical scheme containing information collected during the inspection

tor can describe the defects in writing and by referring to the grid, he can pinpoint the locations of the defects detected in situ. The schematics that correspond to those of the previous inspection may also be taken to the bridge site so that they can be compared with the new marked schematics. The evolution of defects may even be drawn on the same schematics (Figure 10-34) (Wicke 1988). The scale to which the schematics are drawn must be chosen so that they are easy to handle and legible.

In de Brito (1992), there is a complete example of a bridge in which the whole of its envelope has been schematically represented and a set of reference grids has been defined.

To increase the volume of data contained in the schematic drawings, the insertion of text must be avoided in favor of reference symbols for various defects. The inspector must have with him a list of the various standard symbols (Figure 10-35) (OMT 1988).

10.5.14. Identification, Reference State, and Inspection Forms

In this appendix of the inspection manual, the identification, reference state, and inspection forms are presented. All of them are included in the database described in Chapter 9. Instructions about how to fill them out at the bridge site or at headquarters are also given.

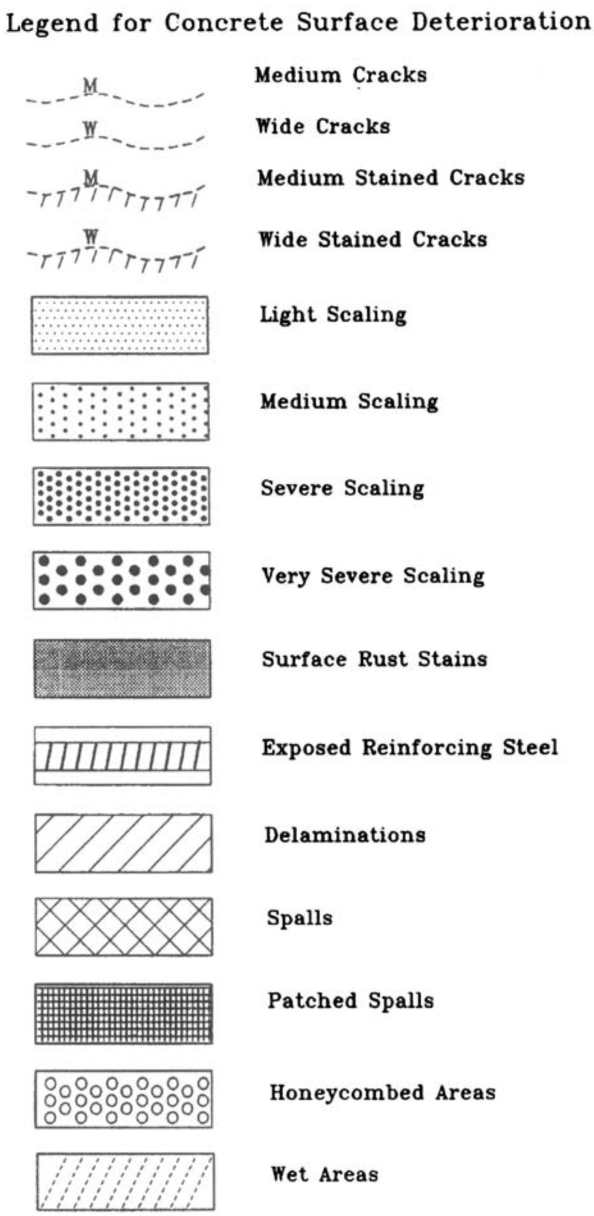

Figure 10-35. Standardization of the reference symbols to represent the defects detected in situ in graphical schemes

10.5.15. Other Appendices

Others appendices may be added to the inspection manual if their usefulness justifies it. One appendix might include all documents related to the management of the bridges (MTRD 1979) (other inspection manuals, present and outdated codes, general laws or internal documents that rule the inspections, etc.). Another possible appendix could include general ideas regarding building processes for bridges and viaducts (MTRD 1979) as well as discontinued construction techniques.

10.6. Computer-Based Inspection Support Module

To facilitate the inspector's tasks and to help him to avoid having to take along an excessive volume of reference information in the shape of manuals to the inspection site, an inspection support module (designated from now on by the abbreviation ISM) has been implemented. The objective of this computer-based module is to permit the downloading of pertinent information to a portable personal computer of small to medium capacity at the bridge site.

Within the Brite 3091 Project mentioned previously (de Brito et al. 1994), a knowledge-based system named Bridge-1 has been actually implemented and tested, even though it includes only corrosion-related defects.

10.6.1. General Organization of the Module

In de Brito (1992) a proposal for the ISM is presented in the shape of screen formats as the user is supposed to see it. The procedure for going from one screen to the other and the available options are also presented there.

The ISM's function is to interact with the user (inspector) during an inspection. Because it is intended to be used in conjunction with portable computers, the files should not be too long and it should not use very heavy databases. The system outputs are interactive indications about what the inspector should try to investigate at the site and, eventually, a provisional inspection report that is no more than a memo aid.

The first objective of the module is, faced with the data provided by the inspector that a certain defect has been detected, to give indications about the locations and matters to be investigated next. Furthermore, the module should provide some general information on the bridge, provide the most appropriate diagnostic methods and repair techniques, and define the probable causes of the defect. Finally, the module allows the creation of a provisional report that will be a fundamental element to consult when filling out the database inspection form or preparing any other supplementary reports.

Before each inspection and faced with the limitations of the portable computer, the inspector must store in the computer's memory only the general information about the bridges that he plans to visit before returning to headquarters. The remaining module data are fixed: defects, causes, repair techniques, and diagnostic methods classification and their respective correlation matrices.

The first data to be inserted is the code number that identifies the bridge. It can be chosen from a pop-up menu that appears on the screen with the code numbers of all the bridges whose general information is stored in the computer at that moment.

Next, the first defect to be detected is identified by its code number and designation (Table 10-1) (de Brito 1992). This code number can also be chosen directly from another menu on the screen, which contains code numbers and designations of every defect (Figure 10-36) (de Brito et al. 1994). After the defect is identified, the type 1 parameters, needed to rate it according to the criteria proposed in Chapter 11, are shown automatically on the screen (Figures 10-37, 10-38 and 10-39) (de Brito et al. 1994). These parameters are listed in the defect forms fully presented in de Brito (1992). The user must take note of the data needed and take the necessary measures to rate the defect detected.

Faced with this information, the ISM allows six utilization options: (1) bridge general information, (2) related diagnostic methods, (3) probable causes, (4) associated defects, (5) recommended repair techniques, and (6) a provisional inspection report. It is also possible to report a new defect and reinitiate the process or to quit the program.

BRIDGE INSPECTION

Cross-section: deck A-2
Choose a defect detected on the bridge:

	A-C01	Rust stain
	A-C07	Delamination / spalling
	A-C13	Crack over / under a bar
	A-D01	Exposed bar
	A-D04	Corroded bar
→	A-D05	Bar with reduced cross-section
	A-D06	Broken bar
	A-E02	Obstruction due to rust in bearings
	A-E03	Broken retainer-bars
	A-E06	Corrosion in bearings

More ↓

Figure 10-36. Example of the information provided by the ISM in terms of possible (corrosion-related) defects in concrete bridges

10.6.2. General Information about the Bridge

This option corresponds to the database identification form discussed in Chapter 7. It is a kind of "identity card" of the bridge under inspection and contains important additional information that allows for unequivocal identification of each bridge.

The identification form contains three information sub-modules:

- bridge site;
- general design information;
- general construction information.

BRIDGE INSPECTION

DEFECT: A-D05 Bar with reduced cross-section

REHABILITATION URGENCY:

0. Mainly black rust in areas of maximum moments with a local loss over 3%
1. Mainly black rust in areas of maximum moments with a local loss under 3%
2. Predominantly black rust in intermediate areas
3. Predominantly reddish rust

OPTION [0 TO 2] . .

Figure 10-37. Example of the use of the ISM in rating a defect (bar with reduced cross-section A-D05) in terms of rehabilitation urgency

BRIDGE INSPECTION

DEFECT: A-D05 Bar with reduced cross-section

IMPORTANCE TO THE STRUCTURE'S STABILITY:

A. Reinforcement in the deck, main beams, columns, abutments or foundations

C. Reinforcements in the auto-safes, parapets, sidewalks surface and approach slabs

OPTION [A TO C] ..

Figure 10-38. Example of the use of the ISM in rating a defect (bar with reduced cross-section A-D05) in terms of importance to the structure's stability

None of this information may be altered or deleted when using the ISM, because it is an integral part of the headquarters database.

10.6.3. Related Diagnostic Methods

This option allows the user to know which diagnostic methods are most appropriate to resume the investigation of a defect that has just been detected. Two lists (high correlation and low correlation) of diagnostic methods related to the defect are shown on the screen (Figure 10-40) (de Brito et al. 1994), in accordance with the criteria used to prepare the defect—diagnostic methods correlation matrix. The user is supposed to write down everything that may be useful to him and proceed with the use of the ISM or with the inspection. The hypothesis for using the experience acquired in the successive inspections to correct and update the correlation matrix must be considered.

BRIDGE INSPECTION

DEFECT: A-D05 Bar with reduced cross-section
AVERAGE DAILY TRAFFIC OVER THE BRIDGE: 20.000 vehicles
DETOUR LENGTH: 5,0 km

VOLUME OF TRAFFIC AFFECTED BY THE DEFECT:

k- degree of obstruction of normal traffic over the bridge caused by the defect

k VALUE [0.0 TO 1.0] ..

Figure 10-39. Example of the use of the ISM in rating a defect (bar with reduced cross-section A-D05) in terms of affected traffic

BRIDGE INSPECTION

DEFECT: A-D05 Bar with reduced cross-section
CROSS-SECTION: deck A-2
TYPE OF INSPECTION: Routine Inspection
Please select the Diagnostic Methods you used to conclude the defect:

(A)	HIGH CORRELATION
M-A01	Unaided direct visual observation
M-C01	Galvanic cell test
M-K01	Phenolphthalein
M-A02	Using binoculars, micrometer, camera or video equipment
M-A04	Using special means of aerial access
M-A05	Underwater / on water
M-K02	Silver nitrate
M-K03	Rapid chloride test

More ↓

Figure 10-40. Example of the information provided by the ISM in terms of the diagnostic methods related with the defect detected (bar with reduced cross-section A-D05)

10.6.4. Probable Causes

This option allows the user to know which causes have the highest probability of being responsible for a defect that has just been detected. Two lists (great correlation and low correlation) of probable causes of the defect are shown on the screen (Figure 10-41) (de Brito et al. 1994), in accordance with the criteria used to prepare the defects—probable causes correlation matrix. The user must write down everything that may be useful to him and then proceed to the use of the ISM or move on to the inspection. The correlation matrix may be updated by using data collected during the inspections.

10.6.5. Associated Defects

This option allows the user to know which defects have the highest probability of occurring simultaneously with a defect that has just been detected. A list of defects associated with the defect that has just been detected is shown on the screen (Figure 10-42) (de Brito 1992), according to a decreasing sequence of the correlation index associated with each defect (only the defects with an index greater than zero are shown).

The correlation index is obtained as follows (de Brito et al. 1994):

- for each defect detected (defect k), the system reads the corresponding row from the defects—possible causes correlation matrix;
- every time it finds a number different from zero c_{ki}, it travels along the corresponding cause column;

BRIDGE INSPECTION

DEFECT: A-D05 Bar with reduced cross-section
CROSS-SECTION: deck A-2
TYPE OF INSPECTION: Routine Inspection
Please select the probable Causes of the defect:

(A)	HIGH CORRELATION
C-A14	Insufficient reinforcement / prestressing design cover
C-A24	Drainage directly over concrete, joint, bearing or anchorage
C-B09	Deficient concrete compaction / curing
C-B11	Inaccurate reinforcement / prestressing positioning / detailing
C-F01	Water (wet / dry cycles)
C-F02	Natural carbon dioxide
C-F03	Salt / salty water (chlorides)
C-G01	Water (man-caused)
C-G02	Man-caused carbon dioxide
C-G03	Man-caused de-icing salts
C-A20	Excessive exposed areas in structural elements / faulty geometry
C-A23	No prevision of a minimum inclination in quasi-horizontal surfaces
C-A25	Other drainage design faults
C-A26	Lack of waterproofing membrane
C-A28	Incomplete / contradictory / overcompact drawings
C-B01	Wrong interpretation of the drawings

More ↓

Figure 10-41. Example of the information provided by the ISM in terms of the probable causes of the defect detected (bar with reduced cross-section A-D05)

- every time it finds a number different from zero c_{ji}, it adds the value $c_{ki}\, c_{ji}$ to the correlation index of the defect in row j;
- the correlation index CI_{kj} of each defect j with the defect detected k is given by:

$$CI_{kj} = \sum_{i=1}^{N} c_{ki} c_{ji} \tag{10-1}$$

where

N = total number of possible causes (Table 10-2) (de Brito 1992).

The presentation of this correlation index as described is not sufficiently enlightening for several reasons (de Brito 1992):

- the index absolute value (e.g., 64 or 10) has no physical meaning to the user;
- an index with a lower absolute value may identify a defect with a higher probability of occurring simultaneously with the one detected than one with a higher index;

BRIDGE INSPECTION

DEFECT: A-B04 Settlement / failure of the approach slab
LOCATION: West abutment
TYPE OF INSPECTION: Routine Inspection
Please select the defects associated with the defect detected:

		Correlation index (% / total)
A-I11	Damaged utilities	40,9 / 18
A-A04	Vibration	38,6 / 17
A-G08	Rippling	38,6 / 17
A-C10	Longitudinal crack	36,4 / 16
A-C11	Transverse crack	36,4 / 16
A-C12	Diagonal crack	36,4 / 16
A-E04	Cracked roller	36,4 / 16
A-E05	Roller failure	36,4 / 16
A-E09	Lead crushing	36,4 / 16
A-E11	Elastometer crushing	36,4 / 16
A-A01	Permanent deformation	31,8 / 14
A-F03	Transverse shear movement	31,8 / 14

Figure 10-42. Example of the information provided by the ISM in terms of the defects associated with the defect detected (settlement/failure of the approach slab A-B04)

this will occur if, for example, two defects occur almost always simultaneously but generally not accompanied by any other defects;

- finally, an analysis of the correlation matrix points to the fact that almost all the defects are interrelated, even though indirectly (in the example presented in Figure 10-42, 80 of the 93 possible defects are associated with the reference defect); therefore, this list shown previously consists almost systematically of most defects with the exception of the one detected, which greatly reduces the interest of this type of information.

For these reasons, it has been decided to rate associated defects in terms of the percentage of correlation index relative to the highest theoretical correlation index. The latter is obtained as follows (de Brito 1992):

- for each defect detected (defect k), the system reads the corresponding row from the defects—possible causes correlation matrix;
- every time it finds a number different from zero c_{ki}, it adds the value 4 to the highest theoretical correlation index of any defect with defect k; this is correspondent to accepting that the possible cause i of the defect k is highly correlated both with the defect k and with all the others.

To limit the list shown on the computer screen, only the defects with a percentage of correlation equal to or greater than 30 are selected (Figure 10-42).

The percentage of correlation has the physical meaning of a probability of occurrence of a certain defect taking into account that another has been detected.

The user may choose one (or more) defects associated with the defect detected from the list shown on the screen and insert other additional data that will be part of the provisional inspection report. The user can write down everything that he thinks may be useful to him and proceed with the use of the ISM or with the inspection. There is also the possibility of using the experience acquired in the successive inspections to correct and update the correlation matrix.

10.6.6. Recommended Repair Techniques

This option allows the user to know which repair techniques are the most appropriate to eliminate a certain defect that has just been detected or to eliminate its causes. Two lists (high correlation and low correlation) of repair techniques recommended for the defect are shown on the screen (Figure 10-43) (de Brito et al. 1994), in accordance with the criteria used to prepare the defects–repair techniques correlation matrix. The user is supposed to write down everything that may be useful to him and proceed with the use of the ISM or with the inspection. The correlation matrix may be updated by using data collected during the inspections.

A further stage of development of the ISM includes its use in facilitating the insertion of data in the definitive inspection form and the items "Maintenance Work Needed" and "Repair Work Needed" (the latter of a provisional nature). The user may select from the screen (Figure 10-43) (de Brito et al. 1994) the repair techniques that he considers most useful to minimize the defect detected. For each repair technique selected, the type 3 parameters (described in detail in Chapter 11), used to estimate the maintenance and/or repair work needed to eliminate the defect detected, are shown on the screen. The user must

BRIDGE INSPECTION

DEFECT: A_D05 Bar with reduced cross-section
CROSS-SECTION: deck A-2
TYPE OF INSPECTION: Routine Inspection
The related repair techniques for the defect are:

(A) HIGH CORRELATION

1. R_D01 Concrete Patching (with reinforcement / prestressing cleaning)
2. R_D02 Concrete Patching (with reinforcement / prestressing splicing / replacement)

(B) LOW CORRELATION

not specified

Figure 10-43. Example of the information provided by the ISM in terms of the repair techniques recommended for the defect detected (bar with reduced cross-section A-D05) collect in situ all the data necessary to determine these parameters and insert them in the provisional report. A list of the type 2 parameters (Chapter 11), which "measure" the defect and are related to the type 3 parameters through standardized relationships contained in the repair forms or the common sense and experience of the user (inspector), is also provided on the screen. These parameters should be a part of the definitive inspection form, and therefore the inspector must quantify them at the bridge site.

At an even further stage of development, immediately after the recommended repair techniques are provided, a list of the type 4 parameters (Chapter 11), which allow the automatic selection of the most adequate repair techniques as a function of the particular characteristics of the defect detected, is provided on the screen. The criteria that regulate this selection must be contained within the repair forms. The user collects the information needed, inserts it in the system that provides the answer, and immediately demands the type 3 parameters of the elected technique, also to be collected by the inspector.

10.6.7. *Provisional Inspection Report*

This option allows the inspector to use the ISM as a memo aid, storing all the information related to a certain defect that has just been detected in a provisional file. The file can then be used to prepare the definitive report and inspection form. The information is provided by the user who fills out the corresponding screen (Figure 10-44) and the output is a sequential file whose designation is also provided by the user. The user does not have to worry about the fact that certain items may be missing on the screen, since he can always complement the information later on.

10.7. Bridge Dossier

The bridge dossier is the document that contains all the relevant information about the bridge from the design/preliminary studies stage to the normal use/maintenance stage, in-

PROVISIONAL INSPECTION REPORT
Bridge code number: ....
File designation: ....

DEFECTS DESCRIPTION

1. Defect No.: ......
1.1. Location: ....
1.2. Defect form code number: ....; 1.3. Grid reference: ....
1.4. Parameter measured: ....; 1.5. Value: ....
1.6. Classification: ....
1.7. Remarks (causes, consequences, etc.): ....
....
....
....
....

More ↓

Figure 10-44. Provisional inspection report screen within the ISM

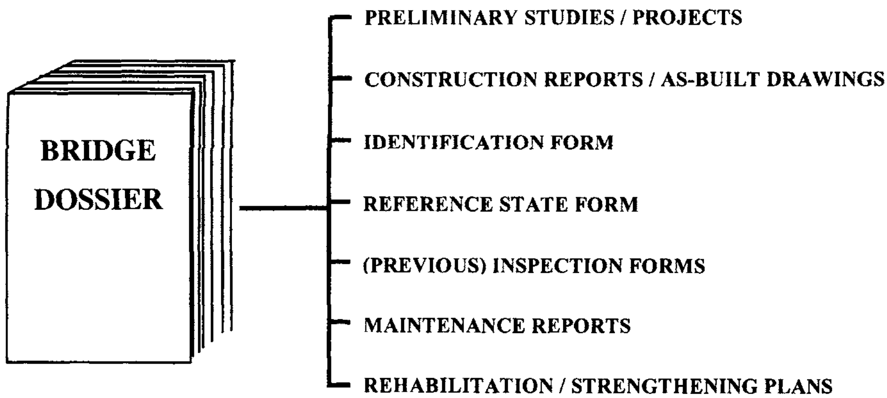

Figure 10-45. Schematic representation of the bridge dossier

cluding the construction stage (Figure 10-45) (de Brito 1992). It is a kind of traditional format (paper) counterpart to the computer-based database described in Chapter 9. It allows the storage of a great volume of information in the shape of drawings, reports, schemes, photos, and so on which would not be possible and/or economically feasible to include in the database.

The information within the bridge dossier is not necessarily limited to technical data. Administrative, legal, and even cultural aspects may also be dealt with (MTRD 1979).

The bridge dossier organization that is proposed next is based on the organization used by the Roads Department of the French Ministry of Transportation (MTRD 1979), described in Chapter 7. The bridge dossier may then be divided into the following subdossiers (de Brito 1992):

- Design/preliminary studies;
- Construction/load tests;
- Reference state;
- Operation/maintenance;
- Repair/rehabilitation.

A detailed description of each subdossier is presented next.

10.7.1. Design/Preliminary Studies

All the preliminary studies that precede the bridge project must be stored in this subdossier. Of these, the following deserve mention:

- location;
- traffic limitations;
- environmental impact;
- industrial and agricultural impact;
- surrounding urban environment impact;

- economic feasibility;
- hydraulic impact;
- integration in multinational community projects.

Obviously, for most bridges only a part of these studies will be made. A final report must accompany each preliminary study and must be stored in the dossier.

The definitive project (generally restricted to stability and roadways/railways) makes up the second part of this subdossier. In addition to the description, justification, and the recorded calculations, the contract specification, the work/material quantitative measurements and the budget must be included along with a paper copy of the definitive drawings. Together with the elements relative to structural safety, the utilization limitations in terms of loads and clearances must be specified as well as the predicted design traffic (traffic type, reference code used, classification of the bridge in terms of weight, standard vehicle used, uniform live load, design speed of the roadway/railway, and all the information that may be useful in the future if the possibility of increasing the bridge load-bearing capacity is under consideration). Finally, the bridge and the road/railway in which it is included must be classified according to the general rules of the management authorities.

This subdossier content is mostly fixed, because it does not undergo any changes after the beginning of the construction of the bridge.

10.7.2. Construction/Load Tests

This subdossier contains all the information concerning the construction of the bridge. It starts with the contract proceedings, to which the following elements are added: the proposals and curricula of every contractor, the jury's report justifying the choice made, and all the guarantees and other documents handed over by the contactor.

A summary of the expropriation of land that has taken place is also necessary. All the documents prepared by the supervisor during construction are also added: theoretical and actual work chronograms, the proceedings of meetings between the owner and the contractor, quantities of work measured, auxiliary construction drawings, monthly budgets, invoices detailing all the payments made, reports of the tests and soundings made, a justification for all the delays undergone, penalties and bonuses, etc.

This subdossier closes with a report of all the tests (materials, static and dynamic global load tests, and others) performed at acceptance of the bridge by the owner. A report showing all changes registered from the design stage as well as defects detected, accompanied by a description of all the works performed and paid for by the contractor to eliminate those defects, finalizes the description of the handing over of the bridge. Administrative documents (proceedings from the acceptance, payment of guarantees, etc.) are also stored in the subdossier.

The as-built drawings prepared in the meantime, as well as the photos taken during construction and the global load tests, are annexed to the subdossier.

Just like the previous one, this subdossier's nature is static, because it does not undergo any changes after the bridge reception and its inauguration to traffic.

10.7.3. Reference State

This subdossier contains all the information necessary to characterize the presently valid reference state of the bridge, which was previously defined in this chapter. In principle, it must be enforced right after the bridge construction but it can also be altered if the bridge undergoes significant repairs or rehabilitation. There is also the possibility that the reference state is enforced only in the middle of the bridge service life, because it has never been subjected to periodic surveillance or because the bridge has remained a long time without it.

The identification and reference state forms, included in the database described in Chapter 9, are the backbone of this subdossier. In them the main characteristics of the structure are described and the traffic using the bridge is also described. The most crucial hypotheses used in defining the design traffic must be tested after the bridge is put in service through a traffic counting program. The main results of this program are inserted into the reference state form and the complete report is stored in this subdossier.

The use of the as-built drawings included in the previous subdossier or stored on microfilm is indispensable in defining the reference state. Some information relative to subdossier 2 may be duplicated as long as the duplication makes it easier to use the bridge dossier.

Simple schematic drawings of all the outer surfaces must be prepared to represent graphically the defects detected. A reference grid that allows the swift identification of the area where the defect has been detected is superimposed on the schematics.

After the reference state has been established, this subdossier does not undergo any further changes unless it is necessary to replace the current reference state by another under the circumstances described previously. In that case, all the information described here must be duplicated to characterize the new reference state.

10.7.4. Operation/Maintenance

In this subdossier, all information obtained subsequent to the creation of the reference state of the bridge and within the scope of its normal use is stored. Structural repair or rehabilitation work, which may contribute to the alteration of the reference state, are included in the next subdossier.

All the surveillance (inspection) and maintenance program results and all the facts that may influence the bridge's level of service are stored. Specifically, the reports from traffic counting programs carried out on the bridge over a period of time are stored here. Abnormal situations, such as a sudden increase in traffic volume or heavy truck loads, must be noted. In the same way, if any deficiencies in structural behavior are detected after the occurrence of an accident (natural or human-caused), they must be reported and filed in this subdossier.

In addition to the documents discussed previously, this subdossier is divided fundamentally into two sections: inspection and maintenance. In the first section, all the forms from the periodic inspections (routine or detailed) performed at the bridge are filed, as well as the eventual reports that complement them. These forms may be obtained directly from the database presented in Chapter 9. The nonperiodic inspections (structural assessment or initial characterization of the bridge subsequent to repair work) are included in the last subdossier. For each inspection, a description of the personnel, equipment, tests performed and defects detected must be made. The defects must be identified, located, measured, and classified in accordance with the criteria discussed in Chapter 9. The information must be illustrated graphically with photos and schematics described previously on which the defects detected are marked. The photos must be numbered, dated, and referenced in terms of both the bridge and the schematic drawings.

Bridge maintenance is described under the item "Maintenance Work Needed" of the inspection form and is detailed in department reports. These include: a description of the work performed, the corresponding quantities measurement and budget (predicted and factual), personnel and equipment used, time spent, disturbance imposed on the normal traffic flow, special means of access used, extraordinary safety measures taken or recommended for the future and so on.

Through the use of the bridge dossier, the inspector must be aware of all the conclusions reached during the previous inspections as well as any interventions made (at the bridge and its accesses) and of the particular aspects to investigate.

From this description, it is easy to understand the dynamic character of this subdossier that undergoes constant updating in terms of the inspections, maintenance work, and other studies performed in time and related to the bridge.

10.7.5. Repair/Rehabilitation

This subdossier contains information related only to situations in which the bridge has a serious structural or functional deficiency, which must be eliminated. Under these circumstances, it is necessary to promote a structural assessment or to again characterize the bridge reference state.

Therefore, this subdossier is composed of the descriptions of the successive nonperiodic inspections and of the actions taken. For each inspection, the following elements are presented: a report justifying the inspection in terms of the results of the periodical inspection; the inspection planning (subjects to investigate, equipment and personnel needed, location of the structural elements to investigate, special means of access, traffic limitations during the inspection, etc.); and the results obtained (reports of the tests performed, time spent, inspection costs, recommended repair work, etc.).

The item "Repair Work Needed" from the inspection form is developed in order to produce a contract concerning the repair/rehabilitation of the bridge so that the work can proceed. This contract is included in the subdossier along with the proposals of the contractors and the jury's report. Similar to subdossier 2, the documents compiled by the supervisor during the work as well as the as-built drawings are also filed. If load tests are necessary, all documents related to them are also included in the subdossier.

If, as a result of the repair/rehabilitation or any other factor, it is necessary to create a new reference state, which must be specified in this subdossier and the documents needed to implement it must be filed. The new reference state is described in subdossier 3.

Like the previous subdossier, this subdossier is dynamic in nature even though it is updated with much less frequency. It is possible that the bridge might not require this subdossier during its service life.

CHAPTER 11

MAINTENANCE STRATEGIES

11.1. Introduction

The resources available for the management of bridges are limited in any country. In order to keep all bridges in optimal condition, the necessary funds would have to be almost unlimited. In practice, it is up to the technical personnel to provide advice to the authorities on how to select the work that needs to be performed most urgently. To do that, a rating system for all problems that may occur at any given bridge must be used. This system must be, as far as possible, independent of the subjectivity of each inspector.

The reference system presented is divided in two subsystems referred to as maintenance and repair (which also includes strengthening, deck widening, live load posting, and even demolition/replacement). The first subsystem is based on very simple criteria, mostly the results obtained from the routine and detailed inspections, as defined in Chapter 10. The second subsystem needs information that can only be obtained during a structural assessment and the use of a preliminary economic study (Branco and de Brito 1995).

11.2. Maintenance or Repair?

The management of a bridge network is performed at two different levels: the daily routine level and the level at which a problem of medium to great importance of a structural or functional nature is detected. Routine management consists of maintaining the bridges in service and solving any small problems detected during the maintenance inspections either routine or detailed. In principle, it is not necessary to use experts, either to identify the defects or to eliminate them. Problems are solved as they arise, and generally there is no excessive urgency to eliminate the defects. The amounts of money required for each bridge are also relatively small.

When a significant structural or functional defect is detected, the possibility of performing a large amount of repair work, possibly including strengthening of the superstructure, widening of the deck, or even replacement of the bridge, is proposed. The possibility of doing nothing must also be weighed against the other maintenance possibilities and the need to limit the maximum axle load through posting is verified. It is not possible to make a competent decision without implementing a thorough structural assessment of the existing bridge. An approximate cost estimate must be assigned to each option, which generally implies the need for preliminary economic (and structural) studies. If part of the problem is due to a functional deficiency, it is necessary to perform a traffic census and to predict needs in terms of maximum traffic flow and live loads. These studies are onerous and may be time-consuming. Whichever active option is selected, the corresponding costs will be very high, which means that the decision cannot be made lightly.

For these reasons, the decision system presented has been divided into two subsystems: maintenance/small repair and rehabilitation/replacement, which will be henceforth referred as maintenance and repair.

The maintenance subsystem uses repair techniques defined as maintenance in the classification presented in Chapter 10 (Table 10-3), as well as small repairs, that is, repairs of (semi-) structural defects of little importance (either because its elimination does not involve great sums of money or because it is not necessary to enroll experts to resolve them). The repair subsystem provides assistance in choosing the best repair, rehabilitation, or replacement option, when a significant defect that is capable of jeopardizing the bridge's functionality is detected. It is basically a decision of an economic nature (even though it is based on data provided by structural engineers and traffic studies), in which the costs are quantified in accordance with the cost function presented in Chapter 12.

The practical way to separate the scope of the two subsystems is as follows: whenever there is a need to perform a structural assessment, elimination of defects in their origin is included in the repair subsystem; if a decision is made to go forward with any necessary work, based solely on the data obtained from the periodical inspections, this decision must comply with the maintenance subsystem criteria.

11.3. Defects Rating System

The decision system division in maintenance and repair, as well as the defects rating system presented next, were both based on the bridge management system used by the Pennsylvania Department of Transportation (McClure and Hoffman 1990). A detailed description of the Pennsylvania maintenance subsystem can be found in Section 6.2.2 and Table 6-1.

The maintenance/small repair subsystem presented is fundamentally based on the rating attributed to the defects detected during the routine and detailed inspections. In order to make this rating, as far as possible, independent of the inspector's subjectivity, it is necessary to start by standardizing the terminology for describing the defects. A classification of all the defects that may be found in concrete bridges is presented in Chapter 10 (Table 10-1). Also presented were classifications of probable causes of those defects (Table 10-2) and a defects–possible causes correlation matrix (Table 10-6). This information has been used to compile defect forms, which are also described in Chapter 10 and wholly presented in de Brito (1992).

The final part of each defect form is dedicated to the corresponding rating (Figure 8-9). Simple, verifiable, or measurable in situ (with very simple equipment) criteria are presented even though there is a chance that the inspector's opinion may influence the actual rating. The rating system is based on three fundamental aspects (de Brito et al. 1994), (McClure and Hoffman 1990) and (Andrey 1987): rehabilitation urgency, importance to the structure's stability, and traffic volume (functionality) effects of the defect (Table 11-2) (de Brito et al. 1994). The following criteria have been customized to fit the purposes of proposed management system.

Rehabilitation urgency (this rating is the inspector's responsibility and is performed during a regular inspection visit):

0—immediate action required (e.g., if an imminent collapse is suspected when a foundations rotation—defect A-B3—is detected);

1—short-term (up to 6 months maximum) action required (e.g., if the detection of corroded bars—defect A-D4—reveals localized losses of adherence with the reinforcement totally exposed);

2—medium-term (up to 15 months maximum corresponding to the next visit) action required (e.g., if the gap between the borders of a joint in which obstruction caused by rust—defect A-F5—was detected has been reduced to less than its normal length);

3—long-term action required (in the next visit, the defect previously registered in the inspection form is rated again) (e.g., if the detection of a rust stain on concrete elements—defect A-C1—is not followed by conditions fit for corrosion of the adjoining reinforcement bars to progress and if the area affected has only a low to medium aesthetic value).

This rating corresponds to the criterion "activity urgency" of the Pennsylvania system (McClure and Hoffman 1990), in which its practical application has been simplified by eliminating an intermediate level (Table 6-9).

Importance to the structure's stability (this rating is constant for each defect type as a function of its location):

A—eminently structural defect associated with main structural elements (bridge deck, beams, columns, abutments, and foundations) (e.g., any permanent deformation of the superstructure—defect A-A1);

B—semistructural defect associated with main structural elements or structural defect associated with secondary structural elements (bearings, joints, etc.) (e.g., a bearing cracked roller—defect A-E4);

C—semistructural defect associated with secondary structural elements or defect in nonstructural elements (wearing surface and waterproof membrane, drainage system, sidewalks, hand railings, curbs, etc.) or nonstructural defect (e.g., inadequate/inexistent lighting—defect A-I12).

This rating corresponds to a simplification of the criterion "bridge maintenance activity" of the Pennsylvania system (McClure and Hoffman 1990), in which, in addition to limiting the number of levels to three, the questions related to materials fatigue have been eliminated because they usually are not relevant in concrete bridges.

Volume of traffic affected by the defect (this rating, which is inherent in each bridge, must be updated periodically but also depends on the inspection visit):

$\alpha - t.v. \times d.l.\ x\ k \geq n_1$ [vehicle km/day];

$\beta - n_1 > t.v. \times d.l. \times k \geq n_2$ [vehicle km/day];

$\gamma - t.v. \times d.l. \times k < n_2$ [vehicle km/day].

where

$t.v.$ = average daily traffic volume over the bridge (in both directions) [vehicles/day]; a value provided by the bridge authorities based on the most recent traffic census (therefore, being constantly updated); it may be replaced by a corresponding code number *d.l.* = average detour length caused by the total disruption of the bridge [km]; this value is also provided by the bridge authorities (and also requires updating)

k = coefficient that provides the degree of obstruction to normal traffic over the bridge caused by each defect (or by the most conditioning defect in each bridge); it is proposed that *k* be equal to 0.0, 0.5 or 1.0 (without intermediate values); the coefficient's value is provided by the inspector during the inspection visit

n_1, n_2 = fixed parameters defined by the bridge's authorities and calibrated as experience is gained; at a first stage, the values 3,000 and 15,000, respectively, are proposed

This rating corresponds to a significant simplification of the criteria "bridge criticality" and "bridge adequacy" of the Pennsylvania system (McClure and Hoffman 1990), which, at least at the initial stage, are overelaborated. On the other hand, the coefficient *k* was introduced, which is able to differentiate the defects that in fact affect the ability of the bridge to be used by normal traffic. Consequently, a defect, even if significant according to the other criteria, but which does not affect traffic (e.g., an exposed bar in a main girder—defect A-D1), will be rated γ, even if it occurs in a bridge with a very high daily traffic volume with no near alternatives.

Each possible rating of the defects (e.g., 1Bα) is included in one of five distinct groups of priority of action (Table 11-1) (de Brito 1992); for example, action concerning a group 3 defect will always have a lower priority than the action concerning a group 2 defect and will have a higher priority than the action concerning a group 4 defect.

Table 11-2 (de Brito et al. 1994) must be used to rate the defects within each priority of action group.

Relative to the Pennsylvania system (McClure and Hoffman 1990), in the system presented here, there has been an increase in the relative weight given to structural matters and a similar decrease in the weight given to functional matters. This fundamentally is due to the fact that the criterion "bridge adequacy," used in the North American system, considers the possibility of increasing traffic capacity until certain so-called optimum service levels are achieved. The application of these two different sets of criteria to similar bridges may then give rise to an unequal number of points being assigned.

The detected defects may then be ordered in accordance with the total number of points assigned (t.n.p.a.) to each defect (Table 11-3) (de Brito 1992).

In accordance with Table 11-3 (de Brito 1992), the possible ratings can be ranked within the priority of action groups as shown in Table 11-4 (de Brito 1992).

Table 11-1. Priority of action groups

Group	Priority of action
1	Maximum priority
2	Great priority
3	Intermediate priority
4	Small priority
5	Minimum priority

Table 11-2. Pseudo-quantitative concrete bridges defects rating

Criterion	Rating	Points
Rehabilitation urgency	0	30
	1	25
	2	15
	3	5
Importance to the structure's stability	A	40
	B	25
	C	15
Volume of traffic affected by the defect	α	30
	β	20
	γ	10

$30 \leq$ points assigned to each defect ≤ 100

Table 11-3. Total number of points assigned to each priority of action group

Group	Total number of points assigned		
1	≥ 95	and	≤ 100
2	≥ 80	and	≤ 90
3	≥ 70	and	≤ 75
4	≥ 50	and	≤ 65
5	≥ 30	and	≤ 40

Table 11-4. Possible defects rating versus priority of action group

Group	Possible ratings
1	0Aα, 1Aα
2	0Aβ, 0Bα, 1Aβ, 1Bα, 0Aγ
3	0Bβ, 0Cα, 1Aγ, 1Bβ, 1Cα, 2Aβ, 2Bα, 3Aα
4	0Bγ, 0Cβ, 1Bγ, 1Cβ, 2Aγ, 2Bβ, 2Cα, 3Aβ, 3Bα, 0Cγ, 1Cγ, 2Bγ, 2Cβ, 2Bβ, 3Aγ, 3Cα
5	2Cγ, 3Bγ, 3Cβ, 3Cγ

Within each priority of action group, 5 points (positive or negative) may be attributed by the bridge authorities to any activity in order to take into account such intangible factors as (de Brito 1992):

- district to district fund allocations;
- occurrence of serious defects in the same area;
- unavailability of technical means in the region under consideration;
- others.

11.4. Decision Criteria

All defects detected during the inspection that are rated in groups 0 or 1 in terms of rehabilitation urgency should give rise to an estimate made by the inspector of the maintenance work needed to eliminate the defects and the corresponding quantities.

All defects rated in the priority of action groups 1 and 2 should be the object of similar studies. In many cases, it will not be possible to make a precise estimate of the maintenance costs because of a lack of detailed information.

The inspector must then request all material and personnel necessary to collect the information needed about the defects included in the priority of action groups 1 and 2 (in another visit to the site or by any other means). Whenever the results from the periodic inspections are not sufficiently conclusive regarding any structural defect potentially rated in the aforementioned groups, the inspector must recommend that the bridge be subjected to a structural assessment (in which case, the decision is made within scope of the repair subsystem).

A complete list of all the potential work (repair techniques) needed for the bridge's maintenance, such as the list presented in Chapter 10, must be compiled. A unit cost, which must be regularly updated, is appended to each job. The repair form presented in Chapter 10 does not only that, but also provides a compact description of the repair technique. The cost of eliminating each defect can then be calculated and introduced in the item "Maintenance Work Needed" of the corresponding inspection form in the database, as described in Chapter 9.

Based on the defects detected, the decision criteria are (Branco and Brito 1995):

- the first bridge to be acted upon must be the bridge that has the defect with the highest point assignment; all defects of the same type on the bridge (even if with fewer points) and all defects that the authorities think may be eliminated economically with the same equipment and personnel are also taken care of; for this selection, it is possible to implement a rehabilitation module within the management system that considers the defect—repair technique correlation [through the correlation matrix presented in (de Brito 1992)], based on various parameters (materials type, construction procedures, efficiency degree, workmanship, cost, etc.); however, the final decision must always be made by the inspector or a repair expert and it is to be inserted into the database ("Maintenance Work Needed");
- global costs related to the work referred to previously are deducted from the available global budget;

- the bridge that has the defect with the second highest number of points assigned must be acted upon next in a similar way, and so on.

In the end, all decisions may be made based on nontechnical criteria. In any case, this rating system may function as an element worth consulting for the final decision.

11.5. Maintenance/Small Repair Subsystem Working Procedure

The general working procedures of this subsystem are presented next. The data that are necessary to collect from the inspections and the costs indexes are listed along with the results that are supposed to be obtained. The flowchart schematically represents the operations performed in the application of the subsystem.

11.5.1. Subsystem Input

The input of the subsystem includes (de Brito 1992):

- the inspection forms described in Chapter 7 with the ratings of all defects detected in the routine and detailed inspections;
- the "Maintenance Work Needed" forms prepared by the inspector with estimates of the amount of work that is necessary;
- a correlation between each defect detected and at least one repair technique to eliminate it;
- a list of unit prices for each repair technique considered within the maintenance scope (preferably in man hours per unit);
- the man hour cost for the present year (there is no need to plan maintenance more than one year in advance);
- the maintenance budget for the current year.

11.5.2. Subsystem Output

The output of the subsystem includes (de Brito 1992):

- a list of all the maintenance work to be performed during the current year in accordance with its priority of action; the list identifies each work in accordance with the corresponding bridge and a standard repair technique; it provides quantities and costs and should also identify the defects that each repair technique supposedly eliminates, with the same reference as that used in the inspection form;
- a list of all the maintenance work needed that will not be performed during the current year as a result of budget limitations, with the same information as the preceding list; this list may be used to justify a request for an increase in the maintenance budget and for the selection of work that cannot be put off until next year;
- the deficit or surplus generated by the maintenance budget.

11.5.3. Subsystem Flowchart

Figure 11-1 (de Brito 1992) presents the maintenance/small repair subsystem flowchart.

11.6. Subsystem Implementation Strategy

The implementation of this subsystem is rather complex and requires a wide range of tools that, at least at an initial stage, will not be available. Therefore, it is wise to define a subsystem implementation strategy that allows its functioning with only some of those elements. As the tools are developed, the subsystem will become more and more automated, until it reaches a development stage at which human intervention after inspections becomes very limited. Nevertheless, intervention is paramount in every decision since only common sense can effectively deal with intangible factors. It will be then possible to speak with the propriety of a knowledge-based system. A proposal for the strategy of development of the maintenance/small repair subsystem is presented next (de Brito 1992).

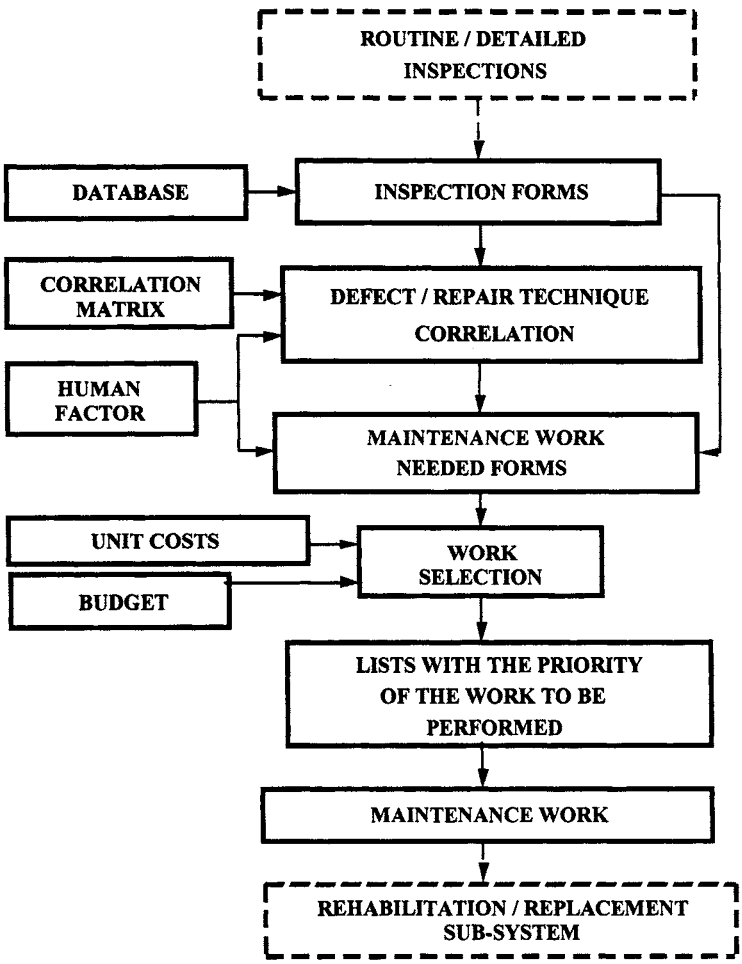

Figure 11-1. Maintenance/small repair subsystem

11.6.1. Development Stages of the Subsystem

During inspections, it is necessary to rate the defects detected in accordance with the criteria described previously. With the help of the defects forms, which are presented in detail in de Brito (1992), the inspector knows exactly which parameters he needs to determine in order to rate each type of defect. These parameters, although they are very important at the bridge site, are not as important in subsequent stages. Therefore, it was considered unnecessary to include and store them in the inspection forms. However, the computer-based inspection support module (ISM) may automatically provide the user with the parameters used to rate each defect detected (Figures 10-37 to 10-39).

The total number of points assigned to each defect is not the only information needed to determine the maintenance work to be recommended. According to the defects–repair techniques correlation matrix, for each defect there is a group of techniques, whether classified within the maintenance scope or not, that are potentially adequate. At a later stage, it is the system itself that selects the best repair technique for each particular defect (in some cases, more than one technique may have to be used simultaneously), based on a certain set of parameters. These parameters may be included in the repair forms discussed in Chapter 10.

However, at an initial stage, the inspector (or someone from maintenance headquarters) will have to make this decision and enter it into the system. Contrary to what happens with the repair/replacement subsystem, no economic analysis is performed to select the best repair technique or the defects with the highest level of urgency to be acted upon. The decision is made exclusively based on the defect rating and the estimated cost of each repair technique.

To calculate this cost, a certain set of parameters must be identified for each repair technique. Its cost must consist of two parts: fixed costs and variable costs. The fixed costs are paid whenever a technique is used, independent of the amount of work performed. The variable costs are directly proportional to the measured quantities of a certain set of parameters (area, volume, length, etc.), depending on the technique, which must be clearly identified in each repair form.

The quantification of these values is not simple because it is well known that in practice they may vary a great deal in accordance with certain circumstances. The following simplifying rules are considered:

1. The fixed costs for each technique may be considered proportional to the distance of the bridge from maintenance headquarters; it is also necessary to identify the repair techniques that use the same equipment and workmanship so that their fixed costs can be assigned only once, whenever appropriate;
2. The variable costs must be related only with parameters that are easy to identify at the bridge site; although the visible size of a certain defect does not necessarily represent the amount of repair work necessary to eliminate it, there is some relationship between these two entities; this relationship must be clearly identified in the repair form to facilitate the inspector's task of quantifying the costs and, at a later stage, to allow the system to do so automatically.

The mathematical formulation presented corresponds to Equations 11-1 to 11-3 (de Brito 1992).

$$C_i = C_{F_i} + C_{U_i} \tag{11-1}$$

where

C_i = total costs of repair technique i [$]

C_{F_i} = fixed costs of repair technique i [$]

C_{U_i} = variable costs of repair technique i [$]

$$C_{F_i} = k_{F_i} l_{Db} \tag{11-2}$$

where

k_{F_i} = fixed cost proportionality coefficient of repair technique i [$/km]

l_{Db} = distance of the bridge from maintenance headquarters [km]

$$C_{U_i} = \sum_{j=1}^{N_i} k_{U_{ij}} Q_j \tag{11-3}$$

where

N_i = numbers of parameters needed to quantify the variable costs of repair technique i

$k_{U_{ij}}$ = cost of repair technique i to produce a unitary quantity of parameter j [$/unit]

Q_j = quantity of parameter j [unit]

During the inspection, the inspector uses the option "Recommended Repair Techniques" of ISM (see Chapter 10), which supplies a list of repair techniques with high and low correlation with the defect detected. Based on his experience and on what he has seen at the bridge site, the inspector can immediately eliminate some of these techniques. For the remaining ones, the inspector should be able to select each technique and view on the screen those parameters that are needed for quantification. At an early stage of development of this subsystem, this would be enough to help. At a later stage, the parameters that define the size of the defect and are related to those that define the quantity of work performed with the repair technique are also shown on the screen and the inspector quantifies them with information collected at the site. This information is then used to fill in the item "Maintenance Work Needed" on the inspection form and to calculate the global costs of the repair technique. At a final stage, the system itself will be able to calculate and fill in this information based on data concerning the defect in the inspection form.

11.6.2. Classification of the Information Necessary for the Subsystem

To clarify this procedure, the parameters discussed are classified and described in greater detail (de Brito 1992):

TYPE 1—Parameters for Rating the Defects

The parameters used for rating the defects are identified in the defects forms. They must be identified within the ISM to facilitate the inspector's task, but they do not need to be included in the inspection form (except in the remarks). The information inserted in the form is the standard rating of the defect.

TYPE 2—Measurable Quantities of the Defect

Measurable quantities of the defect are used to calculate the type 3 parameters because they are related. At the first stage, this relationship is based solely on the common sense and experience of the inspector. At the second stage, these relationships are included in the repair forms and, through the ISM, the type 2 parameters are shown immediately after the type 3 parameters and the user must fill them in. The inspector must then fill in the item "Maintenance Work Needed" using the relationships. At the final stage, the system automatically provides the type 3 parameters based on the type 2 parameters included in the

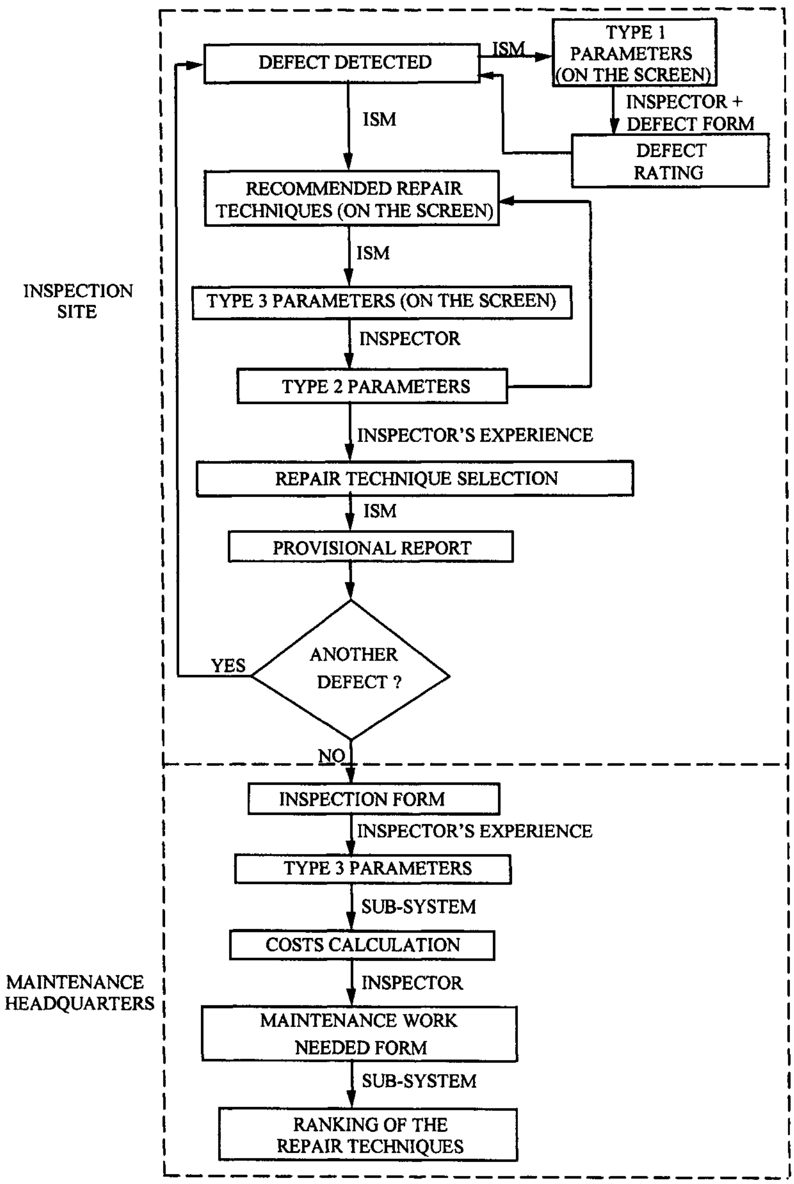

Figure 11-2. Subsystem at the initial stage

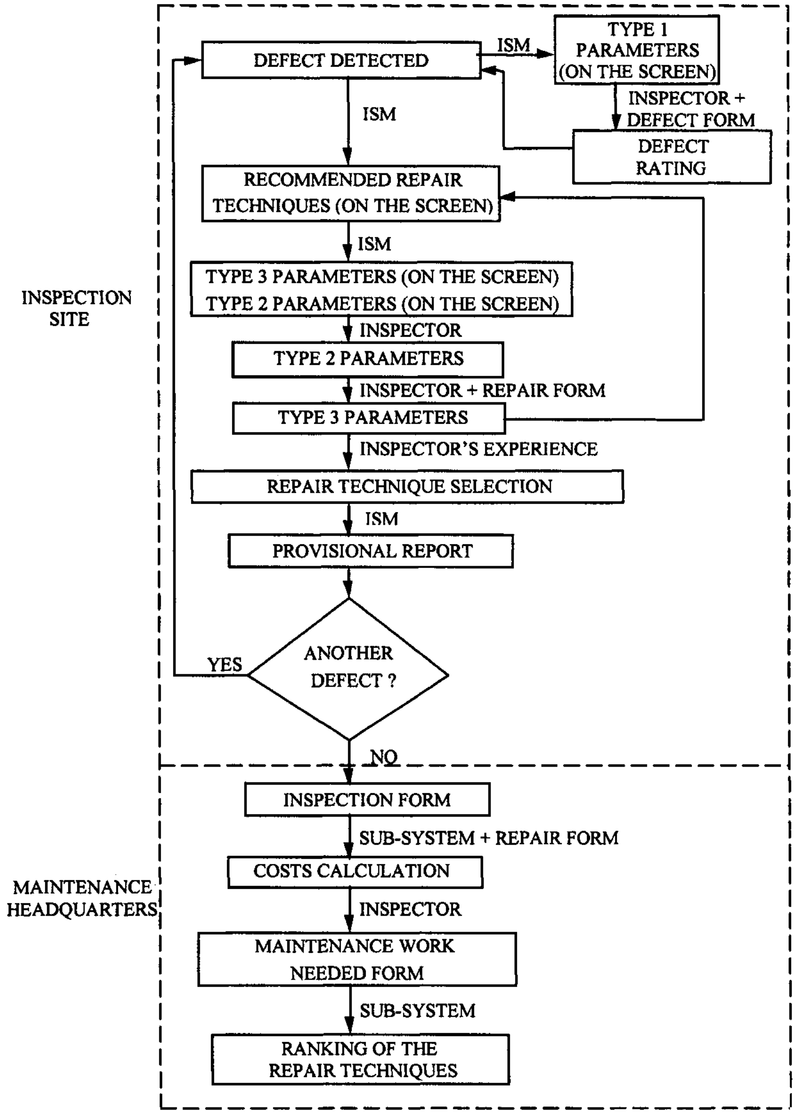

Figure 11-3. Subsystem at an intermediate stage

inspection form filled in by the user with the help of the provisional report of the ISM. The type 2 parameters must be stored in the database.

TYPE 3—Measurable Quantities of the Repair Technique and Other Parameters Needed to Determine Costs

These are the quantities and parameters needed to determine the variable costs of the repair techniques, which, when added to the fixed costs, provide the total cost. They appear on the screen when the user selects a certain repair technique correlated with the defect detected. They may be of two types: measurable quantities (e.g., m^2 or m^3), which, when multiplied by the corresponding unit cost, provide the variable costs; parameters that influence

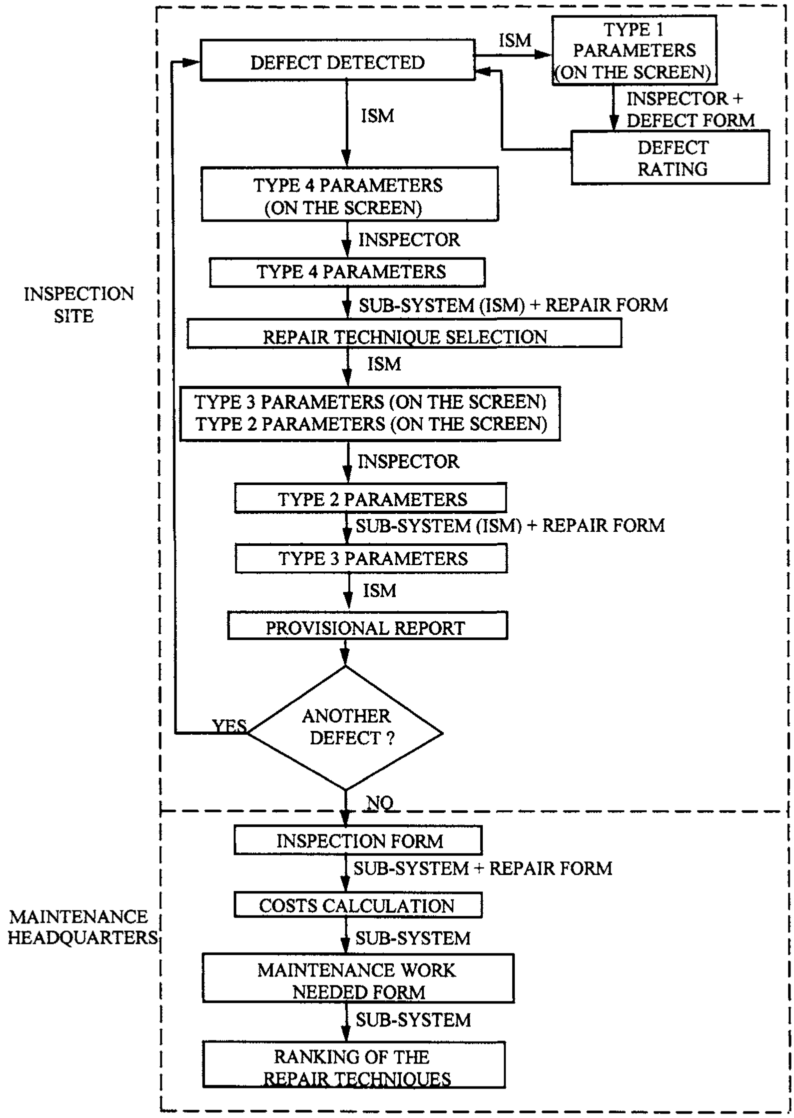

Figure 11-4. Subsystem at the final stage

the unit cost to be considered (e.g., the more or less difficult it is to access the element to be repaired). The quantities can be determined in three different ways: (1) by using the inspector's experience and what he has seen during the inspection; (2) by using the relationship between type 2 and 3 parameters provided in the repair forms, the type 2 parameters values from the inspection form and the inspector's experience; and (3) leaving all the work to the system based on the inspection and repair forms.

TYPE 4—Repair Techniques Selection Parameters

Repair techniques selection parameters are not used at the initial stages of implementation of the subsystem. At a later stage, the repair forms are to contain specific rules for selecting the most adequate repair technique as a function of the individual characteristics of each defect, thus allowing the system itself to make the choice. These rules are based on a set of parameters of the defect measurable (detectable) at the site. The inspector is asked by the ISM to provide values of the parameters. If the right answers are given, he will immediately obtain the recommended repair technique. In the event that he agrees with the system's option, he must only determine the type 3 parameters for that technique (and the type 2 parameters related to them). There is no need to store these parameters in the inspection forms (except in the remarks). Some of the type 4 parameters may coincide with the type 2 or type 1 parameters.

11.6.3. Flowcharts of the Evolution of the Subsystem

Figures 11-2 to 11-4 (de Brito 1992) represent the flowcharts of the maintenance/small repair subsystem at its various stages of development.

The fundamental difference between the stages of development of the subsystem is the amount of information contained in the repair forms. At the initial stage, only the type 3 parameters and the procedures to determine costs are provided. At the intermediate stage, the forms already identify the numerical relationships between the type 2 and type 3 parameters. In the final stage, they identify the type 4 parameters and explicitly describe how the latter determine the choice of repair technique that is most adequate under the circumstances.

Within the scope of the EC project, Brite 3091 Project "Assessment of Performance and Optimal Strategies for Inspection and Maintenance of Concrete Structures Using Reliability-Based Expert Systems," a prototype of a decision-making submodule concerning maintenance, named Bridge-2(M) (de Brito et al. 1997), was developed according to the general principles described in this chapter. The prototype is limited to the corrosion-related defects and approximately corresponds to the initial stage of development described in 11.6.1 and represented by the flowchart shown in Figure 11-2 (de Brito 1992).

PART 5

LONG-TERM COST ANALYSIS

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CHAPTER 12

Long-Term Costs Quantification

12.1. Introduction

As a result of the complexity of the analysis and the large sums of money involved, the bridge rehabilitation/replacement decision subsystem requires a rational and practical method of calculating long-term costs. The view of costs must be as broad as possible because it involves, in addition to the construction and maintenance costs, the benefits that society in general may obtain from the existence of the bridge and the road/railway in which it is included, as well as the costs associated with the disruption of its normal functioning.

In this chapter, a bridge global life cycle cost (LCC) function is described, as well as a software algorithm named COSTS, which allows calculation of the LLC. Other LCC-based concrete bridge cost prediction modeling references include: (Leeming 1993), (Silva Filho et al. 1996), (Frangopol and Estes 1999), (Vassie 1999), (Frangopol et al. 2000), Testa and Yanev 2000), and (Rubakantha and Parke 2000).

12.2. Present Value Prices Analysis

Before describing the cost function, it is fitting to remember some of the basic notions of financial analysis, specifically the present value price analysis method that allows the comparison of values that change with time.

The reference currency is the U.S. dollar. The current dollar expresses the value of goods and services in units that have the same value/purchasing capacity over time. The nominal dollar expresses the value of the same goods and services in terms of cash or "costs" at the moment at which they are carried out (Reel and Muruganandan 1990a). The present value analysis consists of calculating costs and benefits for alternative strategies in current dollars, that is, the dollars necessary at the moment of reference of the analysis (not necessarily today) to obtain goods and services at a future date.

$$C_N = PV_0(1+r)^N \Leftrightarrow PV_0 = \frac{C_N}{(1+r)^N} \tag{12-1}$$

where

C_N = capital invested in year N [$]

PV_0 = present value (year 0) of C_N [$]

r = interest rate

If the value of r is not constant in time, then:

$$PV_0 = \frac{C_N}{(1+r_0)(1+r_1)\ldots(1+r_{N-1})} = \frac{C_N}{\prod_{i=1}^{N}(1+r_{i-1})} \tag{12-2}$$

where

r_i = interest rate corresponding to year i

In general, when comparing alternatives, the one with the least present day cost is the one that is chosen. This allows a comparison of alternative strategies on an equitable basis. The analyst is supposedly in year 0, but the costs and benefits occur in any year i. Therefore, the present day global value (PV_0) of the costs-benefits (C-B) during a certain period of time N (in years) is:

$$PV_0 = \sum_{i=1}^{N} \frac{(C-B)_i}{(1+r)^i} \tag{12-3}$$

or alternatively:

$$PV_0 = \sum_{i=1}^{N} \frac{(C-B)_i}{\prod_{j=1}^{i}(1+r_{j-1})} \tag{12-4}$$

These formulas can be simplified in certain cases if the payment plan is known (Reel and Muruganandan 1990a).

The analysis does not take general prices inflation into account. Nevertheless, real prices at different times (years 0 and N) are related as follows:

$$C_N = C_0(1+f_0)(1+f_1)\ldots(1+f_{N-1}) = C_0\prod_{i=1}^{N}(1+f_{i-1}) \tag{12-5}$$

where

f_i = general prices inflation rate corresponding to year i

If the inflation rate is constant (=f), then:

$$C_N = C_0\,(1+f)^N \tag{12-6}$$

The evolution of real prices with time is measured by the nominal discount rate R that includes the influence of the inflation and of the real discount rate r:

$$(1 + R) = (1 + r)(1 + f) \tag{12-7}$$

If $r, f' \ll 1$, then equation 12-7 may be simplified:

$$(1 + R) \approx (1 + r + f) \Leftrightarrow R \approx r + f \tag{12-8}$$

Therefore, the general prices inflation has no influence on the present value prices analysis, since:

$$PV_0 = \sum_{i=1}^{N} \frac{(C_i - B_i)_r}{(1+R)^i} = \sum_{i=1}^{N} \frac{(C_i - B_i)_0 (1+f)^i}{[(1+r)(1+f)]^i} = \sum_{i=1}^{N} \frac{(C_i - B_i)_0}{(1+r)^i} \tag{12-9}$$

where

$(C_i - B_i)_r$ = real prices [$]

$(C_i - B_i)_0$ = present value prices = current prices related to year 0 [$]

However, if there is a relative variation in prices, that is if general prices have an inflation f and construction prices (costs) another inflation f', the analysis becomes more complex:

$$C_N = C_0(1 + f_0')(1 + f_1') \dots (1 + f_{N-1}') = C_0 \prod_{i=1}^{N} (1 + f_{i-1}') \tag{12-10}$$

where

f_i' = construction prices inflation rate corresponding to year i

If f' is constant in time, then:

$$C_N = C_0(1 + f')^N \tag{12-11}$$

If $r, f, f' \ll 1$, Equation 12-7 may be modified to take into account the influence of f', as follows:

$$(1 + R) \approx (1 + r + f) = [1 + (r + f - f') + f'] \Leftrightarrow R \approx r' + f' \tag{12-12}$$

$$r' = r + f - f' \tag{12-13}$$

in which r' is the adjusted discount rate.

Equation 12-9 is changed to:

$$PV_0 = \sum_{i=1}^{N} \frac{(C_i - B_i)_r}{(1+R)^i} = \sum_{i=1}^{N} \frac{(C_i - B_i)_0 (1+f')^i}{[(1+r)(1+f)]^i} = \sum_{i=1}^{N} \frac{(C_i - B_i)_0}{\left[(1+r)\dfrac{(1+f)}{(1+f')}\right]^i} \approx \sum_{i=1}^{N} \frac{(C_i - B_i)_0}{(1+r')^i} \tag{12-14}$$

Generally:

$$f' \approx f \Rightarrow r' \approx r \tag{12-15}$$

When this does not happen, Equations 12-1 to 12-9 can still be used as long as r' replaces r and f' replaces f.

To illustrate these ideas, let us suppose that in a 10-year period the inflation rate (both for general prices and for construction prices) and the interest rate are constant and equal to 10% and 4%, respectively. A certain product or service cost is $1,000 in year 0 (zero) and it increases exactly according to the inflation rate during that period. In Figure 12-1 (de Brito 1992), the diagrams of the product cost evolution in real prices and present value prices related to the year 0 are plotted. Even though the real prices follow the inflation rate, the corresponding present value prices decrease with time.

Of all the present value prices economic analyses, there are several options that, although not very different in essence, may lead to distinct decisions. Therefore, the most current analyses are described and an option is made in terms of the analysis proposed to implement this algorithm. The most current financial analytic methods are (Sinha 1986):

1. Equivalent Uniform Annual Cost Method;
2. Present Worth of Costs;
3. Equivalent Uniform Annual Net Return;
4. Net Present Value;
5. Rate of Return;
6. Benefit-cost Ratio.

Method 1 combines all investments and subsequent maintenance costs into a single annual cost AC over the analysis period, given by:

$$\mathrm{AC} = (I - T)\left[\frac{r(1+r)^N}{(1+r)^N - 1}\right] + Tr + K \tag{12-16}$$

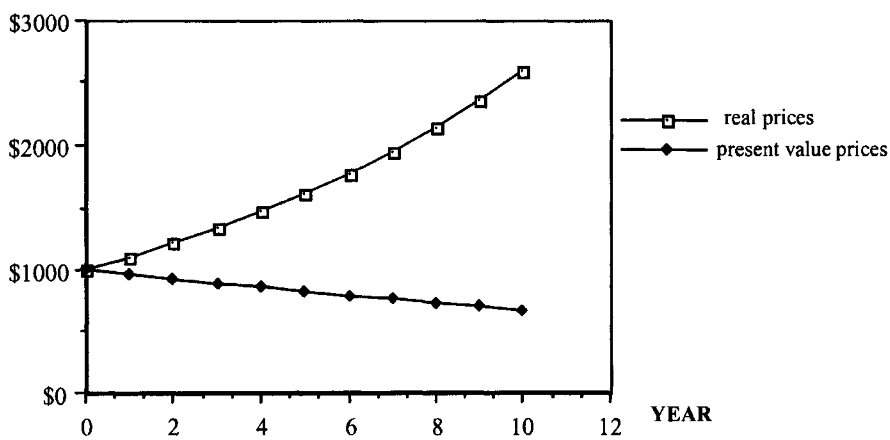


Figure 12-1. Example of the evolution of real prices and present value prices over time

where

I = initial investment [$]

T = residual value after the N years of the analysis [$]

K = annual maintenance cost (considered constant over the analysis period) [$]

r = interest rate (also considered constant over the same period)

Since it does not consider benefits derived from investment, this method (as well as the next) is only applicable to comparative analyses when the benefits (or level of service) are equal in all the alternatives and cost minimization is the only criterion.

Method 2 combines all investments and maintenance costs and other expenses into a single present worth sum (PW), which represents the sum necessary in year 0 to finance the total disbursements over the period of the analysis.

$$\mathrm{PW} = I + K\left[\frac{(1+r)^N - 1}{r(1+r)^N}\right] - T\frac{1}{(1+r)^N} \tag{12-17}$$

Method 3 takes the incremental income and benefits obtained from investment B relative to investment A and converts them into a uniform annual net return $\mathrm{EANR}_{\mathrm{A-B}}$, which is deducted from the equivalent annual costs.

$$\mathrm{EANR}_{\mathrm{A-B}} = B + \Delta T\left[\frac{r}{(i+r)^N - 1}\right] - \Delta I\left[\frac{r(1+r)^N}{(1+r)^N - 1}\right] - \Delta K \tag{12-18}$$

where

B = reduction in the operating costs (defined in 12.8.2.2.) of option B relatively to option A [$]

ΔI = increase in the initial investment of option B relative to option A [$]

ΔT = increase in the residual value after the N years of the analysis of option B relative to option A [$]

ΔK = increase in the annual maintenance costs of option B relative to option A [$]

Method 4 takes the net present value of incremental income and benefits obtained from investment B relative to investment A ($\mathrm{NPW}_{\mathrm{A-B}}$), which is deducted from the present value of the costs.

$$\mathrm{NPW}_{\mathrm{A-B}} = (B - \Delta K)\left[\frac{(1+r)^N - 1}{r(1+r)^N}\right] + \Delta T\frac{1}{(1+r)^N} - \Delta I \tag{12-19}$$

This is the method preferred by AASHTO (AASHTO 1977), because it is based on a value unequivocally defined and it facilitates the options and does not raise any ambiguities about what a benefit is and what a cost is. Relative to method 6, it presents the further advantage of generally being easier to understand by the people responsible for general policy. The software algorithm COSTS presented next is based on this method.

Method 5 determines the interest rate that makes the costs equal to the benefits or income over the analysis period.

$$I\left[\frac{1+(1+r)^N}{(1+r)^N-1}\right]+K-T\left[\frac{r}{(1+r)^N-1}\right]=R \tag{12-20}$$

where

R = uniform annual gross income or benefit [$]

r = interest rate, the unknown quantity determined by trial and error

The benefit-cost ratio B/C (method 6) expresses the ratio between the equivalent annual benefit (or its present value) and the equivalent uniform annual cost (or its present value) of the investment. An alternative with B/C > 1 is considered economically feasible, and the higher its B/C ratio the better.

$$\mathrm{B/C_{A-B}}=\frac{B}{\Delta I\left[\frac{r(1+r)^N}{(1+r)^N-1}\right]+\Delta K-\Delta T\left[\frac{r}{(1+r)^N-1}\right]} \tag{12-21}$$

One of the shortcomings of this method is that it is sometimes unclear what are costs and what are benefits. If a certain item is considered an unachieved benefit (therefore negative) instead of a cost, the value of B/C is affected. This question is discussed in Chapter 9, Section 9.8.2.2.

When there are several alternatives, the decision may be subject to discussion: the option may be the alternative with the highest B/C independent of its cost. One can opt for the alternative with the highest B/C and an initial cost that does not go over a certain limit, in the case of moderate budget constraints. It is also possible to opt for an alternative with B/C > 1 and the lowest initial investment, in the case of severe budget constraints (Sinha 1996). Notwithstanding such inconveniences, this has been the method chosen for the rehabilitation/replacement subsystem described in Chapter 13.

There are other methods of economic analysis, some of which have in some cases fallen into disuse (Sinha 1996):

- Capitalized Cost—present worth of costs with the analysis period being infinity;
- Payback Period—length of time necessary for accumulated benefits to equal investment;
- Break-even Analysis—varying one or more factors until costs and benefits are equivalent over the analysis period.

When the service life of an alternative is different and there is a need for it to equal the analysis period, the most current solution is to use the shortest service life as a reference period and to assign residual values to the remaining alternatives at the end of that period. In certain problems, when the service life of an alternative is a multiple of one of the others, it is common to use the first one as the reference period and to consider several life cycles for the remaining alternatives.

When there are several alternative investment projects, the cost of opportunity concept is important because it defines the cost of losing the opportunity to make a certain profit with the other alternatives left out to attempt to achieve a greater profit with the alternative elected. Another concept, implicit in the saying that "the optimum is an enemy of the good," is that, in order to achieve several goals simultaneously, one is frequently forced to opt for a "satisfactory" solution when faced with the complexity of the problem and the lack of precision of the database.

12.3. The Global Cost Function C

12.3.1. Preliminary Considerations

The main objective of building a bridge is to provide a means of travel for its users. Its design is defined by efficiency levels related to the design speed of the road or railway served and the maximum live load and traffic volume predicted. These levels must be maintained constant throughout the service life of the structure.

Building a public or private utility structure must be considered a form of investment. There is an initial cost to design and build the structure, there are ongoing costs for maintenance and repair, and, when the operating costs no longer justify the structure's existence, it is replaced. However, during the service life the structure must generate benefits that are worth more than the sum of all the costs. If that does not happen, building the structure was an economic blunder, especially because the resources used could have had much greater usefulness in other investments that were omitted because of budgetary constraints (cost of opportunity).

The generation of these benefits is achieved in the sense that the bridge is integrated into a certain itinerary, with "itinerary" defined as the communication link between two locations. The bridges included in it all contribute to the same goal, and the global benefits of the connection are possible only if all the bridges function as predicted. Therefore, it is necessary to assign only a fraction of the global benefits to each bridge of the itinerary. This division corresponds to the notion of "area of influence" of the bridge (Branco and de Brito 1995), which simply is the stretch of road or railway that corresponds to the relative importance of the bridge within the itinerary.

For these reasons, it is necessary to quantify in a practical way the global costs of building, operating, and replacing a structure. Also fundamental to comparative economic studies but even more difficult to determine, it is necessary to predict these costs long before their occurrence. The quantification and prediction of benefits obtained from an infrastructure during its service life is also very difficult, because it involves many parameters such as a prediction of the users' needs, the development of certain regions, the historical/architectonic value of the construction, the social/environmental impact, and others.

From what has been presented regarding the bottom line, it is obvious that it is impossible to perform economic analyses of costs and benefits for a bridge if it is considered separately and is not integrated within a certain itinerary. Therefore, some costs related to communication links, usually considered in economic feasibility studies, have been integrated into the global cost function described next. However, in the description made, it was deliberately chosen not to dwell on concepts related only to communications and transportation. They have been integrated into the economic analysis in a simplified way and only because it was believed that the analysis would suffer from a lack of realism otherwise.

12.3.2. Function Organization

The global cost function is defined by (Branco and de Brito 1995):

$$C = C_0 + C_I + C_M + C_R + C_F - B \qquad (12\text{-}22)$$

where

C = total costs involved in the construction, operation and replacement of the bridge and its area of influence [$]

C_0 = initial costs [$]

C_I = inspection costs [$]

C_M = maintenance costs [$]

C_R = repair costs [$]

C_F = failure costs [$]

B = benefits [$]

Initial costs are those involved in the design and construction of the bridge and its area of influence and include preliminary studies; structural and traffic design; construction of the bridges, its approaches, and area of influence; and load tests before use.

Inspection costs are those involved in a regular inspection of the bridge and its area of influence within the current maintenance scope (i.e., they do not include structural assessments when a serious structural deficiency is suspected). They can be divided into labor costs and equipment costs.

Maintenance costs are those involved in keeping the bridge and its area of influence at their design level of service and exclude any works related to structural aspects.

Repair costs are those involved in work of a structural nature during the bridge's operation (e.g., repair, strengthening, deck widening) and include repair costs themselves and all costs corresponding to structural assessment. It is considered that there are no repair costs concerning the area of influence of the bridge.

Failure costs are those caused by the total or partial impairment of the bridge or its area of influence to fulfill its design function. They can be divided into structural costs and functional costs.

Benefits are the value attributed to the enhancement of the capacity or service level achieved by construction or improvement of the itinerary into which the bridge is integrated. They are necessarily associated with functional aspects and with functional failure costs.

Another way of breaking down the global cost function is in structural costs C_{ST} and functional costs and benefits C_{FU} (Branco and de Brito 1995):

$$C = C_{ST} + C_{FU} \qquad (12\text{-}23)$$

Structural costs include the initial costs C_0 of design and construction, inspection C_I, current maintenance C_M, repair of the bridge C_R, and structural failure costs C_{FSF} (Branco and de Brito 1995):

$$C_{ST} = C_0 + C_I + C_M + C_R + C_{FSF} \qquad (12\text{-}24)$$

Table 12-1. Global cost function C summary

C	**(global cost function)**
C_0	**(initial costs)**
C_{0D}	(design costs)
C_{0C}	(construction costs)
C_{0T}	(testing costs)
C_I	**(inspection costs)**
C_{IL}	(labor costs): C_{ILD} (displacement costs)
	C_{ILT} (testing costs)
C_{IE}	(equipment costs)
C_M	**(maintenance costs)**
C_R	**(repair costs)**
C_{RSA}	(structural assessment costs)
C_{RSR}	(structural repair costs)
C_F	**(failure costs)**
C_{FSF}	(structural failure costs): C_{FFR} (bridge replacement costs)
	C_{FFL} (loss of lives and equipment costs)
	C_{FFH} (architectural/cultural/historical costs)
C_{FFF}	(functional failure costs): C_{FFFD} (traffic delayed costs)
	C_{FFFV} (traffic flow detoured costs)
	C_{FFFL} (heavy traffic detoured costs)
	C_{FEI} (environmental impact/social costs)
B	**(benefits)**
B_D	(traffic delayed benefits)
B_V	(traffic flow detoured benefits)
B_L	(heavy traffic detoured benefits)

Functional failure costs C_{FFF} are associated with a reduction in the bridge service level and the benefits B (defined previously) make up the functional costs and benefits (Branco and de Brito 1995):

$$C_{FU} = C_{FFF} - B \tag{12-25}$$

The proposed global cost function organization is presented in Table 12-1 (de Brito and Branco 1994).

12.3.3. Software Algorithm General Data

The algorithm COSTS was developed to calculate the global cost function C for road bridges. It is initialized with the introduction of the general data from the user analysis:

- initial (Y0) and final (YN) year of the economic analysis;
- year in which the last bridge included in the analysis is to be put out of service;
- current year (CY);

- flag with the option between present value prices and real prices in terms of output of the results;
- year of reference of the economic analysis (YR), to which the present value prices are referred (not necessarily the current year);
- number of bridges included in the analysis (NB);
- sensitivity analysis option flag (SENSI) (9 possible parameters);
- optional result output flag (3 possible degrees of detail of the information).

The sensitivity analysis consists of varying (one at a time) any of the several possible parameters of a predetermined percentage over and below the average value used in the calculations. The influence of this variation in several partial costs is expressed in the final result and enables the user to identify the parameters that it is necessary to calibrate with greater precision.

Next the general files prepared before the program is run are read:

- registered interest rates for the years before the current one;
- registered construction prices inflation rates for the years before the current one;
- average construction costs referred to a predetermined year;
- average costs due to the inspection procedures referred to a predetermined year;
- average costs due to the maintenance procedures referred to a predetermined year;
- parameters needed to quantify the failure costs and the benefits referred to a predetermined year.

The future interest and inflation rates may be predicted in several ways according to the user's wishes:

- by linear regression using values for the 10 years prior to the current one;
- by imposing a predetermined linear progression;
- by imposing year-to-year values.

Based on the rates thus obtained, all of the partial costs described are automatically referred to the current year for subsequent calculations.

12.3.4. Example of Application of the COSTS Algorithm

To illustrate some of the concepts presented in this chapter, an application example of the COSTS algorithm has been prepared, the complete results of which are presented in de Brito (1992). The example concerns an economic analysis between 1983 and 2018, supposedly performed in 1991, involving two road bridges whose service life expiration is predicted for 2030. The future inflation and interest rates have been predicted as constant and equal to 10% and 4%, respectively. To put this into a better perspective, figures and diagrams presented later in this chapter, and some of the general characteristics of the bridges (which henceforth will be designated as bridge 1 and bridge 2) are presented.

Bridge 1 (de Brito and Branco 1994)

Beginning of design/construction: 1991

Predicted end of construction: 1994

Predicted inauguration: 1995

Deck area: 450 m^2

Structural type: unspecified (30 m < maximum span < 40 m)

Road length of the "area of influence": 6 km

Road type: total width of 9,0 m in semi-rough country with a flexible pavement

Road design speed: 90 km/h

Traffic directions on the bridge: 2

Total number of lanes on the bridge: 2

Structural capacity (design vehicle load): 300 kN

Bridge 2 (de Brito 1992)

Beginning of design/construction: 1983

End of construction: 1986

Inauguration: 1987

Deck area: 550 m^2

Structural type: unspecified (30 m < maximum span < 40 m)

Road length of the "area of influence": 12,5 km

Road type: total width of 9,0 m in semi-rough country with a flexible pavement

Road design speed: 90 km/h

Traffic directions on the bridge: 2

Total number of lanes on the bridge: 2

Structural capacity (design vehicle load): 300 kN

12.4. Initial Costs

12.4.1. Definition

The initial costs can be divided into (de Brito and Branco 1994):

$$C_0 = C_{0D} + C_{0C} + C_{0T} \tag{12-26}$$

Design costs C_{0D} include all expenses related to the preliminary studies (traffic, environmental impact, geological/drilling, hydrology, economic feasibility, industrial impact, regional authorities consultation, etc.) and to the various stages of the structural and traffic design itself.

Design costs may themselves be divided into costs concerning the bridge C_{0D_b} and costs concerning the area of influence C_{0D_r} (de Brito 1992):

$$C_{0D} = C_{0D_b} + C_{0D_r} \tag{12-27}$$

Construction costs C_{0C} include all the labor, materials, equipment, building site management, and quality control costs involved in the actual construction of the bridge and its area of influence. Some miscellaneous costs, such as demolition of the existing bridge, right-of-way, utilities, creek diversion, detours, and others, should also be included.

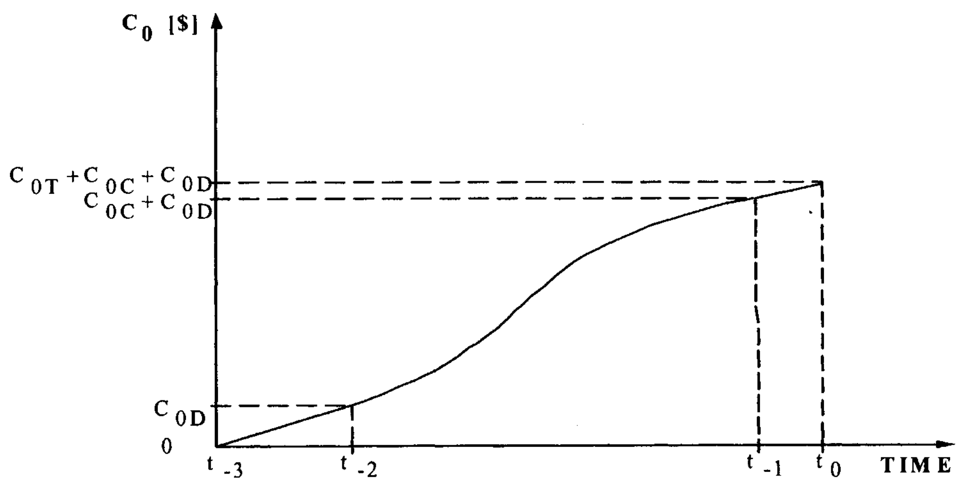
These costs may yet be divided into costs concerning the bridge C_{0C_b} and costs concerning the area of influence C_{0C_r} (de Brito 1992):

$$C_{0C} = C_{0C_b} + C_{0C_r} \tag{12-28}$$

Testing costs C_{0T} are associated with the eventual performance of global tests of the bridge before it is open to traffic (generally load and dynamic tests). No costs were considered for final testing of the roads that make up the itinerary.

All of these costs are schematically presented in Figure 12-2 (de Brito and Branco 1998a).

The definition of the area of influence of each bridge included in a certain itinerary is made based on the percentage $c_{b/r}$ of its initial cost relative to the sum of the initial cost of all the bridges of the itinerary.



t_{-3} — the decision is made to initiate the preliminary studies
t_{-2} — beginning of the construction
t_{-1} — end of the construction
t_0 — the road is open to traffic

Figure 12-2. Representation of the initial costs over time

$$c_{b/r_i} = \frac{C_{0_i}}{\sum_{j=1}^{n} C_{0_j}} \tag{12-29}$$

where

c_{b/r_i} = percentage of the global costs and benefits of the itinerary assigned to bridge i

C_{0_i} = initial cost of bridge i [$]

n = number of bridges in the itinerary

The multiplication of this percentage by the total length of the itinerary L_r provides the length of the road L_{r_i} that is considered assigned to the bridge, whose costs are added to the costs of the bridge.

$$L_{r_i} = c_{b/r_i} L_r \tag{12-30}$$

The initial, inspection, and maintenance costs concerning the itinerary and the benefits assigned to bridge i are obtained by multiplying the respective itinerary global values by c_{b/r_i}.

Bridges represent an average of around 20% of the total initial costs of an itinerary (RRG 1973), which turns the option of allocating the itinerary costs as a function of costs for each bridge—an arguable point. However, it was considered that this would be a simple way of doing it, taking into account that the work is directed at the analysis of bridges, not communications. Usually, management of roads and bridges is done separately with separate budgets. On the contrary, the analysis of the bridges must take into account the itinerary into which they are inserted, something this criterion does. The ideal, of course, would be to have a global roads and bridges management system.

12.4.2. Quantification and Prediction

The quantification of design costs is very easy if the preliminary studies of the amounts spent on each project are correctly attributed. The structural, geotechnical, and road engineers' current fees are also well known. If there is a need to predict such costs (which is unlikely within the normal scope of the management system), they must be related to the construction costs as a percentage of them. Bridge management authorities may statistically validate this percentage.

Construction costs are quantified as the work proceeds or when the contract is signed, depending on the terms of the contract. It is impossible to make an acceptable prediction before time t_{-2} (see Figure 12-2) (de Brito and Branco 1998a), unless all that is needed is a rough estimate, which can be obtained after a preliminary study of the bridge structure (price/m^2 of deck) or of the itinerary (price/km of road). When, for example, the option of replacing an existent bridge is under consideration, the simplified estimate may be used.

In a study made in the United States (Saito et al. 1991), the idea was proposed that the bridge construction cost per square meter of deck depends fundamentally on its structural type (more so for the superstructure than for the infrastructure) and is independent of the skew. Statistically, the costs concerning approaches in overpasses/underpasses represent one third of the total costs.

The costs involved in global bridge tests are also known, whether they are performed by the bridge owner, the contractor, or a third party. Their prediction is also simple as long as there is an approximation of the bridge's geometry and the tests considered necessary. It is also possible to estimate these costs as a percentage of the construction costs obtained from management authorities.

In the algorithm COSTS, the calculation of initial costs is preceded by the reading of a file with data specifically related to the initial costs of each bridge under analysis and the corresponding itinerary (de Brito 1992):

- total area of the bridge deck (ADECK);
- total length of the itinerary (L_r);
- percentage of the initial costs of the bridge under analysis relative to the sum of the initial costs of all the bridges of the itinerary ($c_{b/r}$);
- percentage of construction costs of the bridge and its approaches C_{0C_b} estimated for their design C_{0D_b} and testing C_{0T} costs;
- percentage of the construction costs of the itinerary C_{0C_r} estimated for its design costs C_{0D_r};
- itinerary road type (chosen from a range of current widths, ground outlines, rural or urban areas crossed, and pavement types, to which different costs per km of road correspond); the user may interactively change the (internally assigned) cost if he does not agree with it;
- year of times t_{-3} (Y0C0) and t_0 (YNC0) (see Figure 12-2) (de Brito and Branco 1998a);
- actual values of C_{0D_b}, C_{0C_b}, and C_{0T} differentiated by year for all years preceding the current year;
- actual values of C_{0D_r} and C_{0C_r} differentiated by year for all years preceding the current year;
- estimate of construction costs for the bridge and its approaches (when YNC0 is greater than or equal to the current year YC and an accurate estimate exists);
- structural type of the bridge (chosen from a range of current structures, to which different costs per square meter of bridge deck correspond) and percentage of the approaches in the total construction cost; the user may interactively change the (internally assigned) cost if he does not agree with it;
- estimate of the construction costs of the itinerary (when YNC0 is greater than or equal to the current year YC and an accurate estimate exists).

When YNC0 is less than YC (the bridge is already in service), there is no need to estimate initial costs. Otherwise, an estimate of the total construction costs of the bridge provided by the user or an estimate based on the bridge's deck area and structural type and on the percentage of the approaches, which are part of the global construction costs, can be used. The difference between this estimate and the costs C_{0C_b} already spent is assigned uniformly between the years CY and YNC0 (inclusively). The costs C_{0D_b} and C_{0T} are obtained from the estimate of C_{0C_b} multiplied by the percentage referred to previously. It is assumed

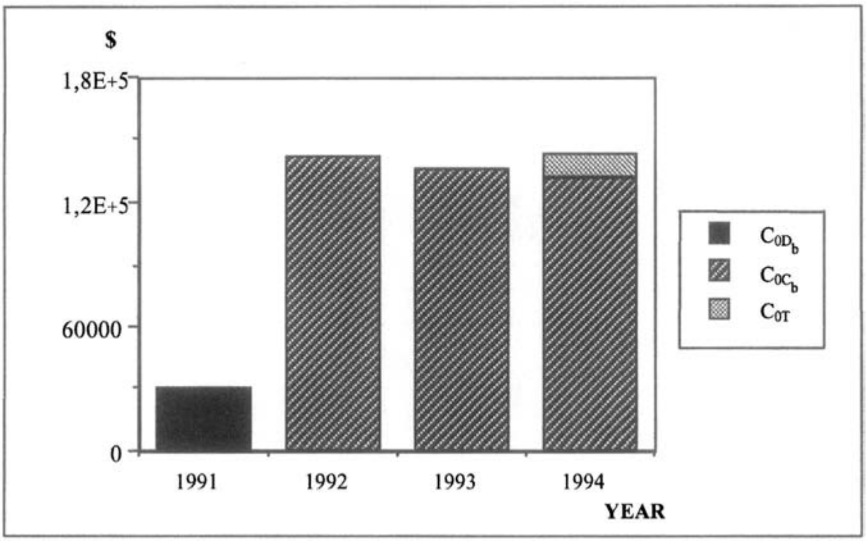


Figure 12-3. Prediction of the initial costs of bridge 1 (from Section 12.3.4.) in present value prices

that the costs C_{0D_b} all occur in the year Y0C0, unless there is a difference between the estimate of C_{0D_b} and the sum of the design costs already spent, in which case the difference is assigned to the current year. The costs C_{0T} are always assigned to the year YNC0. Figure 12-3 (de Brito and Branco 1994) illustrates the prediction of the distribution of future initial costs in time in situations in which the bridge is still in the design stage.

With regard to the area of influence of the bridge, either an estimate of all the construction costs of the itinerary provided by the user is used or an estimate based on the itinerary length and its road type is made. These estimates are multiplied by the coefficient $c_{b/r}$ (Equation 12-29). The distribution of the initial costs not yet registered is made in an identical manner to that for the costs that correspond to the bridge, except for testing costs, which are considered nil. In RRG (1973), it is stated that design costs in roads represent an average of 5% to 10% of construction costs.

12.4.3. Relative Importance

Within the management system as it was conceived, the relative importance of the initial costs is generally small because at decision-making time the initial costs are almost always a fixed quantity. The greatest majority of the decisions that must be made when managing a bridge or a network do not involve the option of replacing it, the only situation in which the prediction of the initial costs is relevant. Even when that option is valid, the ability to provide an accurate estimate of these costs is limited and an approximate number ends up being used.

If the economic study is made before time t_2 (see Figure 12-2) (de Brito and Branco 1998a) and several options for a new structure are being considered, these costs become relevant. However, it is necessary to estimate the influence of different solutions of design/construction in the maintenance, repair, and failure costs, as well as in the benefits, in order to reach a well-grounded conclusion. This can be done through a prediction of the expected service life of certain elements of the bridge, usually replaced during the structure's

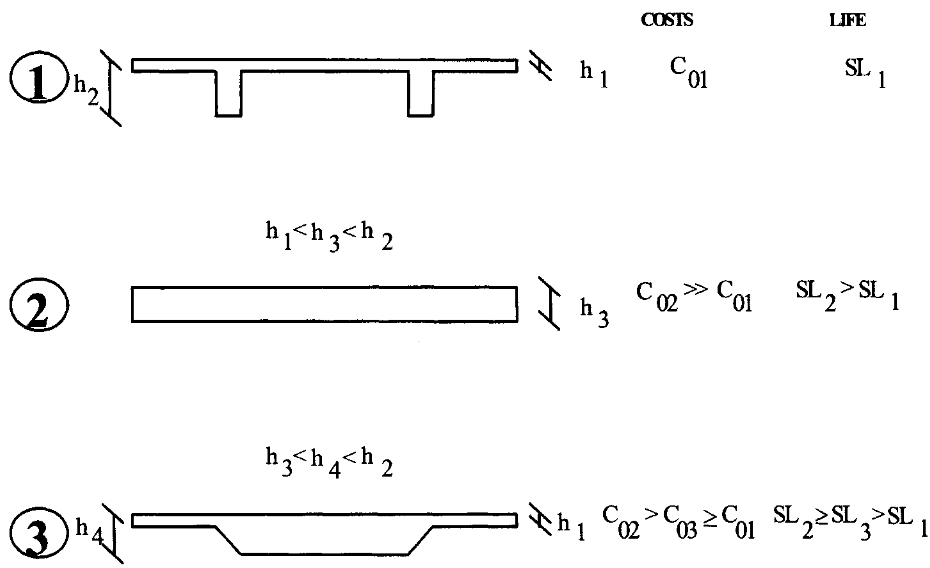


Figure 12-4. Structural resistance and durability: is it possible to have both?

life cycle (e.g., joints), through an estimate of the frequency with which certain type of interventions (e.g., cleaning corroded bars and replacing the cover layer) are performed or even through a prediction of the future current maintenance costs. Otherwise, one is forced to base decisions solely on initial costs, which frequently leads to long-term uneconomic solutions.

An example of such an option is presented in Figure 12-4 (de Brito and Branco 1998a). For a medium span, solution 1 is probably the least expensive to build, but it is also the most expensive to maintain and repair because the corners are vulnerable to corrosion and spalling, especially because the reinforcement is positioned near two outer surfaces. On the contrary, solution 2 is the most durable, but, for the same load-bearing capacity, it is the most expensive to build because its cross section is, of the three options, the one with the largest area. Solution 3 may well be the best choice because its initial costs are similar to those of solution 1 and it has durability similar to that of solution 2.

It would not be difficult to conceive of a parallel example relative to the itinerary design: flexible pavements are substantially less expensive than rigid pavements but require more rigorous maintenance over time and have a shorter service life. Only an economic study customized to the specific case under analysis will enable well-founded decision making.

12.5. Inspection Costs

12.5.1. The Inspection Strategy

The proposed bridge inspection strategy was outlined in Chapter 10 and consists of three inspection types:

1. routine, every 15 months, based fundamentally on direct visual observation of the bridge;
2. detailed, every 5 years, somehow more elaborate than routine one but still general;

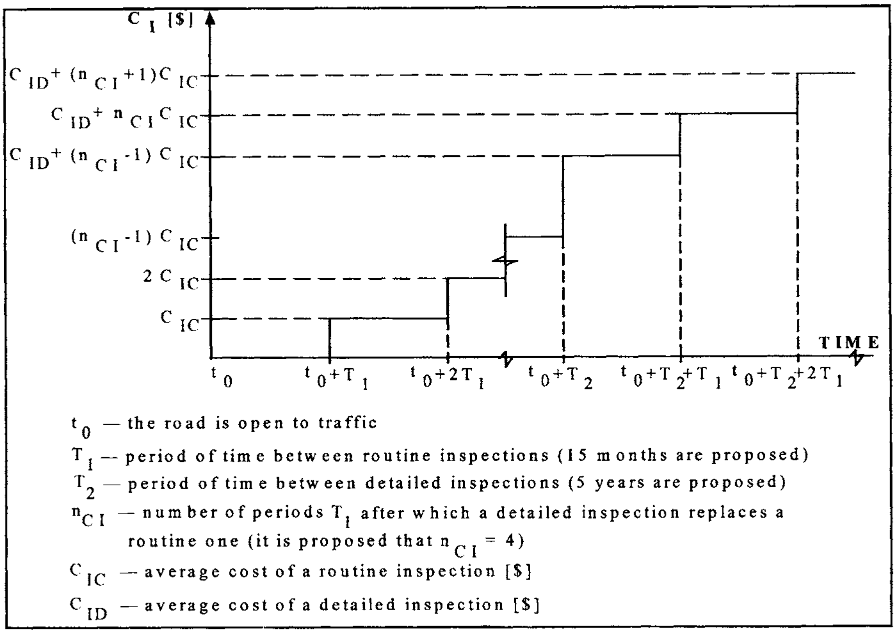


Figure 12-5. Representation of bridge inspection costs over time

3. structural assessment, preferably after detection of important structural deficiencies, elaborate and localized.

Since structural assessments are of a nonperiodic nature and are almost always associated with the performance of repair/rehabilitation works, it is considered more correct to include their expenses in repair costs. Therefore, inspection costs are based on the sum of the routine and detailed inspection expenses. The evolution of accumulated in time inspection costs is represented in Figure 12-5 (de Brito and Branco 1998a).

12.5.2. Definition

Inspection costs are divided into costs that concern the bridge C_{I_b} and costs that concern its area of influence C_{I_r} (de Brito 1992):

$$C_I = C_{I_b} + C_{I_r} \tag{12-31}$$

In turn, bridge inspection costs may be divided into (de Brito and Branco 1994):

$$C_{I_b} = C_{IL} + C_{IE} \tag{12-32}$$

Labor costs C_{IL} include all fees for the personnel who perform the inspections and those who feed the data into the computer database.

Equipment costs C_{IE} include amortization of the more expensive inspection equipment used at the site and also take into account the time for transporting it from one bridge to another. To these costs must be added an average cost of expendables (known for each type of equipment).

Even in the most developed countries of the world, land communications were subject to regular inspections only a relatively short time ago. Some of the techniques applied (load tests and tests of resistance to slippage, measurements of the longitudinal regularity) are still being developed complementarily to obtain the best yield. Finally, the economic analysis of communications is not discussed in depth here, because it corresponds to a very wide domain of knowledge. It is, however, the object of several studies within the scope of pavements management. Therefore, further details of fraction C_{I_r} of the inspection costs are not discussed.

12.5.3. Quantification and Prediction

12.5.3.1. Labor Costs

The quantification of labor costs is an easy task, provided there is adequate control of the inspection personnel assigned to each bridge. Otherwise, the sum of the expenses made with inspection personnel during each period of time is divided by each bridge inspected during the same period, according to a predefined criterion (e.g., the deck area or the inspection type). To approximately predict the same costs, a similar criterion is proposed. The labor costs may be divided into displacement costs (C_{ILD}) and testing costs (C_{ILT}) (de Brito 1992):

$$C_{IL} = (C_{ILD} + C_{ILT})\, k_{DB} \tag{12-33}$$

where

k_{DB} = corrective coefficient defined later in this chapter

The displacement costs C_{ILD} refer to the time spent by the inspection personnel to go from local headquarters to the bridge (de Brito 1992).

$$C_{ILD} = t_D n_p c_{MH} = \frac{2 l_{Db}}{v_i} n_p c_{MH} \tag{12-34}$$

where

t_D = time spent on the journey [h]

n_p = number of persons on the inspection team [man]

c_{MH} = average unit cost of the man-hour for the members of the inspection team [$/man-hour]

l_{Db} = distance from local headquarters to the bridge [km] (known for each bridge)

v_i = average speed of displacement of the inspection team (60 km/h is proposed)

Based on the experience of several management systems, for routine inspections it is proposed that $n_p = 2$. For detailed inspections, it is proposed that n_p depends on the bridge deck area: a minimum of 2 up to 1,000m^2 and an extra person for each additional 1,000 m^2 (Figure 12-6) (de Brito 1992).

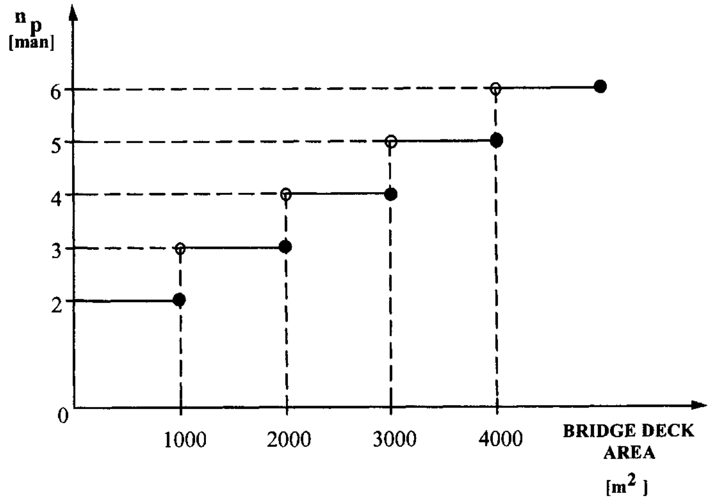


Figure 12-6. Estimated average number of persons in the inspection team for detailed inspections

As a consequence of the values proposed (de Brito 1992):

- routine inspections

$$\frac{C_{ILD}}{c_{MH}}[\text{man-hour}] = \frac{l_{Db}[\text{km}]}{15} \tag{12-35}$$

- detailed inspections (taking into account the deck area, Figure 12-7 (de Brito and Branco 1998a) is obtained).

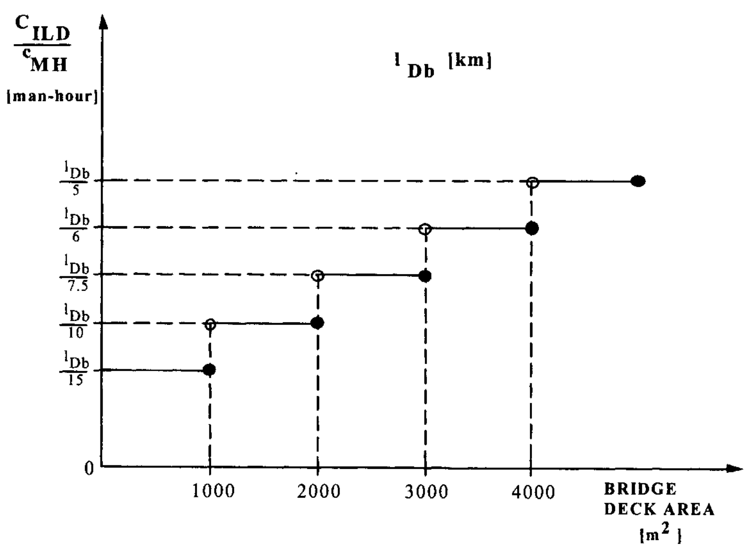
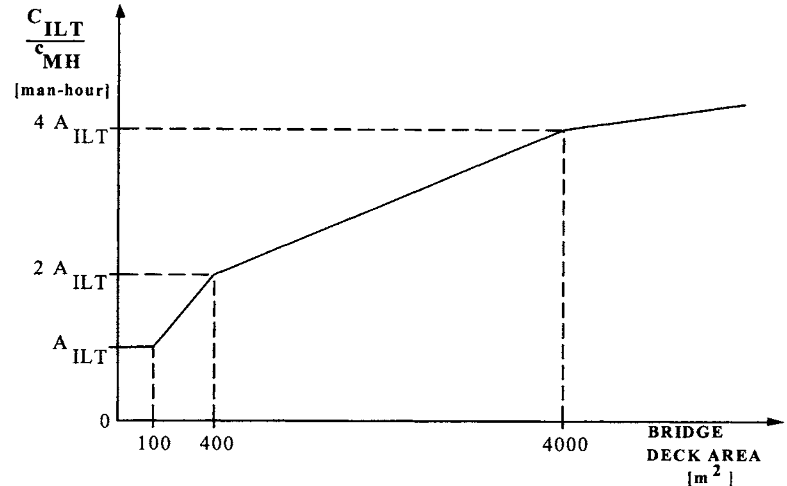


Figure 12-7. Average displacement costs for detailed inspections



A_{ILT} —minimum costs of performing the tests independently of the bridge deck area [man-hour]

Figure 12-8. Model for predicting the average testing costs of inspections

The testing costs C_{ILT} refer to the time spent by the inspection team at the bridge site. A relationship of the type represented in Figure 12-8 (de Brito and Branco 1998a) is proposed.

The coefficient A_{ILT} may be obtained either by statistical analysis or from existing management systems' experience. Based on the authors' experience with bridge in situ tests, it is proposed that $A_{ILT} = 4$ (2 hours of inspection time with a two-person team) for routine inspections and $A_{ILT} = 12$ (6 hours of inspection time with the same team) for detailed inspections.

The corrective coefficient k_{DB} refers to the time spent feeding the inspection results into the computer database and preparing the final report; again based on the authors' experience, a value of 1.5 is proposed, to be calibrated with time.

In the algorithm COSTS, the calculation of the inspection costs is preceded by reading a file with data specifically related to the inspection costs of each bridge under analysis and its corresponding itinerary (de Brito 1992):

- distance of bridge from local inspection headquarters (l_{Db});
- estimative of C_{I_r}/year in terms of percentage of C_{0C_r};
- year in which the service life of the bridge is expected to end (YOSB);
- actual values of C_{ILD}, C_{ILT}, C_{IL}, and C_{IE} differentiated by year for all years preceding the routine year (CY);
- actual values of C_{I_r} discriminated by year for all years preceding the routine year (CY);
- future values of C_{ILD}, C_{ILT}, C_{IL}, and C_{IE} predicted by the user (with no intervention from the system) between the years CY and YOSB, when this is the option for predicting costs;
- future costs values of C_{I_r} predicted by the user (with no intervention from the system) between the years CY and YOSB, when this is the option for prediction of the costs.

In a complete economic analysis, it is always necessary to estimate future inspection costs of the bridge. To do that, the user has four options (de Brito 1992):

- to use a linear regression based on costs registered for the latest years (up to a maximum of 10 years);
- to use his own linear variation for future costs with time;
- to let the system calculate all future costs according to the criteria described previously (Figure 12-9) (de Brito 1992);
- to provide future year-to-year costs based on his own estimates.

For the first two options, all future costs and their trends over time are calculated and provided as real prices for the current year and are later converted by the system to the corresponding year. For the last two options, the costs are calculated and are provided as real prices for the corresponding year. The third option is used automatically whenever the bridge inauguration year (YNC0 + 1) does not precede the current year (CY). The system schedules all future periodic inspections based on the year in which the bridge is opened to traffic and relates all the calculations to the calendar years thus obtained.

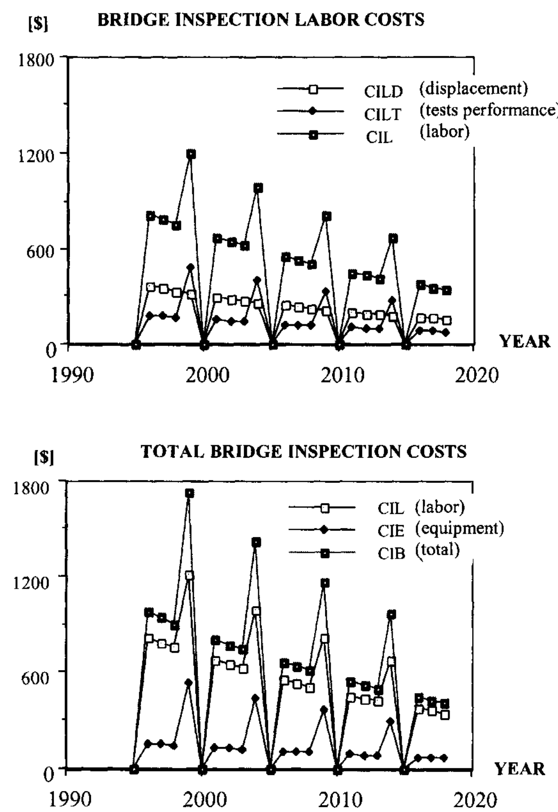


Figure 12-9. Example of prediction of the future inspection costs of bridge 1 (see Section 12.3.4.) in present value prices based on the criteria proposed (it is easy to spot the 5-year cycles correspondent to the detailed inspections and, inside them, the 15-month cycles correspondent to the routine inspections)

Using the algorithm COSTS, the criteria described here to calculate the labor costs C_{IL} are also valid in the determination of equipment costs C_{IE}.

12.5.3.2. Equipment Costs

As with labor costs, equipment costs are easily quantifiable if there is an efficient allocation of inspection equipment to each bridge. Otherwise, it is always possible to know the total equipment costs during a certain period and assign a fraction of these costs to each bridge inspected in the same period, using a criterion similar to the one proposed previously (based on the deck area and inspection type). To obtain simple estimates of these costs, the following criterion is proposed (de Brito 1992).

$$C_{IE} = \sum_{i=1} \frac{t_{E_i}}{SL_{E_i}} C_{E_i} + 2l_{Db} c_{km} \tag{12-36}$$

where

t_{E_i} = average time of use of equipment i in each inspection [h]

SL_{E_i} = active service life of equipment i [h]

C_{E_i} = total costs attributed to equipment i during its service life [$]

these costs include the purchase price, all expenses for maintenance and repair during its service life, and all expendables necessary for the equipment to function well during the same period

l_{DB} = distance of the bridge from the local inspection headquarters [km]

c_{km} = cost of transportation of the inspection personnel per kilometer [$/m] (official rate of displacement per km)

To facilitate the prediction of these costs, the following pattern for time of use of various equipment during an inspection is proposed (de Brito 1992):

- routine inspections

  80%—direct visual observation or tests whose equipment has negligible operating costs (hammer/chains, sclerometer, chemical indicators, displacement transducers, strain gauges, clinometers, etc.);

  20%—galvanic cell.

- detailed inspections

  50%—direct visual observation or tests whose equipment has negligible operating costs;

  20%—galvanic cell;

  10%—magnetometer;

  10%—ultrasonic pulse velocity test;

  10%—core extraction.

If the concept of active service life (SLEi) is not appropriate for certain types of equipment (those that deteriorate independent of whether they are used), Equation 12-36 may still be used by replacing active service life with service life and the average time of use with average time between two consecutive active uses.

It is important to note that, according to proposals made, the global inspection costs of a bridge can be predicted as a function of parameters that are well defined and are related to each individual bridge (the distance from local inspection headquarters and deck area).

12.5.3.3. Itinerary Inspection Costs

In an efficient inspection system, it is possible to assign a correct allocation of the total inspection costs (which are known) to each itinerary. In the long term it is expected that the relationship between the annual inspection costs of each itinerary and its construction costs is approximately the same for all stretches of the communications network. Therefore, it is possible to obtain a fixed parameter for the ratio (annual inspection costs/construction costs) (de Brito 1992):

$$C_{I_r}/\text{year} = x'\,\%\,C_{0C_r} \tag{12-37}$$

where

C_{0C_r} = itinerary construction costs [$];

x' = fixed parameter that needs to be tested in the system after obtaining a first approximation through the experience of the communications network inspection system already implemented (based on the authors' experience in their country, the figure 0.5 is proposed).

Based on a study of road inspection systems in working order, it is probably possible to obtain a more elaborate procedure to predict these costs (similar to the one proposed for bridges; Equations 12-32 to 12-36).

The inspection costs already spent for the area of influence of the bridge are read from the data file previously described, which is specifically related to the inspection costs of each bridge under analysis and its respective itinerary.

In a complete economic analysis, it would be necessary to estimate the future itinerary inspection costs. To do that, the user has four options (de Brito 1992):

- to use a linear regression based on costs registered for the latest years (up to a maximum of 10 years);
- to use his own linear variation for future costs with time;
- to let the system calculate all future costs according to the criteria described previously (Figure 12-10) (de Brito and Branco 1994);
- to provide future year-to-year costs based on his own estimates.

For the first three options, all future costs and their trends over time are calculated and provided as real prices for the current year and are later converted by the system to the corresponding year. For the last one, the costs are calculated and are provided as real prices for the corresponding year. The third option is used automatically whenever the bridge inauguration year (YNC0 + 1) does not precede the current year (CY).

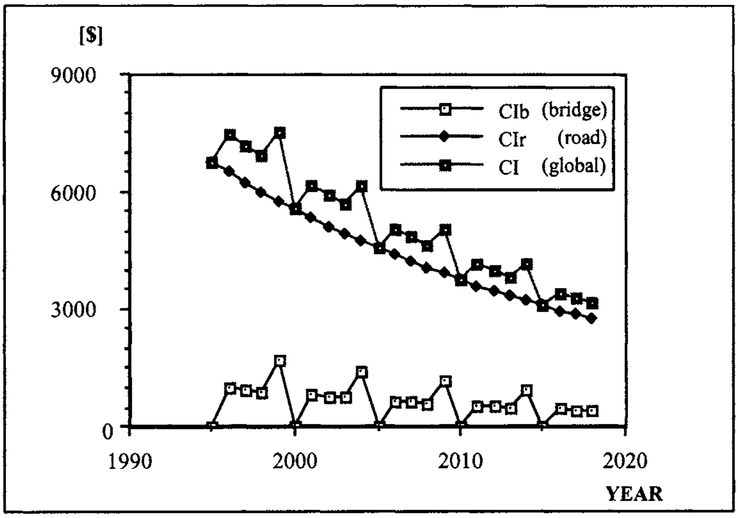


Figure 12-10. Prediction of the future inspection costs of bridge 1 (see Section 12.3.4.) and the corresponding itinerary in present value prices based on the criteria proposed

12.5.4. Relative Importance

Similar to the initial costs, the relative importance of the inspection costs within the bridge management system's scope is very small. These costs do not represent a significant percentage of the global costs and are (or should be) an inevitable operating expense. In principle, these costs are not affected by alternatives for structural repair that may be proposed and, therefore, are not included in the comparative cost analysis (Chapter 13).

However, the quantification and prediction of inspection costs can be useful when preparing an inspection budget or, at the end of each year, to determine how much was spent on the inspection of the bridges (and on each individual one, if possible) and the corresponding itinerary.

12.6. Maintenance Costs

12.6.1. Definition

Maintenance costs are divided into costs concerning the bridge C_{M_b} and costs concerning its area of influence C_{M_r} (de Brito 1992):

$$C_M = C_{M_b} + C_{M_r} \tag{12-38}$$

In Equation 9-22, maintenance costs were separated from repair costs, which raises the question of how to tell them apart. The fundamental concept is that maintenance costs do not involve any of the structural aspects of the bridge. To simplify the application of this concept, in the list of repair techniques presented in Chapter 10 (Table 10-3), each technique has been identified as either a maintenance repair or a structural repair. This means that, under any circumstances, the costs associated with a certain repair technique are always assigned to the same item.

Maintenance costs include all expenses for labor, materials, management, and supervision related to maintaining the functionality of the bridge as designed. Since the work performed is relatively simple, no design or post-application testing costs are expected. The sum of the maintenance costs rises regularly during the structure life, even though a slight increase is to be expected after each periodic inspection.

Maintenance costs for the bridge's area of influence of the bridge have been included, both those costs for current maintenance (patching the wearing surface, cracks sealing, elimination of concavities and other irregularities, etc. (Sharaf and Sinha 1984)) and those designated as structural in accordance with road terminology (repaving the wearing surface, replacement of the sub-basis, etc.). While the first costs detailed occur every year at an almost constant value, the second set of costs occurs at time intervals that correspond to the service life of the pavement.

12.6.2. Quantification and Prediction

12.6.2.1. Bridge Maintenance Costs

In an efficient maintenance system, it is possible to correctly allocate the total maintenance costs (which are known) to each bridge. In the long-term, it is to be expected that the relationship between annual maintenance costs for each bridge and initial costs for each bridge is approximately the same for all the network structures. Therefore, it is possible to obtain a fixed parameter for the ratio (annual maintenance costs/initial costs). This leads to a practical proposal for the prediction of maintenance costs (de Brito 1992):

$$C_{M_b}/\text{year} = x\ \%\ C_{0_b} \tag{12-39}$$

or

$$C_{M_b}/\text{year} = x'\ \%\ C_{0C_b} \tag{12-40}$$

where

C_{0_b} = total initial costs of the bridge [$]

C_{0C_b} = construction costs of the bridge [$]

x, x' = fixed parameters that need to be tested in the system after obtaining a first approximation based on the experience gained from the maintenance systems already in use (values between 1.0 and 2.0 are proposed for x')

Since it is expected that maintenance costs will increase as the bridge ages, a proposal for this evolution is presented graphically in Figure 12-11 (de Brito and Branco 1998a).

These values are compatible to those obtained by the Federal Germany Republic highways system, where annual expenses for bridge maintenance vary between 1.1% and 1.7% of the reconstruction costs (Wicke 1988). In the 1988–1992 maintenance plan of the Swedish National Roads Administration, a percentage of 0.7 of the replacement cost was considered for the maintenance of the bridges (Lindladh 1990). In another study in the Netherlands, it was concluded that the real annual maintenance costs of a bridge used to calibrate the system varied between 1.0% and 2.0% of its initial cost (Van der Toorn and Reij 1990).

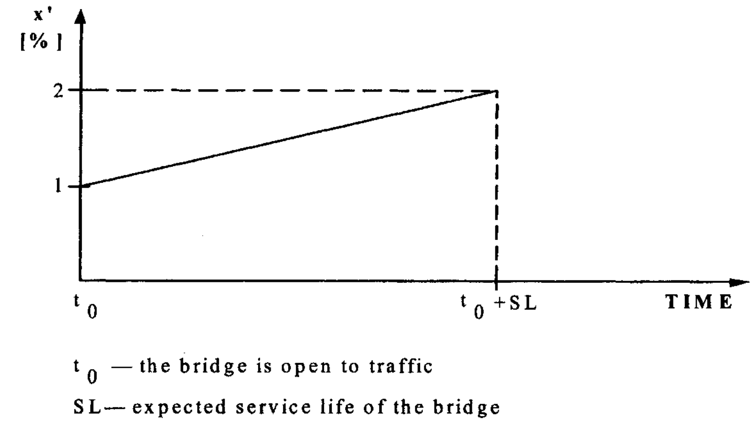


Figure 12-11. Proposed evolution during the service life of the bridge of the ratio (annual maintenance costs : construction costs) percentage, x'

Another possible way to predict bridge annual maintenance costs is graphically represented in Figure 12-12 (de Brito and Branco 1998a). The coefficient A_M may be obtained either by statistical analysis or it may be based on the experience of existing maintenance systems. The value of A_M is to be considered constant with time in current prices (i.e., its evolution with time is parallel to the inflation).

In the algorithm COSTS, the calculation of maintenance costs is preceded by the reading of a file with data specifically related to the maintenance costs of each bridge under analysis and each corresponding itinerary (de Brito 1992):

- value of the x' coefficients (defined in Figure 12-11) (de Brito and Branco 1998a) at the beginning and at the end of the bridge's service life;

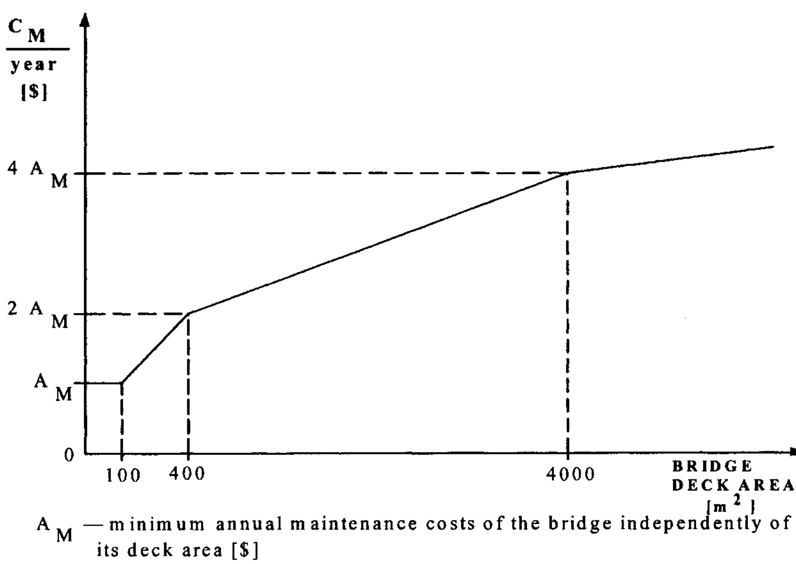


Figure 12-12. Model for prediction of the annual bridge maintenance costs

- values of the coefficients y and y' and of time interval Δt (defined in Figure 12-15) (de Brito and Branco 1998a), considered constant during the service life of the bridge (and of the itinerary);
- actual values of C_{M_b} differentiated by year for all years preceding the current year (CY);
- actual values of C_{M_r} differentiated by year for all years preceding the current year (CY);
- future values of C_{M_b} predicted by the user (with no intervention from the system) between the years CY and YOSB (end of the bridge's service life), when this is the option for predicting costs;
- future values of C_{M_r} predicted by the user (with no intervention from the system) between the years CY and YOSB, when this is the option for predicting costs.

In a complete economic analysis, it is always necessary to estimate future maintenance costs. To do that, the user has five options (de Brito 1992):

- to use a linear regression based on costs registered for the latest years (up to a maximum of 10 years);
- to use his own linear variation for future costs with time;
- to let the system calculate all future costs according to the criteria represented in Figure 12-11 (Figure 12-13) (de Brito 1992);
- to let the system calculate all future costs according to the criteria represented in Figure 12-12 (Figure 12-14) (de Brito 1992);
- to provide future year-to-year costs based on his own estimates.

For the first four options, all future costs and their trends over time are calculated and provided as real prices for the current year and are later converted by the system to the corresponding year. For the last option, the costs are calculated and are provided as real

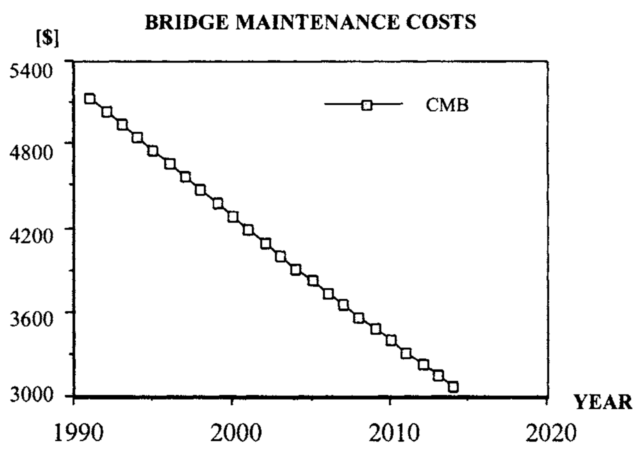


Figure 12-13. Example of prediction of the future maintenance costs of bridge 2 (see Section 12.3.4.) in present value prices based on the criteria represented in Figure 12-11

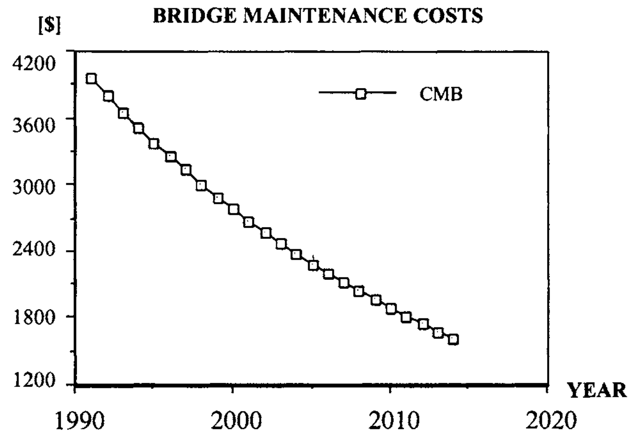


Figure 12-14. Example of prediction of the future maintenance costs of bridge 2 (see Section 12.3.4.) in present value prices based on the criteria represented in Figure 12-12

prices for the corresponding year. The fourth option is used automatically whenever the bridge inauguration year (YNC0 + 1) does not precede the current year (CY).

12.6.2.2. Itinerary Maintenance Costs

The distribution of the global maintenance costs for the various itineraries is made in a simple way, as long as there is some control over the work. Some studies (Sharaf and Sinha 1984) and (Sharaf et al. 1985) indicate that the annual current maintenance costs depend on the age of the pavement, the volume of daily traffic (generally proportional to road width), and weather conditions. However, within the limits of desired precision, it is acceptable to admit that the annual current maintenance costs are constant in time (in a present prices value analysis minus the interest rate), a simplification that is always adopted for long-term economic analyses.

Repaving costs will occur with intervals that are more or less constant and correspond to the average service life of the pavement. In the model proposed, these costs are considered only for flexible pavements. Both current maintenance costs and repaving costs are considered, at current prices, as fixed percentages of the construction costs of the itinerary C_{0C_r} (Figure 12-15) (de Brito and Branco 1998a).

The values of y, y', and δt should be progressively calibrated by a comparison with the real values obtained by the communications management system. As a first approximation, the values of 0.5, 5.0, and 5 years, respectively, are proposed. These coefficients can be made to depend on other parameters (e.g., daily traffic volume), if data are collected to calibrate them. The values proposed aim at expenses with maintenance a bit higher than the registered average in the OCDE countries based on a study published in 1973 (RRG 1973): 5% of the construction costs over a period of 30 years. However, since then the trend has been to increase expenses for maintenance of the infrastructure, because previous costs proved insufficient to maintain an acceptable level of service, particularly when taking into account ever-growing demands.

The maintenance costs of the area of influence of the bridge already spent are read from the data file previously described and are specifically related to the maintenance costs of each bridge under analysis and its respective itinerary.

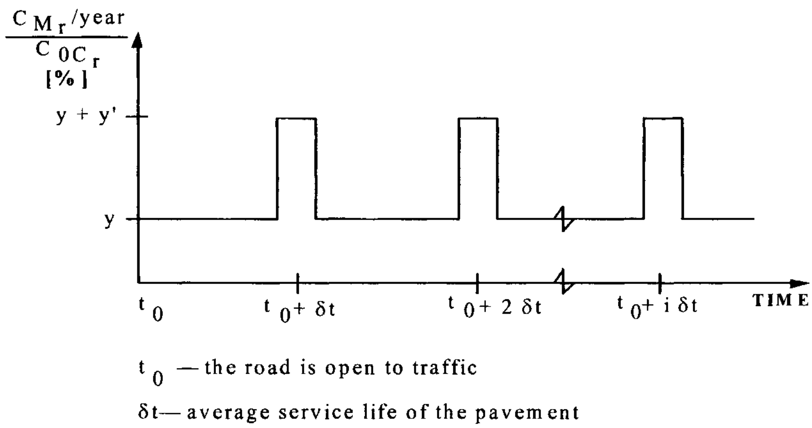


Figure 12-15. Proposed evolution with time of the ratio (annual maintenance costs : construction costs) percentage for the bridge itinerary

In a complete economic analysis, it is necessary to estimate future itinerary maintenance costs. To do that, the user has four options (de Brito 1992):

- to use a linear regression based on costs registered for the latest years (up to a maximum of 10 years);
- to use his own linear variation for future costs with time;
- to let the system calculate all future costs according to the criteria represented in Figure 12-15 (Figure 12-16) (de Brito and Branco 1994);
- to provide future year-to-year costs based on his own estimates.

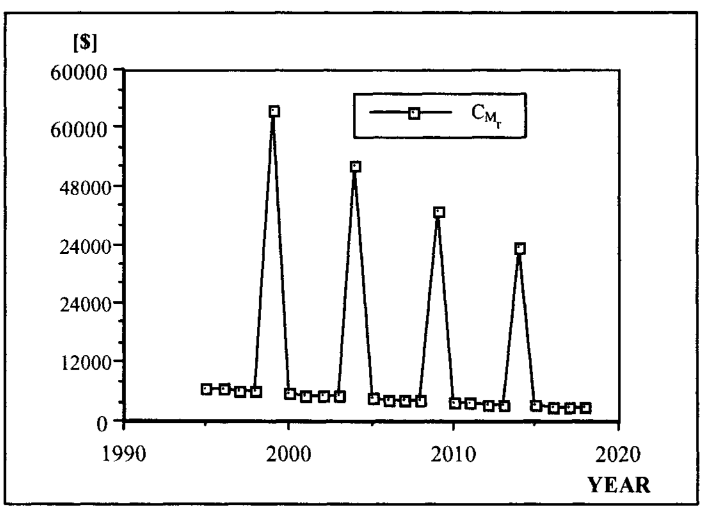


Figure 12-16. Example of prediction of the future maintenance costs of the part of the itinerary (6 km of flexible pavement) assigned to bridge 1 (see Section 10.3.4.) in present value prices based on the criteria represented in Figure 12-15

For the first three options, all future costs and their trends over time are calculated and provided as real prices for the current year and are later converted by the system to the corresponding year. For the last option, the costs are calculated and provided in real prices for the corresponding year. The third option is used automatically whenever the bridge inauguration year (YNC0 + 1) does not precede the current year (CY).

12.6.3. Relative Importance

Maintenance costs, like inspection costs, must be faced as an inevitable consequence of operating a bridge and its respective itinerary. However, their relative importance is much higher and must be taken into account. When several options for designing a new bridge are being considered, it is fundamental to evaluate the influence of the design on future maintenance costs to prevent decisions based uniquely on initial costs, as discussed in Section 12.4.3. The experience acquired or the use of deterioration mathematical models may help to do so. However, with the present state of the art, only very rough estimates are possible.

When comparing repair options that lead to different residual service lives, the prediction of maintenance costs based on a law similar to that presented in Figure 12-11 (de Brito and Branco 1998a) may be useful. It would penalize older bridges. The quantification and prediction of maintenance costs may also be useful when preparing a maintenance budget or, at the end of each year, to determine how much has been spent on maintaining each bridge and the network.

12.7. Repair Costs

12.7.1. Definition

As discussed earlier, only the costs resulting from rehabilitation of a structural nature have been considered within the repair costs. The objective of this type of work may be current repair to eliminate deficiencies unpredicted at the design stage; or enhancement of the bridge service level, either by strengthening or by widening the deck. Separating maintenance from repair is explicitly discussed in the repair techniques list presented in Chapter 10 (Table 10-3).

From the previous description of maintenance costs for the area of influence of the bridge, the result is that no itinerary repair costs are considered in the analysis proposed here. Repaving costs are included in the maintenance costs, and those related to changes in the pavement type, the road layout, or its width are treated only in Chapter 13 (rehabilitation/replacement subsystem) as costs necessary to enhance the bridge's service life (particular cases).

The repair costs of the bridge may be divided into (de Brito and Branco 1994):

$$C_R = C_{RSA} + C_{RSR} \tag{12-41}$$

As discussed in Section 12.5.1., it is considered more logical to include structural assessment costs (C_{RSA}) in repair costs, because it immediately precedes the repair work. The costs C_{RSA} include all fees for the personnel who perform the inspection, the cost of amortization and expendables of the equipment used, and the fees involved in the preliminary structural design of the various options of repair or enhancement for the bridge under consideration.

Structural repair costs C_{RSR} include all the labor, materials, equipment, administration, and quality control costs involved in the construction of the option that is judged the best by the decision system and is approved by the authorities.

12.7.2. Quantification and Prediction

12.7.2.1. Structural Assessment Costs

The quantification of structural assessment costs is relatively easy. The costs involved in the inspection itself (labor and equipment costs) are well known, since the bridge that is the object of the inspection is perfectly identified. The task is further facilitated if the inspection is awarded to a firm or laboratory outside the road administration services. If it is necessary to impose temporary traffic restrictions at the bridge during the inspection (shutting off all or some of the lanes), there are other costs involved that are more difficult to quantify. Some proposals concerning this subject are made in Section 12.8.

Structural analysis costs can be easily quantified as fees paid to an outside consulting firm or as an estimate of the salaries paid by the management authorities to their technical personnel entrusted with the job.

It is impossible to predict structural assessment costs before the need occurs, that is after structural defects of some significance have been detected during a periodic inspection. Even then it is very difficult to estimate the costs beforehand because the assessment depends to a great extent on the type of defect that has triggered it. An approximate prediction can be made if the following parameters are estimated: equipment and personnel needed; information wanted; number of structural elements (or deck area) to be investigated. If all these elements can be collected (it must be noted that in inspections of this type there are many surprises and that the time and money spent in performing it may have nothing to do with the prediction), the use of some of the expressions proposed in Section 12.5.3. and some good sense may make it feasible to estimate inspection costs.

Another possibility for predicting these costs is to consider them as a fixed percentage of the repair work cost. Only the experience gained from several bridge management systems can enable a correct estimate of this percentage. To predict structural analysis costs, it is also proposed that a percentage of the repair work costs according to the current fees for this type of work be used. As a first approximation, the value of 10% is proposed (also including structural analysis costs).

In the algorithm COSTS, the estimate of the costs C_{RSA} is made using a percentage of the total repair costs C_R provided by the user or calculated by the system for each bridge.

12.7.2.2. Structural Repair Costs

Structural repair costs are naturally quantified as the works proceed or at its onset, according to the type of contract in effect. A structural repair cost prediction is impossible before the conclusion of the structural analysis studies and the various repair options available are defined, based on definitive work quantities. For each option, a list of work must be prepared; for each one of them, the following items are needed:

- repair techniques (according to the list already shown in Table 10-3);
- quantities (m^2, m^3, etc., according to the respective repair form); this prediction is fallible and must be conservative; the inspector's experience is fundamental.

The multiplication of quantities by unit prices contained in the repair forms (subject to updating) gives the cost of each job. The fixed costs of some repair techniques must be included and whenever possible spread to the different works made on the same bridge.

The global cost of each option is the sum of the elemental corresponding work. Several alternatives must be tested:

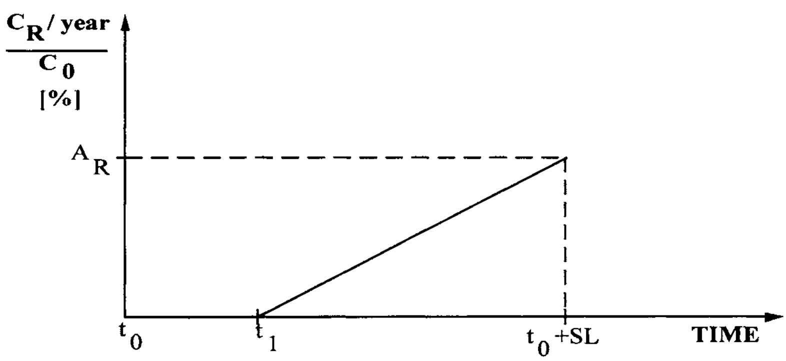
- to use repair type A for the bridge assuming it has an additional *x* years before it is replaced (the type of repair may include strengthening or deck widening);
- to use repair type B to keep the bridge in operation for an additional *y* years before replacement (see case study in Section 13.4.4.1.);
- and so forth;
- immediate replacement.

The cost of each option (estimated as discussed previously) is the input of the decision system and must be compared with the other costs and benefits associated with them, which also need to be quantified (see 12.8).

An easy-to-apply proposal to estimate the long-term global annual repair costs and their evolution is represented in Figure 12-17 (de Brito and Branco 1998a).

The coefficient A_R can be obtained by statistical analysis or through the experience gained through existing management systems. As a first estimate, it is proposed that A_R = 5.0 and $t_1 - t_0$ = 10 years.

A publication adapted to situations in the United States (Ahlskog 1988) states that, with current interest rates, it is always better to rehabilitate and institute rigorous maintenance procedures for a bridge than to replace it. It is noted that it is in the public interest to spend up to about 6% of the replacement cost in annual maintenance and repair of bridges to keep than in service. Reduction of the budget for replacement of the existing bridges in favor of the budgets for maintenance and rehabilitation is also recommended. This value seems a bit high when compared with those currently quoted and may reflect the environmental aggressiveness and traffic intensity registered by the network that is the object of the study.



t_0 —the bridge is open to traffic

SL —expected service life of the bridge

$t_1 - t_0$ —period during which no repair work at the bridge is supposed to occur

A_R—percentage of the annual repair costs at the end of the service life

Figure 12-17. Proposed evolution along the bridge service life of the ratio (annual repair costs : initial costs), percentage

In the algorithm COSTS, the calculation of repair costs is preceded by the reading of a file with data specifically related to the repair costs of each bridge under analysis (de Brito 1992):

- estimates of C_{RSA} and C_{RSR} in terms of percentages of C_R;
- estimates of A_R and $t_1 - t_0$;
- actual values of C_{RSA} and C_{RSR} differentiated by year for all years preceding the current year (CY);
- future values of C_{RSA} and C_{RSR} predicted by the user (with no intervention from the system) between the years CY and YOSB (end of the bridge's service life), when this is the option for predicting costs.

In a complete economic analysis, it is always necessary to estimate the bridge future repair costs. To do that, the user has five options (de Brito 1992):

- to use a linear regression based on costs registered for the latest years (up to a maximum of 10 years);
- to use his own linear variation for future costs with time;
- to let the system calculate all future costs according to the criteria represented in Figure 12-17 (Figure 12-18) (de Brito and Branco 1994);
- to provide a prediction of repair actions in discrete years;
- to provide future year-to-year costs based on his own estimates.

For the first three options, all future costs and their trends over time are calculated and are provided as real prices of the current year and are later converted by the system to the

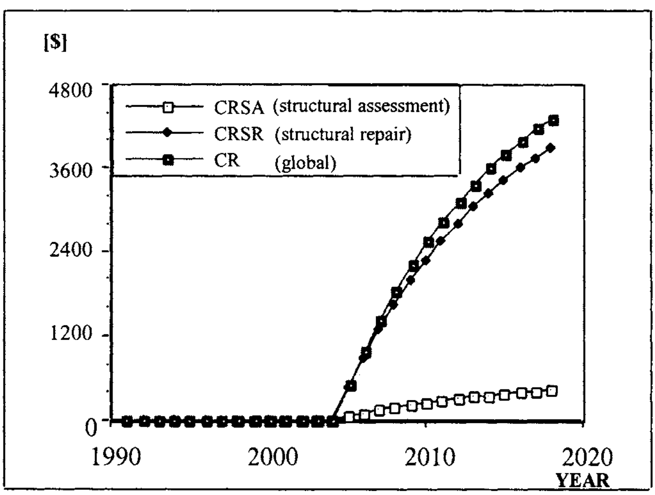


Figure 12-18. Example of prediction of the future repair costs of bridge 1 (see Section 12.3.4.) in present value prices based on the criteria represented in Figure 12-17

corresponding year. For the last two options (not different in essence), the costs are calculated and are provided as real prices of the corresponding year. The third option is used automatically whenever the bridge inauguration year (YNC0 + 1) does not precede the current year (CY).

12.7.3. Relative Importance

Unlike maintenance costs, repair costs are not distributed over time but rather are localized during certain periods of the structure's life (after the detection of a significant structural or functional defect). They are also significant in the sense that they may represent a sum of the same scale as the initial costs. Therefore, decisions that involve repair cannot be made lightly and demand in-depth preliminary studies (structural assessment and structural design).

It is impossible to obtain sufficiently accurate estimates of these costs before there are results from the structural assessment. Nevertheless, when comparing options that lead to different expectations of residual life, the estimate of long-term repair costs according to a trend similar to that presented in Figure 12-17 (de Brito and Branco 1998a) may be useful. This, of course, would penalize older bridges.

12.8. Failure Costs

12.8.1. Definition

As discussed in Section 9.3., the failure costs are incurred when the bridge does not completely fulfill design expectations (i.e., its predicted service life). In other words, a bridge is not built with the objective of being safe from the structural point of view but rather to be used by the traffic predicted. Structural safety is not an end but a means, and the most important objective of the bridge is its functionality (de Brito 1992).

Failure costs may be divided into (de Brito and Branco 1994):

$$C_F = C_{FSF} + C_{FFF} + C_{FEI} \tag{12-42}$$

The structural failure costs CFSF include all money expended as a result of a structural collapse of the bridge (or a situation in which such a collapse is imminent and the bridge must be closed to traffic) and, because it must be replaced immediately, the predicted residual service life is lost. It does not include any costs connected with the passage of traffic (these are included in functional failure costs), except those that are due to its interruption during the period when the existing bridge collapsed until the opening of a new bridge to traffic. These costs are totally attributable to the bridge under analysis because its collapse was what jeopardized the normal service level of the itinerary, not the condition of the pavement of the itinerary or the condition of the remaining bridges.

The functional failure costs CFFF correspond to the value attributed to the fact that not all the predicted traffic can use the bridge (or that its design speed must be reduced during certain periods of the day or as a result of a defect). The costs of detouring traffic and the consequent delays caused in adjoining bridges must also be taken into account. These costs are also assignable in principle to the bridge under analysis because any anomalies in the normal flow of traffic are due exclusively to the bridge and not to the remaining itinerary length.

The environmental impact and social costs C_{FEI} correspond to the value attributed to the consequences of constructing the itinerary and the bridges within it in the environment and its population. Their quantification is both extremely difficult and polemic. Only part of these costs must be assigned to the bridge under analysis according to its share of responsibility.

12.8.2. Quantification and Prediction

12.8.2.1. Structural Failure Costs

General Definition

The question of quantifying the costs of a structural failure does not come up naturally within the scope of management systems because it is not anticipated that any bridge will collapse. However, the prediction of these costs is fundamental and a practical way of predicting them must be found.

A structural failure occurs when resistance R is exceeded by the internal effect S resulting from exterior actions (Figure 12-19). Since both of these values are probabilistic, they are associated with density functions $f_R(r)$ and $f_S(s)$, respectively. It is possible to associate a probability P_f to the structural failure.

$$P_f = P(P - S \leq 0) = \int_{-\infty}^{+\infty} F_R(x) f_s(x)\,\mathrm{dx} \tag{12-43}$$

where

$F_R(r)$ = R distribution function

The structural failure of a bridge is much more complex than simply the failure of a certain material in a known cross section caused by a well-quantified external action. Bridges are generally complex redundant structures (even though they are easier to analyze than buildings), which does not make it any easier to calculate P_f, which is seen here as the probability of the bridge being put out of service as a result of an irrecoverable structural deficiency. P_f depends on the age of the bridge and on its degradation level, and that influence must be explicitly taken into account in the management system.

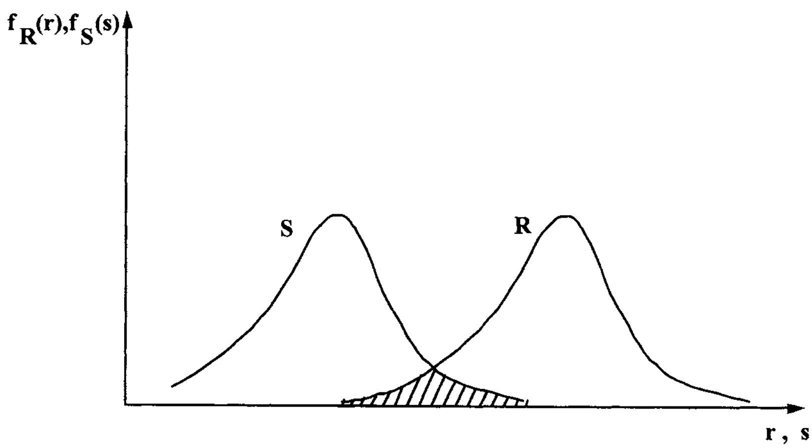


Figure 12-19. Simplified graphic representation of the failure probability (dashed area) for the fundamental case

$$C_{FSF} = P_f\, C_{FF} \tag{12-44}$$

where

C_{FF} = total estimated cost of the bridge's actual collapse (or of the end of its service life before expected) [$]

The cost of the collapse can be divided into (de Brito and Branco 1994):

$$C_{FF} = C_{FFR} + C_{FFL} + C_{FFH} \tag{12-45}$$

The bridge replacement costs C_{FFR} include all the extra expenses caused by the necessity of immediately replacing a bridge that theoretically still had a few remaining years of service. To estimate the costs for the immediate collapse and replacement of the bridge, a comparison must be made with the present situation (in which the bridge was to be replaced in only a few years). The costs that arose as a result of traffic interruption after the collapse must also be included.

The value of lost lives and vehicle and equipment costs C_{FFL} is the conventional value attributed to the lives of people (or what society is prepared to save each human live) who were using the bridge at the time of its sudden collapse.

The architectural/cultural/historical costs C_{FFH} are a way of overevaluating bridges that are especially significant from these points of view. An increase in the global collapse costs CF may result in an overevaluation of the existing bridge and weaken the demolition/replacement option in favor of repair/strengthening as the appropriate bridge option.

Bridge Replacement Costs

The question is how to quantify C_{FFR} when there is a residual service life RL_2 for the bridge. The costs C_{FFR} can be defined as the difference between the following two costs, pegged to the year in which the possibility of the bridge collapsing was put forward (de Brito 1992).

1. immediate collapse and replacement

$$C_1 = C_{0_1} + C_{I_1} + C_{M_1} + C_{R_1} + C_{F_1} - RV_1 \tag{12-46}$$

where

C_1 = total costs of option 1 [$]

C_{0_1} = initial costs of the new bridge [$]

C_{I_1} = inspection costs of the new bridge during the residual service life of the existing bridge (if it did not collapse) period [$]

C_{M_1} = maintenance costs of the new bridge during the same period [$]

C_{R_1} = repair costs of the new bridge during the same period [$]

C_{F_1} = failure costs of the new bridge during the same period [$]

RV_1 = residual value of the new bridge at the end of the residual service life of the existing one, if it did not collapse [$]

2. replacement of the existing bridge at the end of its predicted residual service life

$$C_2 = C_{I_2} + C_{M_2} + C_{R_2} + C_{F_2} - RV_2 \tag{12-47}$$

where

C_2 = total costs of option 2 [$]

C_{I_2} = inspection costs of the existing bridge during its predicted residual service life [$]

C_{M_2} = maintenance costs of the existing bridge during the same period [$]

C_{R_2} = repair costs of the existing bridge during the same period [$]

C_{F_2} = failure costs of the existing bridge during the same period [$]

RV_2 = residual value of the existing bridge at the end of its predicted residual service life (supposedly nil) [$]

The bridge replacement costs are then given by (de Brito 1992):

$$C_{FFR} = C_1 - C_2 \tag{12-48}$$

Equations 12-46 and 12-47 may be simplified if certain hypotheses are accepted. To quantify C_{0_1}, it can be assumed that the new bridge is exactly like the existing bridge. A simple solution is to use the existing bridge cost at real prices value taking into account the inflation rate of the intervening years (de Brito 1992).

$$C_{0_1} = C_{0_2} \prod_{i=1}^{(SL_2 - RL_2)} (1 + f_{i-1}) \tag{12-49}$$

where

C_{0_2} = real prices—initial costs of the existing bridge when it was built [$]

SL_2 = total service life of the existing bridge [year(s)]

RL_2 = present residual service life of the existing bridge [year]

$(SL_2 - RL_2)$ = present age of the existing bridge [year]

f_i = inflation rate corresponding to year i

This equation can be further simplified in the following way (de Brito 1992):

$$C_{0_1} \approx C_{0_2}(1 + f)^{(SL_2 - RL_2)} \tag{12-50}$$

if an average value f is used for the inflation rate during the existing bridge's residual service life.

The costs C_{I_1} and C_{I_2} are easy to predict if the formulation proposed in 12.5. is used. They can be eliminated (in the period subsequent to the beginning of the service life of the new bridge) in Equations 12-46 and 12-47 ($C_{I_1} = C_{I_2}$), if it is accepted that the periodic inspections and their corresponding costs are not affected by the age of the bridge, which is perfectly reasonable.

The costs C_{M_1} and C_{M_2} can also be predicted if the formulation proposed in 12.6. is used. Figure 12-11 (de Brito and Branco 1998a) would be particularly useful within this context to take into account an increase in the maintenance expenses with the bridge's age. However, if such costs are not considered relevant (or if the proposal schematically represented in Figure 12-12 (de Brito and Branco 1998a) is opted for), they may be eliminated (in the period subsequent to the beginning of service life of the new bridge) in Equations 12-46 and 12-47 ($C_{M_1} = C_{M_2}$).

To estimate C_{R_1} and C_{R_2} in the long term, Figure 12-17 (de Brito and Branco 1998a) could provide a possible solution because it considers an increase in repair costs with the bridge's age and relates it to the known initial costs.

The costs C_{F_1} and C_{F_2} may be eliminated from this formulation (in the period subsequent to the beginning of the service life of the new bridge) in order to simplify it and because, in the long run, it is expected that they will be identical for both situations. However, it is always necessary to quantify C_{F_1} and C_{F_2} in the period that extends from the collapse of the existing bridge to the new bridge being opened to traffic.

Finally, the value RV_1 can be estimated, using the hypotheses that the amortization is constant over time as well as the utility of the bridge (benefits-costs) (de Brito 1992):

$$RV_1 = \frac{\frac{SL_1 - RL_2}{SL_1} C_{0_1}}{\prod_{i+1}^{RL_2} (1 + f_{i-1})} \tag{12-51}$$

where

C_{0_1} = initial costs of the new bridge [$]

SL_1 = total service life of the new bridge [year]

RL_2 = residual service life of the existing bridge [year]

$(SL_1 - RL_2)$ = residual service life of the new bridge at the predicted end of the service life of the existing one [year]

f_i = inflation rate corresponding to year i

This equation can also be simplified in the following way (de Brito 1992):

$$RV_1 = \frac{\frac{SL_2 - RL_2}{SL_1} C_{0_1}}{(1 + f)^{RL_2}} \tag{12-52}$$

if an average value f is used.

In principle, $RV_2 = 0$.

The determination of costs C_{FFR} can be substantially simplified if they are considered equivalent to the present residual value of the existing bridge RV_2' (de Brito 1992):

$$RV_2' = \frac{RL_2}{SL_2} C_{0_2} \prod_{i+1}^{SL_2 - RL_2} (1 + f_{i-1}) \tag{12-53}$$

where

C_{0_2} = real prices initial costs of the existing bridge when it was built [$]

SL_2 = total service life of the existing bridge under normal circumstances [year]

RL_2 = present residual service life of the existing bridge [year]

$(SL_2 - RL_2)$ = present age of the existing bridge [year]

f_i = inflation rate corresponding to year i

This equation is valid only if the hypotheses used in Equation 12-51 are also valid. This formulation, although simpler than the formulation presented previously (Equation 12-48), has the disadvantage that it only takes into account in the calculation of the bridge replacement costs the initial costs of rebuilding (and therefore gives different results). It can also be further simplified (de Brito 1992):

$$RV_2' = \frac{RL_2}{SL_2} C_{0_2} (1 + f)^{(SL_2 - RL_2)} \tag{12-54}$$

if an average value f is used.

In the algorithm COSTS, the calculation of failure costs is preceded by the reading of a file with data specifically related to the failure costs of each bridge under analysis and its corresponding itinerary (de Brito 1992):

- total length of the bridge (l_b);
- design traffic speed at the bridge (v_b);
- coefficient k_{FFH} (defined further in Equation 12-57);
- total number of lanes on the bridge (n_l);
- number of separated lanes on the bridge (n_f);
- average rate of accidents in the itinerary (r_a) and in the detours (r_{aD});
- average cost of vehicle fuel in the itinerary (c_{fv}) and in the detours (c_{fvD});
- average maintenance cost of the vehicles in the itinerary (c_{mv}) and in the detours (c_{mvD});
- real values of annual traffic volume TRAF (that have crossed the bridge) and TRAFD (that have been detoured to other bridges) differentiated by year for all years preceding the current year (CY);

- description of the distribution of daily traffic volume for an average 24-hour working day including the rush hours (Figure 12-23) (de Brito 1992);
- description of the distribution by weight of traffic using the bridge (Figure 12-32) (de Brito 1992);
- description of the adjoining bridges (alternative routes), to which traffic on the bridge under analysis may have to be detoured when the need arises (with an indication of the percentages for each bridge or route relative to normal traffic and heavy trucks and the detour length);
- load parameter Q_0 that defines the design load capacity of the bridge;
- definition of the fraction of traffic that is detoured from the bridge when it is under repair;
- future traffic volume values TRAF predicted by the user (with no intervention from the system) between the years CY and YOSB (end of the bridge's service life), if this is the option for predicting traffic.

To predict failure costs, it is indispensable to estimate the structural failure probability P_f. For that, the user has two options (de Brito 1992):

- to use his own linear variation for structural failure probability with time;
- to use the system's predefined linear variation (as a first approximation, $Pf = 10^{-4}$ at the beginning of the bridge's live and 5×10^{-4} at the end).

These options are still very limiting. In the future, the system will include a means to derive this probability based on the results of the nonperiodic inspection (structural assessment) and other data. To do that, it is necessary to implement reliable mechanisms of degradation and to quantify their influence in structural failure probability (see Chapter 13).

In the prediction of the future traffic volume, the user has three options (de Brito 1992):

- to use a linear regression based on volumes registered for the latest years (up to a maximum of 10 years);
- to use his own linear variation for future traffic volume with time;
- to provide future year-to-year traffic volume based on his own estimates.

When the current year (CY) precedes the year in which the bridge is to be opened to traffic (YNC0 +1), there are no stored data available about daily traffic volume distribution during an average 24-hour working day. This distribution varies immensely from one bridge to another as a function of local conditions. This information is provided to the system, for which the user has two options (de Brito 1992):

- to use his own traffic distribution;
- to use the predefined distribution for an average working day (with two rush hour periods, at early morning and late afternoon), and a rate that is three times lower on public holidays (Figure 12-20) (de Brito and Branco 1998a).

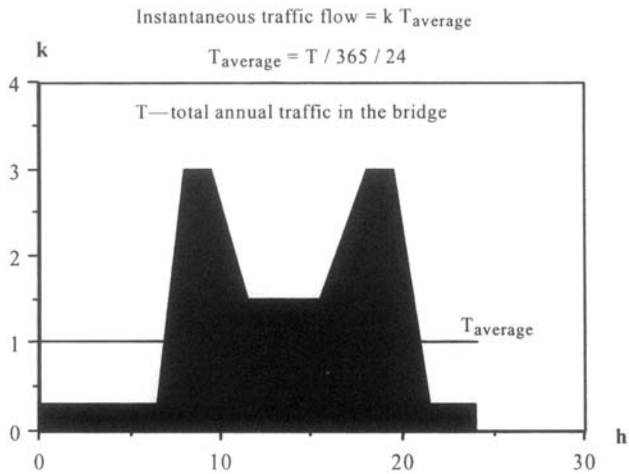


Figure 12-20. Proposed default daily distribution of traffic flow at the bridge during a typical work day (used for both bridges of the example of application)

When CY ≤ YNC0 +1, there is no information on file about the weight distribution of traffic using the bridge. To feed this information into the system, the user has two options (de Brito 1992):

- to use his own traffic distribution;
- to use the predefined distribution (with about two-thirds of all vehicles having less than 25 kN and a linearly decreasing variation for heavier weights) (Figure 12-21) (de Brito and Branco 1998a).

In the algorithm COSTS, the calculation of costs C_{FFR} is made year by year based on the formulation derived from Equations 12-46 and 12-48, on the proposals for prediction of

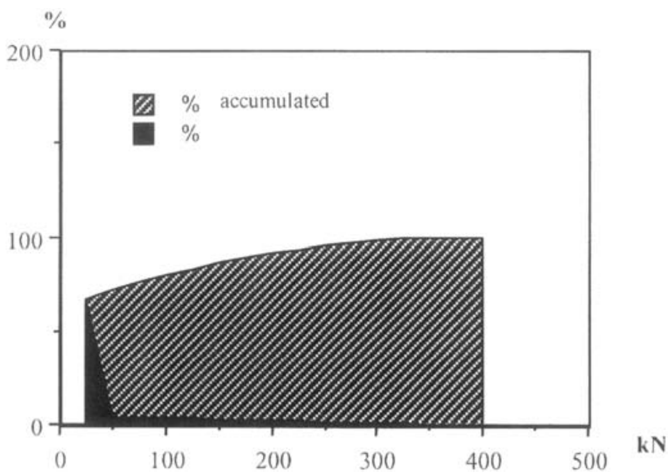


Figure 12-21. Proposed default distribution of traffic in terms of weight (used for both bridges of the example of application)

long-term partial costs described in Sections 12.5, 12.6, and 12.7 and on the traffic definition parameters registered or predicted. In the calculation of the values C_{F_1} and C_{F_2}, the structural failure costs associated with the new and existing bridges have been ignored, because they were considered approximately equal, even though the failure probability of a new bridge should, in principle, be lower than the corresponding value for a bridge with several years in service. As for the functional failure years, only those that occur in the period between the (hypothetical) collapse of the existing bridge and the opening to traffic of the bridge that is replacing the former bridge are quantified. Beyond that period, the functional failure costs of the new and the existing bridge are the same (since the bridges are supposed to be identical) and, therefore, are not relevant to this analysis.

Of all the functional failure costs during the period mentioned, only some are taken into account. For option 1 (immediate replacement of the existing bridge), only the cost incurred to detour traffic in terms of volume C_{FFV} are considered. There are no costs incurred as a result of delayed traffic C_{FFFD} because all the usual traffic has to be detoured during that period. The costs incurred due to the detouring of heavy traffic in terms of load C_{FFFL} are also left out because they are the same as for option 2 and, therefore, are not relevant to this analysis. For option 2 (replacement of the existing bridge only at the end of its predicted service life), only costs C_{FFFD} are considered. There are no costs C_{FFFV} since, under normal circumstances, no traffic is detoured. This situation may occur only if there is total traffic saturation during the 24 hours of an average working day. This simplification is a departure from reality made in the algorithm since there usually are traffic detours from any given bridge long before it reaches saturation levels. For the reasons pointed out previously, the costs C_{FFFL} were also omitted for option 2.

The system calculates and lists structural failure costs of previous years to the current year, in which the bridge obviously has not collapsed. On the other hand, in principle, the future structural failure costs will not occur either, because one of the greatest objectives of the management system is to prevent the structural collapse of a bridge during its predicted service life. The question of whether these costs should be part of the economic analysis must then be put forward. It is proposed that they should and that they should be accepted as a fair bridge insurance premium that the authorities manage and are prepared to pay to an insurance company (Figure 12-22) (de Brito and Branco 1994).

Loss of Life and Equipment Costs

The quantification of C_{FFFL} is extremely difficult and is susceptible to error in individual cases. An average is proposed taking into account documented cases of structural collapses of bridges (de Brito 1992).

$$C_{FFFL} = k_c\, n_v\, (n_{pv}\, c_p + c_v) \tag{12-55}$$

where

c_p = value of each human life [$]; this is a very controversial value, which is possible to obtain from an insurance companies' life insurance policy; it usually is calculated based on a certain number of yearly average salaries; the number of years varies greatly from country to country;

c_v = average value of each vehicle that crosses the bridge [$/vehicle]; this value can be obtained from national statistics about existing vehicles;

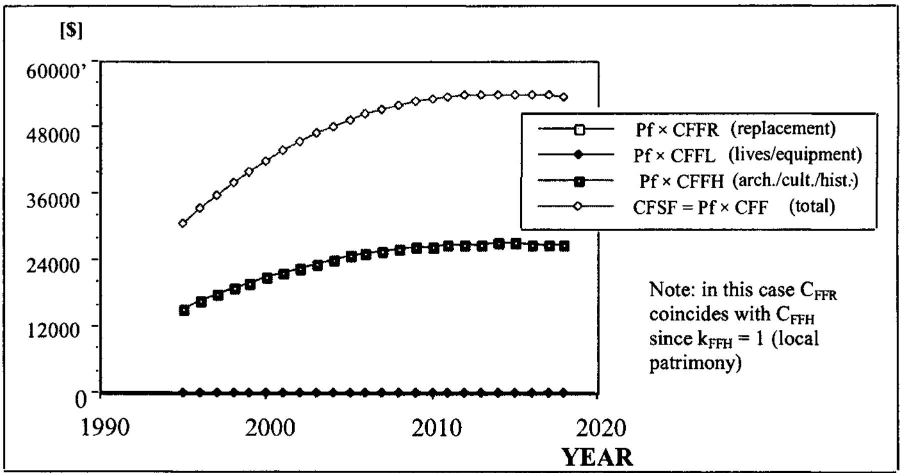


Figure 12-22. Example of prediction of the future structural failure costs of bridge 1 (see Section 12.3.4.) in present value prices based on the criteria proposed

n_{pv} = average number of persons per ordinary vehicle [vehicle^{-1}] (obtained from the same sources as c_v); $n_{pv} = 2.0$ is proposed;

n_v = average number of vehicles on the bridge during a certain period [vehicle];

k_c = corrective coefficient that gives the percentage of bridges that actually collapse when they reach a level of safety corresponding to structural collapse out of the total number of bridges discontinued specifically because of structural deficiencies; a figure of 20% is proposed.

$$n_v = \frac{TF_d\left(\dfrac{l_b}{v_b} + \Delta t\right)}{24} \tag{12-56}$$

where

TF_d = average daily volume of traffic on the bridge in both directions [vehicle/day]

l_b = total length of the bridge [km]

v_b = bridge design speed [km/h] (the design speed of the itinerary served by the bridge is correct for this effect)

Δt = annual average delay of the traffic crossing the bridge relative to the design speed [h]

In the algorithm COSTS, the calculation of the costs C_{FFFL} is made year by year based on the formulation explicit in Equations 12-55 and 12-56, for traffic definition parameters registered or predicted and on the internal calculation of Δt, which is explained in detail further in Figure 12-27 (de Brito and Branco 1994).

Architectural/Cultural/Historical Costs

When a bridge has a significant architectural, cultural, or historical value, the costs associated with its collapse C_{FF} must be artificially incremented through a factor k_{FFH} according to its classification (de Brito 1992).

$$C_{FFH} = k_{FFH}\, C_{FFR} \tag{12-57}$$

Although it is difficult to evaluate, a proposal of values for *kFFH* is presented (de Brito 1992):

$$\begin{aligned} K_{FFH} &= 0\text{—current nonclassified bridge} \\ &= 1\text{—local heritage} \\ &= 5\text{—national heritage} \\ &= 10\text{—national monument} \end{aligned} \tag{12-58}$$

These costs are a way to devalue solutions that involve the demolition and replacement of bridges of cultural interest, which should be rehabilitated even if at higher initial costs.

In the algorithm COSTS, the calculation of the costs C_{FFH} is made year by year through the direct application of Equation 12-57 after the coefficient k_{FFH} has been attributed and inserted into the bridge's specific data file by the user (Figure 12-22) (de Brito and Branco 1994).

12.8.2.2. Functional Failure Costs

Costs and Benefits

Before quantifying and predicting functional failure costs, it is necessary to clarify exactly what functional costs are and what benefits are, both of which are fundamentally related to functionality as discussed previously.

Functional costs can be generally defined as the value that would be attributed if the bridge functioned in order to provide a certain level of service (or to have a certain utility), which is considered the reference situation (generally defined during the design stage), as opposed to its real functionality. This value can be measured in several ways:

- delay in crossing the bridge multiplied by the volume of traffic delayed;
- volume of traffic that is stopped from crossing the bridge and is forced to detour (this may occur as a result of two different causes: the bridge does not have sufficient width for the potential volume of traffic; or the bridge does not have the load-bearing capacity necessary for a certain percentage of the heaviest vehicles).

On the contrary, the benefits that accrue are based on the value that would be attributed to the possibility of the bridge and its respective itinerary being able to provide better service (or be of greater utility) than the reference situation defined at the design stage. This concept can also be used to compare different solutions for repair, strengthening, deck widening, or replacement of a particular bridge (in this situation, one of the options is considered as the reference situation with nil benefits and all others are evaluated in relative terms). A benefit is equivalent to a decrease in functional failure costs or a negative functional failure cost. Their quantification is made the same way the costs are quantified.

When a bridge is conceived, the potential traffic and the service life loads considered have certain limits. What happens when these limits are increased (as a result of normal evolution or because the authorities want it to happen)? Even if the functionality of the bridge has not decreased in the meantime, the bridge capacity necessary to respond to the new demands may be insufficient. Is this lack of capacity a cost or a benefit not attained? It depends on what the reference situation is considered to be:

- if it is at the existing bridge's design stage, all solutions that increase its functionality generate benefits;
- if the reference is at the design stage of a new bridge (or of the existing bridge after rehabilitation) supposedly built to solve traffic problems, any solution that does not achieve the new service level represents a functional cost.

However, this inability to respond to demands unpredicted in the initial design should be considered only in situations in which an enhancement (strengthening, deck widening, or replacement) of the bridge is under consideration. In a comparison of traditional repair options, it is unnecessary to evaluate these costs (or benefits) because they are the same for all options.

In the algorithm COSTS, the reference situation is the nonexistence of the bridge and its itinerary, so all services it provides are considered benefits. This option results in an overevaluation of the benefits as compared with all the costs. However, the allocation of functional costs and benefits to each bridge is done in a different way. While the estimated functional costs are all attributed to the bridge under analysis (because it is considered as the only limiting factor to the normal flow of traffic), the estimated benefits only exist if the entire itinerary functions as predicted. Therefore only some of the benefits are attributed to the bridge under analysis, in accordance with its relative importance within the itinerary. As discussed earlier, the simplified criterion used was to multiply the total benefits from the itinerary by the coefficient $c_{b/r}$ (Equation 12-29), which reflects the relative weight of the initial investment in the bridge under analysis within the total initial investment in all the itinerary bridges.

General Definition

It is obvious that the functional failure costs and the benefits cannot be disassociated. Therefore, their quantification and prediction are discussed jointly. The functional failure costs can be divided into (de Brito and Branco 1994):

$$C_{FFF} = C_{FFFD} + C_{FFFV} + C_{FFFL} \tag{12-59}$$

The benefits are divided identically (de Brito and Branco 1994):

$$B = B_D + B_V + B_L \tag{12-60}$$

The traffic delayed costs C_{FFFD} (associated with the benefits B_D) result from the slowing down of traffic crossing the bridge, especially during rush hours.

The traffic flow detoured costs C_{FFFV} (associated with the benefits B_V) are those that arise when traffic is detoured from one bridge to adjoining bridges because of saturation of the first bridge in terms of traffic flow or as a result of its temporary closure.

The heavy traffic detoured costs C_{FFFL} (associated with the benefits B_L) are those that arise when a certain portion of exceptionally heavy traffic must be detoured from one bridge to adjoining bridges because of its insufficient structural capacity in terms of maximum live load.

Users Costs

The functional costs and benefits of an itinerary and the bridges included in it are but an increase or decrease of the users' costs of the circulating vehicles as compared to a basis situation. These expenses play a very important role in the choice of the itineraries and in the design of its layout because, if accounted for along the entire itinerary service life, may represent many times the construction cost [in a study relative to the OCDE countries (OCDE 1973), it is noted that these costs represent present value prices between 300% and 1000% of the construction costs over a 30-year period].

The following items are included in the users' costs (Sinha 1986):

1. increase/decrease of user travel time;
2. increase/decrease of motor vehicle running costs;
   a. fuel (the most important item, generally considered separately from the other motor vehicle running costs);
   b. engine oil;
   c. tires;
   d. maintenance and repair;
   e. depreciation (related to mileage);
      i. Items b to e are generally grouped in the so-called vehicle maintenance costs.
3. increase/decrease of traffic accident costs.

It can be said that the initial investment in the construction of an itinerary must be paid for gradually through a reduction of the following main costs: travel time (level of service), fuel, vehicle maintenance (energy saving), and traffic accidents (safety).

It is not easy to quantify these costs, because normative documents within this field already exist (AASHTO 1977). In the analysis made here, there was no intention to dwell excessively on these subjects, thereby permitting simplified hypotheses and formulas.

The travel time cost depends on whether the vehicles are trucks (depending on the volume and type of cargo) or passengers' cars. For the latter, the cost depends on a series of factors (Sinha 1986):

- number of persons in auto;
- purpose of trip;
- environment: time of day, traffic volume;
- use to which time saving is put;
- socioeconomic background of the driver;
- amount of time savings per trip.

The global cost is obtained by multiplying the value attributed to each hour saved by the volume of traffic and the average time saved. It has been proved (Sinha 1986) that motorists are less sensitive to an incremental unit of time savings when travel time savings are very small or very large.

The fuel and vehicle maintenance costs are affected by other factors (Sinha 1986):

- road type (its design, layout, and maintenance);
- vehicle type (weight, horsepower, efficiency);
- driver (accelerating and braking consume much more fuel than uniform speed driving);
- environment (weather, topography).

Of these, the most influential factors are fuel and vehicle price. The influence of trucks on the average vehicle maintenance costs is negligible when their percentage within the entire stream of vehicles is less than 5% (Sinha 1986).

The total cost is obtained by multiplying the value attributed to average fuel and maintenance costs per vehicle/km by the traffic volume and the length of the stretch of road. In special situations (bottlenecks, crossroads, etc.), an average value for fuel and maintenance costs per vehicle per hour at (almost) nil speed can also be used.

Traffic accident costs are greatly dependent on the type of accident (Sinha 1986):

- fatal;
- personal injury (in the United States, 30 to 35 less serious on average than fatal accidents (Lima 1985));
- property damage (in the United States, about three times less serious on average than the personal injury accidents (Lima 1985)).

To determine total accident costs for a particular stretch of road and to plan for its reduction, the following information is needed (Sinha 1986) and (Lima 1985):

- accident rate (in percentage of vehicles/km or in number/year);
- distribution by hours of the day;
- unit cost per accident by type;
- locations with more accidents;
- traffic volume;
- type of road;
- traffic design and traffic factors involved.

Among the latter, the following are paramount (Sinha 1986):

- horizontal curves and vertical grades >5% ⇒ about 20% more accidents than any other section;
- illumination;

- intersections;
- one-way streets ⇒ reduce vehicular accidents by 20% to 40% and yield a high reduction in pedestrian accidents;
- excess speed ⇒ fatalities increase two to three times and injuries increase one to three times;
- traffic volume ⇒ the accident rate is directly proportional to the log 10 of traffic volume.

In this book, the simplification of considering only a single type of accident has been adopted, including all accidents and with a weighed unit cost, which can be obtained from the statistics of traffic regulation authorities.

In the present value economic analysis, it is assumed that the benefits obtained through new investments in public works will not affect the benefits obtained from other utilities already in existence. This is not always the case in the area of transportation. For example, the construction of a new section of road may contribute to a decrease in the number of passengers on the railway stretch that, until then, was the best communication link for its users. These synergistic effects should be quantified to validate the economic analysis.

Conversely, the benefits determined with the algorithm COSTS are benefits to society in its broadest sense. Therefore, toll taxes have not been considered in the calculations. However, they must be taken into account in the studies, because they reflect the value that the users place or are willing to place on the new utilities available to them, as a trade-off of the ones that already exist and their cost.

Finally, other factors such as an increase in comfort, convenience, or safety as well as aesthetic/environmental aspects of the road layout have not been considered in the users' costs. This mostly is due to the fact that its quantification is extremely difficult to compute and is subject to debate.

Traffic Delayed

Traffic wanting to cross a particular bridge (especially near densely populated areas) is not constant throughout the day and also depends on the month of the year (or on the season). A curve showing typical daily traffic flow as a function of the time of day is presented in Figure 12-23 (de Brito 1992).

A bridge without functional costs must have a traffic capacity in terms of flow at the design speed (TF_0) higher than the maximum traffic flow wanting to cross the bridge at any given time (TF_{max}) (de Brito 1992):

$$TF_0 \geq TF_{max} \text{ [vehicle/h]} \Rightarrow C_{FFFD} = 0 \tag{12-61}$$

$$TF_0 = n_{vv}\, v_b \tag{12-62}$$

where

v_b = itinerary design speed or the maximum speed allowed on the bridge or the speed corresponding to the itinerary design service level [km/h]

n_{vv} = maximum number of vehicles in movement per km in all the road lanes at its design speed according to the valid safety rules [vehicle/km]; this value depends

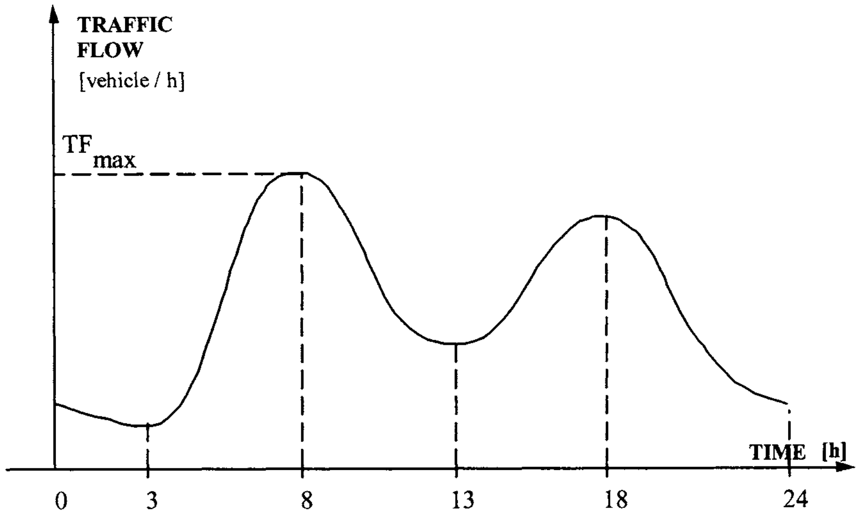


Figure 12-23. Schematic representation of the traffic flow as a function of the hour of the day

on the traffic speed v_b (higher speed ⇒ greater distance between vehicles ⇒ fewer vehicles per km) and on the total number of lanes (n_l) and of separated lanes (n_f) of the road and on the bridge (theoretically the same); it also depends on other factors such as the width of the lanes and road margins, the transversal clearance, and the vertical grade that, for simplicity's sake, are ignored in this model.

In the algorithm COSTS, the following formula has been used (de Brito 1992):

$$n_{vv} = \frac{6480000\, n_1}{v_b^2} \frac{n_f}{2} \tag{12-63}$$

deduced based on the following hypotheses:

- spacing time: 4.5 s for v_b = 90 km/h and varying in proportion to the square of the design speed; the spacing time tends toward the reaction time when traffic volume tends toward bridge capacity (which occurs for speeds of around 50 km/h);
- speed at contact of two vehicles moving in the same direction at the design speed, based on the supposition that the vehicle in front brakes suddenly: 0 m/s.

As an alternative, the values listed in the North American Highway Capacity Manual (TRB 1985) corresponding to different levels of service on highways and roads of a total width of 12.5 m could be used. In Figure 12-24 (de Brito 1992), a diagram is presented in which a comparison is made between Equation 12-63 and these values.

In many bridges, the situation $TF_0 < TF_{\max}$ occurs immediately after construction because it is not economically feasible to design a bridge for its maximum daily traffic flow and have it practically empty most of the time. Even when that does not occur right after construction, it is possible that it may occur later on since the volume of traffic that crosses the bridge tends to increase with passing years. If the rush hour is isolated in a

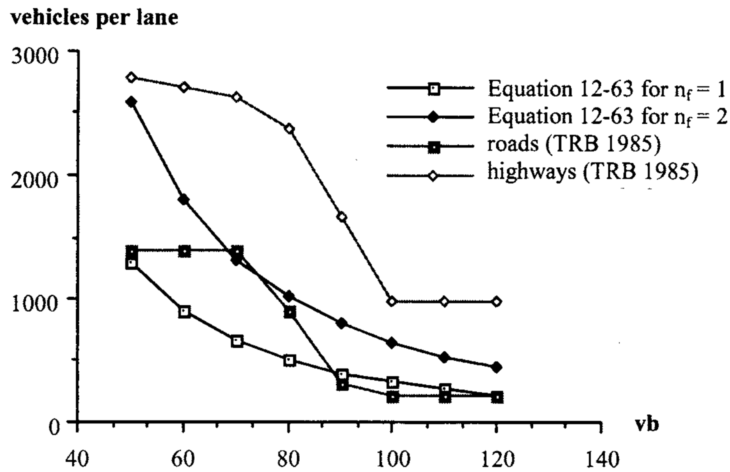


Figure 12-24. Number of vehicles per km of road lane as a function of the itinerary design speed (comparison between Equation 12-63 and Reference TRB 1985)

time-dependent diagram, a representation similar to that shown in Figure 12-25 (de Brito 1992) is obtained.

As the traffic flow $tf(t)$ crossing the bridge at the design speed reaches the maximum possible value, the speed at which traffic moves starts to decrease. The volume of traffic crossing the bridge that corresponds to a flux higher than TF_0, T_D, undergoes a translation in time Δt due to the speed reduction (obviously, the delay undergone by each individual vehicle is not the same because every vehicle that tries to cross the bridge during the period Δt_0 undergoes some delay, much smaller than Δt).

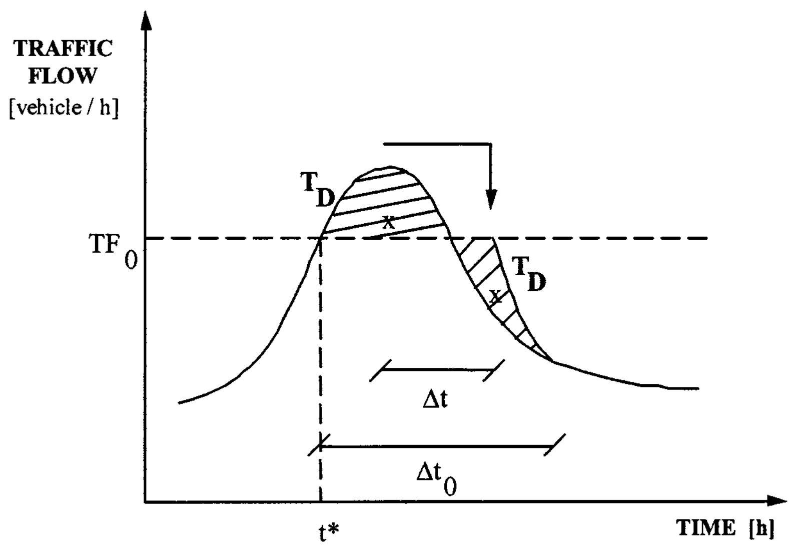


Figure 12-25. Typical consequence of a rush hour in which the maximum capacity of the bridge is exceeded (translation of traffic in the shades area T_D of an average time period Δt)

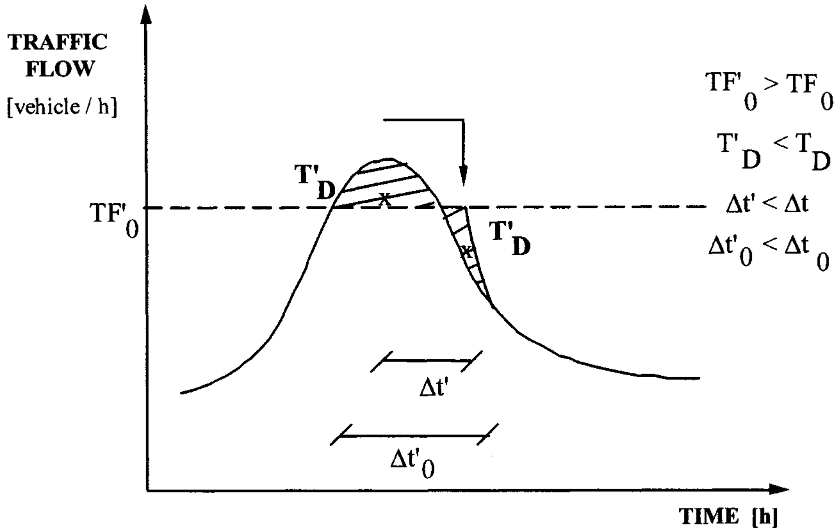


Figure 12-26. After the bridge capacity has been increased from TF_0 to TF'_0, the volume of traffic delayed and the average delay time decreases

Theoretically, the functional costs are proportional to (de Brito 1992):

$$\int_{t^*}^{t^*+\Delta t_0} t\, f(t)\, \Delta t(t)\, \mathrm{d}t \tag{12-64}$$

If Δt is the "distance" (measured in time) between the gravity center of the shaded area T_D (Figure 12-25) (de Brito 1992) if the bridge capacity were boundless and its real gravity center taking into account the limitation TF_0, it can be said that the functional costs in terms of travel time are proportional to $T_D\, \Delta t$ [vehicle hour].

Let us suppose that two solutions for a certain bridge are under analysis:

- option 1—do nothing;
- option 1—widen the deck to increase the maximum traffic flow capable of crossing the bridge at the design speed to TF'_0 (Figure 12-26) (de Brito 1992).

The benefits in terms of travel time obtained from option 2 are directly proportional to $(T_D\, \Delta t - T'_D\, \Delta t')$.

This philosophy can also be applied to a situation in which it is necessary to block one of several lanes of a bridge (e.g., if the bituminous pavement is in poor condition and must be replaced). In that case, additional traffic delays can be quantified as a functional cost of the repair. When comparing repair options, these costs will penalize those that involve the blocking of lanes and may result in the choice of a solution with higher initial costs.

It may happen that a bridge has functional costs related to traffic that moves in one direction (during a certain time of day) and no problems (during the same period) for traffic moving in the opposite direction. The situation may be inverted as the day passes. In any event, it is easy to quantify $T_D\, \Delta t$ if there is information about the traffic flow that reaches the bridge in each direction.

Of the users' costs mentioned previously, the delay of traffic when crossing the bridge does not affect the average itinerary accident rate, so the respective costs are not considered. Costs that result in increases in travel time have already been discussed. Finally, it is interesting to quantify the increase in fuel and vehicle maintenance costs caused by traffic decreasing speed (possibly even coming to a stop) when crossing the bridge. The following simplified model has been adopted: it is assumed that the traffic rolls at the design speed until it reaches the bridge; there it stops for a period equal to the average annual delay (Equation 12-56), a period necessarily different (i.e., smaller) than the actual delay of the traffic affected; once on the bridge, the traffic resumes moving at the design speed. In the simplified model, the bridge functions as a crossroad.

Traffic costs for crossroads can be divided into the following categories (Sinha 1981):

1. stoppage time (or nearly zero speed) of the vehicles (in \$/vehicle hour);
2. operation cost of the stopped vehicles (in \$/vehicle hour);
3. deceleration and acceleration times of vehicles (in \$/vehicle hour);
4. operating cost of decelerating and accelerating the vehicles (in \$/stops).

Costs 1 and 3 were taken into account in the analysis related to increases in travel time when crossing the bridge. For the costs 2 and 4, it is proposed to use a single cost (in \$/vehicle hour) that is equivalent to the costs of fuel and vehicle maintenance when it is (theoretically) stopped.

The result of this analysis is that the functional costs (and benefits) in terms of traffic delayed C_{FFFD} can be divided into travel time costs C_{FFFDT} and vehicle running (fuel/maintenance) costs C_{FFFDF} (de Brito and Branco 1998b):

$$C_{FFFD} = C_{FFFDT} + C_{FFFDF} \tag{12-65}$$

The following formulation is proposed to quantify the functional failure costs (and benefits) associated with traffic delayed in terms of travel time C_{FFFDT} (de Brito and Branco 1998b):

$$C_{FFFDT}/\text{year} = T_D \, \Delta t \, k_{wp} n_{wpv} \frac{\text{GNP}}{N_{wp} H_w} \tag{12-66}$$

where

T_D = total annual volume of traffic expected to cross the bridge at a rate higher than its capacity at the road design speed, thereby incurring delays [vehicle] (obtained from a traffic census)

Δt = average delay of traffic T_D [h] (substantially higher than the average delay of all the traffic affected)

GNP = U.S. Gross National Product [\$/year] (national statistics)

N_{wp} = total number of working people in the country (national statistics)

H_w = average number of working hours per year [h/year] (average number of working days in a year × 7.5 hours)

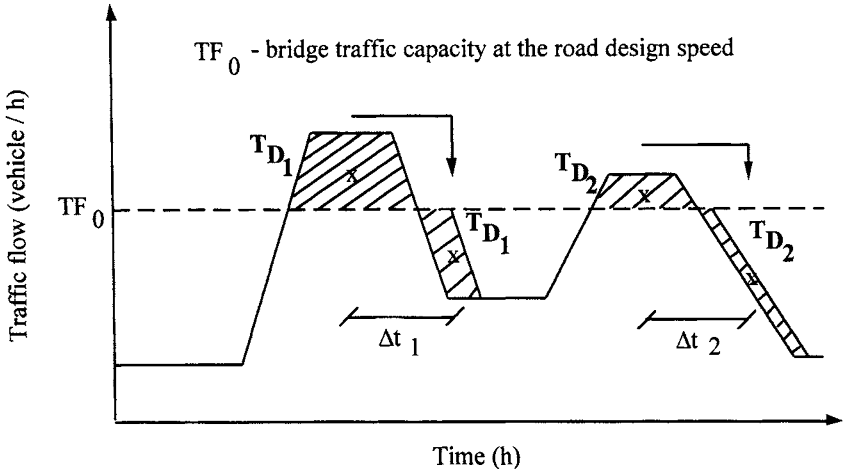


Figure 12-27. Simplification proposed for the daily traffic flow distribution

n_{wpv} = average number of working people per vehicle [vehicle^{-1}] (motorized vehicle statistics); it is proposed that $n_{wpv} = 1.5$

k_{wp} = percentage of working people in the vehicles during actual working hours; it is proposed that $k_{wp} = 80\%$

To determine the annual functional failure costs due to traffic delays, it is necessary to be familiar with the evolution of traffic throughout the year. It is reasonable to assume that bridge capacity is not reached on non-working days (Saturdays, Sundays, and public holidays) and, depending on the location of the bridge, during certain months of the year (generally the summer months). For the remaining days of the year, it is useful to know whether there are significant differences in traffic flow at other times during the year. If that is not the case, a general function can be devised with average values for each working day of the type represented in Figure 12-23 (de Brito 1992). This can be further simplified (using only linear variations) as suggested in Figure 12-27 (which assumes two rush-hour periods for a normal working day) (de Brito and Branco 1998b).

The total costs C_{FFFDT_i} for day i are obtained by (de Brito 1992):

$$C_{FFFDT_i} = (T_{D_1}\, \Delta t_1 + T_{D_2}\, \Delta t_2)\, k_{wp} n_{wpv} \frac{\text{GNP}}{N_{wp} H_w} \tag{12-67}$$

The costs C_{FFFDT} for the entire year are easily determined by (de Brito 1992):

$$C_{FFFDT}/\text{year} = D_w\, C_{FFFDT_i} \tag{12-68}$$

where

Dw = average number of working days in a year [day/year] (52 weeks of 5 working days minus the days during holidays and the days corresponding to public holidays; a rate of 10% is considered to take into account illnesses and other reasons for missing work); for the sake of simplicity, it is assumed that there are no traffic delays at the bridge on non-working days.

The following formulation is proposed to quantify the functional failure costs (and benefits) associated with traffic delays in terms of fuel and vehicle maintenance:

$$C_{FFFDF}/\text{year} = T\, c_{fmvs}\, \Delta t \qquad (12\text{-}69)$$

where

T = total annual volume of traffic on the bridge [vehicle/year]

c_{fmvs} = average fuel and stopped vehicles (or at a speed of almost zero but with the motor running in both cases) maintenance cost [$/(vehicle h)]

Δt = average delay of bridge traffic relative to the design speed [h]

This analysis does not make sense if the bridge is not the critical factor for traffic fluidity. If enhancement of the bridge has no practical consequences on the itinerary traffic flow because of other bottlenecks, the removal of the bottlenecks must be considered before or simultaneously with the enhancement of the bridge.

In the algorithm COSTS, the calculation of C_{FFFD} is made year by year based on Equations 12-61 to 12-69, on traffic prediction parameters registered or predicted and on the internal determination of $T_D\, \Delta t$ in the subroutines TRAFFIC or TRAFFIC2. They use the information (obtained from a file or provided interactively) relative to distribution of the daily traffic flow on an average working day. The distribution is defined by linear sections (Figure 12-27) (de Brito and Branco 1998b), and the flux at each moment is represented by a percentage of the average instantaneous flux, if it were constant throughout the year. When $TF_{max} \leq TF_0$, there are no costs (Figure 12-28, **a.**) (de Brito and Branco 1998a). When that is not the case, the simplified hypothesis, which states that the variation in traffic flux after translation follows an evolution similar to the initial variation represented in Figure 12-28, **b.**) (de Brito and Branco 1998a), has been adopted.

When the volume T_{D_1} fills the extra capacity of the bridge between the two rush-hour periods, a single rush-hour period remains that extends from early morning to late afternoon and the average delay increases substantially (Figure 12-28, **c.**) (de Brito and Branco 1998a). When the bridge reaches saturation throughout the day, surplus traffic volume must be detoured and is associated with the costs C_{FFFV} (Figure 12-28, **d.**) (de Brito and Branco 1998a). This hypothesis is a simplification of reality, since the traffic starts being detoured from the bridge long before it reaches saturation.

In the algorithm, it is assumed that there are no benefits related to the delay in the traffic crossing the bridge B_D caused by its construction, unless it is possible to quantify the delays for the adjoining bridges that cease or decrease when the bridge under analysis is built. This quantification is extremely difficult and, at the present stage of the algorithm, has not been attempted.

In practice, it is well known that traffic behavior during rush hours varies according to the time of day. On a typical working day and for a bridge with access to a major city, there are usually two rush-hour periods: one in early morning and another in late afternoon. In the first rush hour, the majority of people are trying to reach their place of business by a certain hour. To do so, they are willing to anticipate the beginning of their journey while taking into account the usual delays. Only a fraction of the traffic chooses to arrive at its destination later than the rush-hour period. In the afternoon, most working people leave their jobs at approximately the same time and the tendency is to try to return home as soon as possible. The result is that the greater part of the traffic is delayed. Only a few people

a.

b.

c.

d.

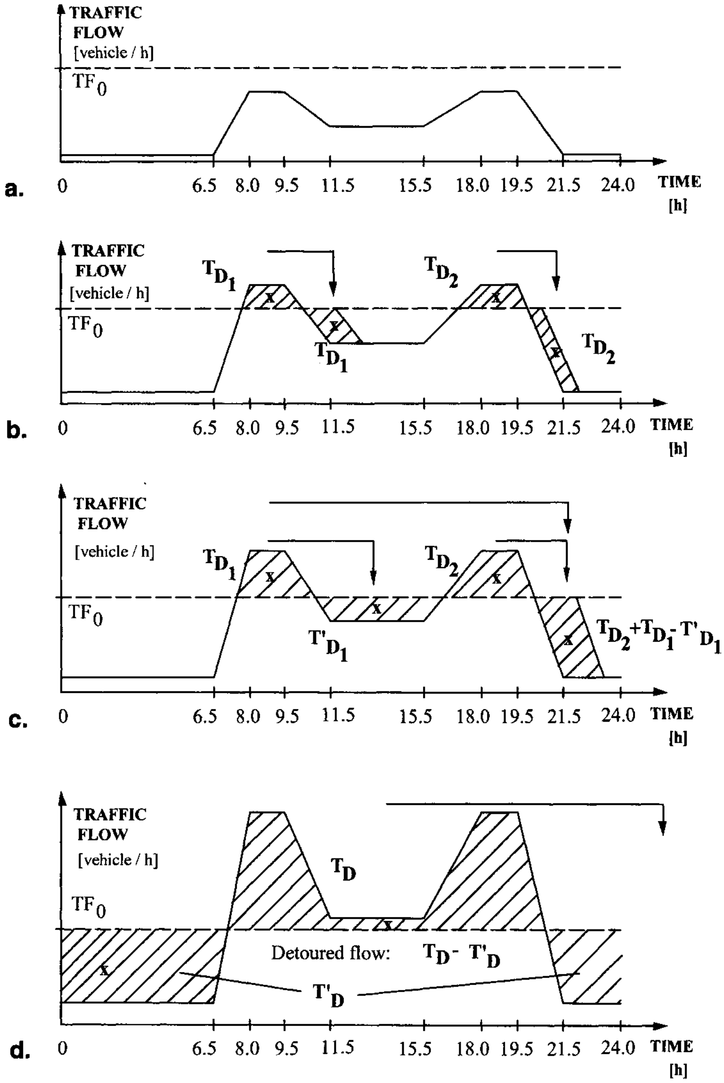
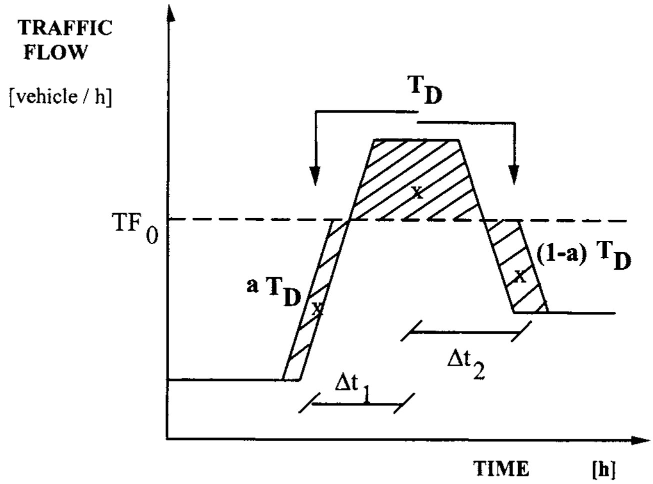


Figure 12-28. Typical evolution of traffic flow in a bridge of constant traffic capacity in which the flow keeps increasing: **a.**, there are no functional failure costs; **b.**, functional failure costs come up (two rush hours); **c.**, saturation point between the two rush hours ⇒ a single rush hour that takes almost all day; **d.**, saturation during the whole day ⇒ traffic

have a sufficiently flexible work timetable to allow them to anticipate the beginning of the return trip.

As discussed earlier, the subroutine TRAFFIC considers any circumstance in which the volume of traffic that surpasses the bridge's maximum capacity is transferred to sometime after the rush-hour period. In order to adapt the algorithm COSTS closer to reality, a subroutine TRAFFIC2, functioning as an alternative to the first subroutine, has been devised. In this new subroutine, the user provides the percentage of traffic affected by the rush hour that anticipates the beginning of the journey to take into account the delays. The



a—percentage of the rush hour excess traffic volume that anticipates its journey to minimize delays

TF_0—traffic capacity of the bridge at the design speed

Figure 12-29. Model of evolution of traffic flow in rush hours easily adaptable to the preferences of the users

remaining percentage is transferred to a period after the rush hour. The rush-hour traffic transfer in terms of the percentage noted above is represented schematically in Figure 12-29. Contrary to subroutine TRAFFIC, subroutine TRAFFIC2 has the disadvantage of being more limited in terms of representing the daily traffic distribution: it only allows sections of varying flow between sections of constant flux and the increasing flux sections must alternate with the decreasing flux sections (as shown, for example, in Figure 12-27) (de Brito and Branco 1998b).

Similar to Equation 12-67, the costs C_{FFFDT_i} for the rush hour period i due to traffic delays are given by (Figure 12-29) (de Brito 1992):

$$C_{FFFDT_i} = (aT_{D_1}\,\Delta t_1 + (1-a)\,T_D\Delta t_2)\,k_{wp}n_{wpv}\,\frac{\mathrm{GNP}}{\mathrm{N}_{wp}H_w} \qquad (12\text{-}70)$$

In Figure 12-30 (de Brito 1992), an example in which the results obtained using subroutines TRAFFIC and TRAFFIC2 for the same bridge and daily traffic distribution (Figure 12-20) (de Brito and Branco 1998a) is presented. In subroutine TRAFFIC2, it was assumed that the greater part (80%) of the traffic affected by the early morning rush hour is anticipated at the beginning of the journey. In the late afternoon rush hour, the same percentage of excess traffic is unable to anticipate its journey back home. In this example, the predicted annual traffic grows linearly with time. It has been verified that during the initial years traffic does not suffer any delays even during the rush hours, because bridge capacity at the design speed has not been reached. After that period, the average delay time grows nonlinearly with the global traffic. The figure also shows that, as expected, the possibility of

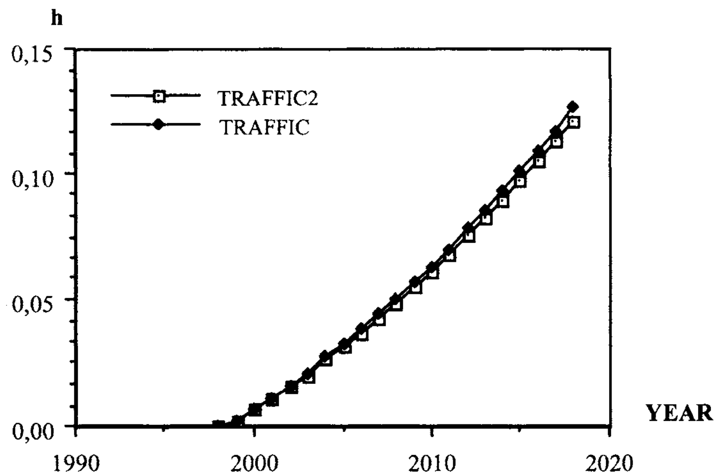


Figure 12-30. Comparative analysis of the results obtained using subroutines TRAFFIC and TRAFFIC2 to determine the global traffic average delay when crossing bridge 2 (see Section 12.3.4.) on an average weekday

adapting the traffic to the rush hours (anticipating or delaying the beginning of the journey—TRAFFIC2) contributes to a decrease in the average delay.

Volume of Traffic Detoured

In some cases, the functional failure of the bridge results from its inability to accommodate all the traffic trying to cross the bridge, thus forcing part of the traffic to be detoured to nearby bridges. Another possibility is that the bridge will have to be closed to traffic in one or both directions (or that it can no longer withstand the loads because of a structural failure). In that case, all traffic that normally would use the bridge must be detoured.

To quantify all traffic detoured T_V [vehicle] during the period in which the bridge is under repair, Equation 12-71 is proposed (de Brito 1992).

$$T_V = \sum_{i+1}^{n_{RV}} \int_0^{24\,\mathrm{h}} tf_d(t)\,\mathrm{d}t = \sum_{i+1}^{n_{RV}} T_{V_i} \tag{12-71}$$

where

n_{RV} = number of days of bridge repair (estimate)

$tf_d(t)$ = detoured traffic flow as a function of the time of day (and possibly of the day of the week or of the year) and of the number of directions closed to traffic [vehicle/h] (statistics of the bridge or of the itinerary)

T_{V_i} = total volume of traffic detoured in day i [vehicle/day]

The average detour length l_D [km], which is theoretically constant in time, is given by Equation 12-72 (de Brito 1992).

$$l_D = \frac{\sum_{j=1}^{n_{PD}} T_j \, l_{D_j}}{T_V} \tag{12-72}$$

where

n_{PD} = number of possible detours (known for each bridge)

T_j = total volume of traffic detoured to route j during the whole period [vehicle]

l_{D_j} = detour j length [km] (also known for each detour)

This concept can also be used in the comparison of different repair solutions. The options that consider blocking traffic in one or in both directions will be penalized, and it can easily happen that they are dropped in favor of "more expensive" solutions that maintain traffic fluidity during the entire repair process.

The detour of traffic that normally would use a certain bridge has consequences for all users' costs, as discussed previously. Therefore, the functional costs (and benefits) resulting from traffic detoured in terms of volume C_{FFFV} (traffic flow detoured costs) can be divided into travel time costs C_{FFFVT}, (vehicle) fuel costs C_{FFFVF}, other vehicle running costs C_{FFFVM} (also known as vehicle maintenance costs), and traffic accident costs C_{FFFVA} (de Brito and Branco 1998b):

$$C_{FFFV} = C_{FFFVT} + C_{FFFVF} + C_{FFFVM} + C_{FFFVA} \tag{12-73}$$

The following equation is proposed to quantify the functional failure costs (and benefits) associated with the volume of traffic detoured in terms of travel time C_{FFFVT} (de Brito and Branco 1998b):

$$C_{FFFVT}/\text{year} = \frac{T_V/\text{year } l_D}{v_b} k_{wp} n_{wpv} \frac{\text{GNP}}{N_{wp} H_w} \tag{12-74}$$

where

T_V/year = total annual detoured traffic in the bridge [vehicle/year]

v_b = design road speed in the detours [km/h]

k_{wp}, n_{wpv}, GNP, N_{wp}, H_w, and l_D have the same meaning as in Equations 12-66 and 12-72

The functional failure costs (and benefits) associated with the volume of traffic detoured in terms of fuel C_{FFFVF} are given by the following equation (de Brito and Branco 1998b):

$$C_{FFFVT}/\text{year} = T_V/\text{year} \left[(c_{fv_D} - c_{fv}) L_r + c_{fv_D} l_D \right. \tag{12-75}$$

where

c_{fv_D} = average fuel cost in the detours [\$/vehicle km] (official statistics)

c_{fv} = average fuel cost in the bridge itinerary [\$/vehicle km] (estimate)

L_r = total length of the itinerary [km]

T_V/year and l_D have the same meaning as in Equation 12-74

In Equations 12-75 to 12-77, it is assumed that the traffic that must be detoured from a bridge within the itinerary supposes that the entire itinerary is shut down and that no vehicle will circulate within it. However, the average detour length l_D must be added to the total length of the itinerary L_r to obtain the new route length used by the detoured traffic. This is a simplification but it can be further elaborated based on the analysis of case studies. If the concept of itinerary, as defined in Section 12.3.1, is restricted to the stretch that connects two locations and is not intersected by any other, then the formulation adopted is correct.

The functional failure costs (and benefits) associated with the volume of traffic detoured in terms of vehicle maintenance costs C_{FFFVM} are given by the following equation (de Brito and Branco 1998b):

$$C_{FFFVM}/\text{year} = T_V/\text{year}\,[(c_{mv_D} - c_{mv})\,L_r + c_{mv_D} l_D \tag{12-76}$$

where

c_{mv_D} = average vehicle maintenance cost in the detours [$/vehicle km] (official statistics)

c_{mv} = average vehicle maintenance cost in the bridge itinerary [$/vehicle km] (estimate)

T_V/year, L_r, and l_D have the same meaning as in Equation 12-75.

The functional failure costs (and benefits) associated with the traffic detoured in terms of traffic accidents CFFFVA are given by the following Equation 12-77 (de Brito and Branco 1998b):

$$C_{FFFVA}/\text{year} = T_V/\text{year}\left[(r_{a_D} - r_a)\,L_r + r_{a_D} l_D\right] c_a \tag{12-77}$$

where

r_{a_D} = average accident rate in the detours [$/vehicle km] (official statistics); these rates are reliable only if obtained in situ

r_a = average accident rate on the bridge itinerary [$/vehicle km] (estimate)

c_a = average cost per accident (weighed average of accidents that involve fatalities, personal injury, or just property damage; police or insurance company official statistics)

T_V/year, L_r, and l_D have the same meaning as in Equation 12-75

A traffic detour can also result from delays of normal traffic in alternate routes. The global traffic (i.e., the traffic that uses the bridge under analysis plus the traffic that uses the detours under normal circumstances) must be maintained. This means that the volume of traffic using the detours will increase. If delays in critical sections (such as other bridges) result, these delays must be quantified using the proposed equations (12-61 to 12-70) and the respective costs must be added to C_{FFFD}.

In the algorithm COSTS, the calculation of these costs is made year by year based on Equations 12-71 to 12-77, on the traffic description parameters registered and predicted,

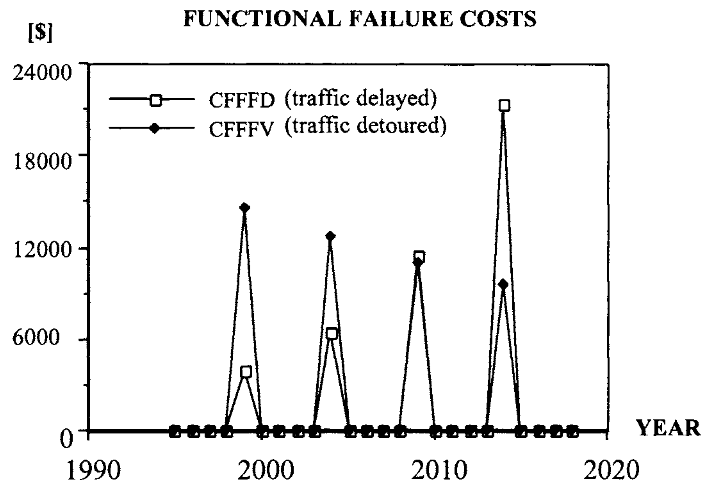


Figure 12-31. Prediction of functional failure costs associated with repair of the bituminous pavement of bridge 1 (see Section 12.3.4.) at present prices

and on the definition of future volumes of traffic detoured. For that, the user has four options (de Brito 1992):

- to use linear regression based on detoured traffic volume registered for the latest years (up to a maximum of 10 years);
- to use his own linear variation of future detoured traffic volume with time;
- to provide a prediction of repair actions in discrete years (the years in which repairs are expected to occur and cause either partial or total traffic blockage are provided along with estimates of time of repair ERT and the average number of lanes simultaneously affected n_{lr});
- to let the system calculate all future costs for the option of replacing the bituminous wearing surface of the bridge deck every 5 years throughout its service life (Figure 12-31) (de Brito 1992).

For the first two options, detoured traffic volumes T_V are calculated or provided directly. For the third option and for the years in which a repair action is anticipated, the following equation applies (de Brito 1992):

$$T_V = \alpha \, TF_d \frac{\text{ERT} \quad n_{1r}}{n_1} \qquad (12\text{-}78)$$

where

α = corrective coefficient defined later in this chapter

T_V = total volume of traffic detoured during the repair [vehicle]

TF_d = average daily traffic volume on the bridge in both directions [vehicle/day]

ERT = estimate of the time of repair [day]

n_{lr} = average number of lanes simultaneously affected during the repair

n_l = total number of lanes on the bridge

Initially, it was thought that the traffic that would under normal circumstances use the lanes blocked by repair works would be totally detoured as long as the repair was being done. As an alternative, it could be considered that the traffic at the bridge would not be affected, contrary to the bridge capacity for letting traffic cross it at the design speed, which would be reduced from the normal TF_0 to TF_0' (Figure 12-25) (de Brito 1992).

$$TF_0' = TF_0 \frac{(n_1 - n_{1r})}{n_1} \tag{12-79}$$

This naturally would lead to delays for the vehicles passing over the bridge and would result in an increase in the costs C_{FFFD}. A third alternative would be a combination of the two previous options, with the choice depending on the alternate routes available. This was the option chosen because it is the one that gives the user more freedom. In this context, a corrective coefficient α has been defined as the ratio between the traffic that is actually expected to be detoured from the bridge and the fraction of the total traffic at the bridge proportional to the average number of lanes simultaneously affected by the repair. In this situation, the additional costs C_{FFFD} (covering the period during which the repair is made) are calculated for a capacity TF_0' and traffic volume equal to the normal volume minus the fraction T_V.

In the fourth option, Equation 12-78 is also used but with $n_{lr} = 1$ and (de Brito 1992):

$$\text{ERT} = \frac{1000\, l_b}{50} n_1 \tag{12-80}$$

where

l_b = total length of the bridge [km]

n_l = total number of lanes on the bridge

which corresponds to a repair pace of 50 m of lane per day.

An example of the use of this option of calculating the costs resulting from periodic repairs of the bridge's wearing surface is represented graphically in Figure 12-31 (de Brito 1992). The coefficient α from Equation 12-78 is considered to be equal to 20%. In this case, the percentage of increased annual traffic is lower than the interest rate, and therefore the costs C_{FFFV} at present value prices decrease with time. On the contrary, because of the nonlinear relationship between the average delay time in crossing the bridge and the total volume of traffic, even at present value prices, the costs C_{FFFD} increase with time.

In the algorithm, the benefits B_V are calculated supposing that the reference situation is the nonexistence of the bridge and that therefore all the traffic that, now or in the future, uses the bridge benefits from the fact that traffic does not have to be detoured to nearby bridges. However, only that fraction of the benefits that correspond to its relative importance in the itinerary is quantified (Equation 12-81) (de Brito 1992).

$$B_V/\text{year} = c_{b/r}\, T\left[\frac{l_D}{v_b} k_{wp} n_{wpv} \frac{\text{GNP}}{N_{wp} H_w} + (c_{fvD} + c_{mvD} + r_{aD} c_a - c_{fv} - c_{mv} - r_a c_a) L_r + (c_{fvD} + c_{mvD} + r_{aD} c_a)\, l_D\right] \tag{12-81}$$

where

$c_{b/r}$ = percentage of the total itinerary benefits and costs assigned to the bridge under analysis (Equation 12-29)

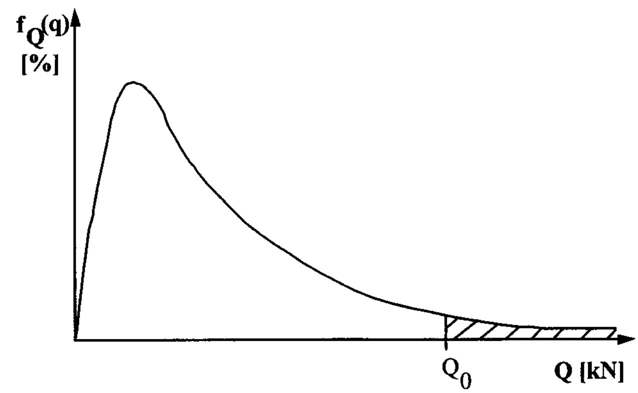
T = total annual traffic that uses the bridge [vehicle/year]

l_D, v_b, n_{wpv}, GNP, N_{wp}, H_w, L_r, c_{fvD}, c_{mvD}, r_{aD}, c_a, c_{fv}, c_{mv}, and r_a have the same meaning as in Equations 12-74 to 12-77

Heavy Traffic Detoured

In some cases, functional failure of the bridge results from its lack of structural capacity to withstand live loads above a certain limit, thus forcing certain heavy vehicles to be detoured to adjoining bridges.

The maximum load that a bridge can withstand is a function of the number of axles of the vehicle, the maximum axle load, the spacing between axles, the longitudinal and transversal location of the point loads, the ultimate limit state under consideration, the transverse cross section of the bridge under analysis, the vehicle speed, and so forth. Therefore, it is not an easy value to quantify. To facilitate the reasoning, let us assume that for a certain bridge it is possible to quantify the maximum admissible load through a single parameter Q. It is therefore possible to identify the fraction of traffic that potentially can use the bridge as a function of this parameter. A typical distribution of the traffic as a function of Q is represented in Figure 12-32 (de Brito 1992).



$f_Q(q)$ —density function of the load Q of the potential traffic at the bridge

Q_0 —maximum load admissible at the bridge [kN]

Figure 12-32. Potential traffic using the bridge that has to be detoured because of its structural incapacity (cross-lined area)

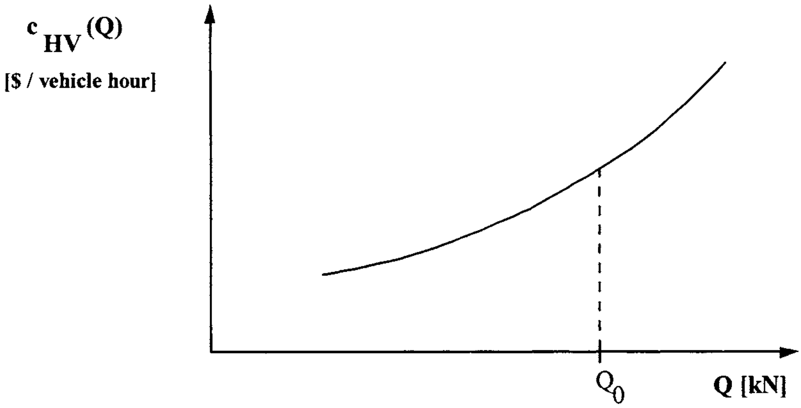


Figure 12-33. Total running costs of heavy trucks as a function of the load parameter Q

If T is the total potential traffic using the bridge over the course of one year [vehicle/year], then the potential traffic detoured due to structural incapacity T_Q [vehicle/year] is given by (de Brito 1992):

$$T_Q = T \int_{Q_0}^{\infty} f_Q(q)\, dq \tag{12-82}$$

To estimate the cost of detouring this volume of traffic, it is important to quantify its (operation) running costs. The vehicles affected are heavy cargo trucks whose running costs c_{HV} increase with their net weight (Figure 12-33) (de Brito 1992).

These costs must include:

- equipment costs (amortizations, depreciation, expendables—fuel, engine oil, tires, maintenance, repair, insurance);
- workmanship costs (according to the type of personnel needed: drivers, mechanics, assistants, etc.);
- others (licenses, parking, tolls).

These costs may be obtained from the accounting records of land transportation operators.

This concept is particularly useful when one of the options for action is the strengthening of the deck or the infrastructure with the objective of increasing the maximum allowable load that can pass over the bridge. The benefits (or the reduction of failure costs) of this option can be quantified and compared with the extra costs of strengthening.

Of the running costs mentioned previously, the cost that has not been considered is the potential increase in the average rate of heavy vehicles accidents in the detours from the itinerary that includes the bridge under analysis. The respective costs can be calculated through a formulation identical to Equation 12-77, in which the traffic detoured as a whole is replaced by the fraction of heavy traffic detoured. The vehicles' running costs (travel time, fuel, and maintenance) have already been considered globally in the costs c_{HV} referred to previously.

The following equation is proposed to quantify the functional failure costs (and benefits) associated with traffic detoured in terms of load C_{FFFL} (heavy traffic detour costs) (de Brito and Branco 1998b):

$$C_{FFFL}/\text{year} = \frac{T\, l_{DH}}{v_b} \int_{Q_0}^{\infty} f_Q(q) c_{HV}(q)\, dq + T\left[\left(r_{a_D} - r_a\right) L_r + r_{a_D} l_{DH}\right] c_a \int_{Q_0}^{\infty} f_Q(q)\, dq \qquad (12\text{-}83)$$

where

l_{DH} = average detour length for the heavy trucks [km] (probably longer than the equivalent length for lighter traffic even though obtained in a similar way; Equation 12-72)

v_b = design speed in the detours or maximum allowed speed for this type of vehicles [km/h] (the lesser of these values)

$f_Q(q)$ = density function of the load Q of the potential traffic at the bridge

$c_{HV}(q)$ = total annual running costs for heavy trucks as a function of its load [$/year]

Q_0 = maximum load admissible at the bridge [kN]

T, L_r, c_a, r_a, and r_{a_D} have the same meaning as in Equation 12-81

The detour of heavier traffic may also result in delays of the normal traffic in the alternate routes. If there are delays at the most critical sections of the detours, their costs must be added to c_{FFFD} after being quantified using the equations proposed (12-61 to 12-70). However, very heavy traffic flow is insignificant when compared with global traffic, other than for exceptional cases. Nevertheless, the influence of heavy traffic on the average speed of circulation of light vehicles is pronounced.

It is interesting to examine here the hypothesis for limiting the load of motorized vehicles passing over the bridge by posting. The costs and benefits of this type of action may be quantified through Equations 12-59 to 12-83. Let us assume that the two following solutions for a bridge are under analysis:

- option 1—to do nothing but to limit the maximum admissible load at the bridge to a value $Q_0' < Q_0$;
- option 2—to strengthen/repair the bridge to maintain the design maximum admissible load of Q_0.

The relative functional costs of option 1 would be (de Brito 1992):

$$\frac{T\, l_{DH}}{v_b} \int_{Q_0'}^{Q_0} f_Q(q)\, c_{HV}(Q)\, dq \qquad (12\text{-}84)$$

Option 2 would obviously have additional costs related to the strengthening. In a simplified way, it can be said that if the costs relative to functional failure of option 1 are lower than the repair costs of option 2, option 1 is the better solution and vice versa.

In the algorithm COSTS, the calculation of costs C_{FFFL} is made year by year based on the formulation explicit in Equations 12-82 and 12-83, traffic definition parameters registered and predicted, the definition of volume of heavy traffic detoured and the respective total running costs. These values are obtained from the distribution by weight of the traffic using the bridge (contained in the specific bridge file or provided by the user), an up-to-date costs list (contained in the general file related to failure costs), and the description of the adjoining road network in terms of its capacity to accommodate heavy traffic detoured (also contained in the bridge file).

In the algorithm, there is no option to consider benefits related to the traffic detour in terms of load B_L caused by its construction, unless it is possible to quantify the volume of heavy traffic detoured from other bridges nearby, whose average detour length decreases after the bridge under analysis is built.

The values obtained in a present value analysis for the functional failure costs and the benefits split into the various items already described are presented in Figure 12-34 (de Brito and Branco 1994).

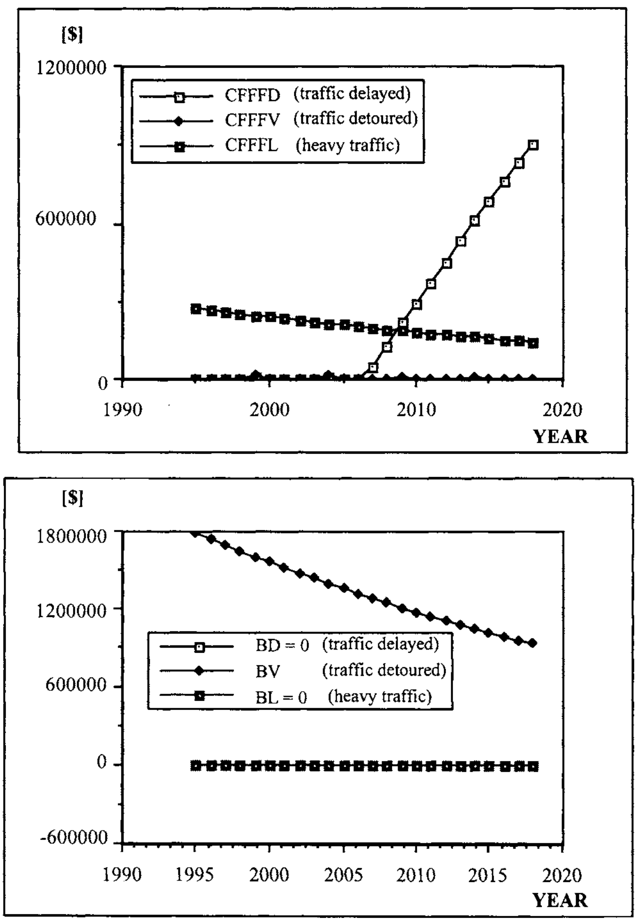


Figure 12-34. Example of prediction of the future functional failure costs and of the benefits of bridge 1 (see Section 12.3.4.) and the corresponding itinerary in present prices based on the criteria presented

12.8.2.3. Environmental/Social Impact Costs

Included in the environmental impact/social costs are items as varied as (Sinha 1986):

- changes in the underground water;
- changes in water courses;
- air quality;
- changes in the local fauna/flora;
- increase in the noise levels;
- properties limits;
- pedestrian accidents;
- inhabitants displacement;
- archeological/historical sites;
- changes in the visual aspect of the landscape;
- installation of polluting industries;
- inhabitants health;
- rubbish deposits;
- cutting down a certain number of trees;
- demolition of houses with regional architectonic value;
- others.

From an analysis of these items, it can be concluded immediately that the quantification of most of them is very difficult, particularly at the preliminary studies stage. However, different society groups of people attribute totally different values to these items, which makes their quantification polemic. In optimization of layout studies, there are three ways to look at environmental costs (RRG 1973):

1. The designer ignores them and produces as a first step an "aggressive" solution that he will later adjust based on his subjective judgment. One of the advantages of this method is the fact that the designer knows the cost of the relative environmental enhancement by directly comparing the costs of the solutions.
2. "Directive principles" are incorporated into the programs so that the designer can reach an optimal solution for a set of conditions that are adapted to environmental demands. For example, it is possible to introduce limitations to the passage of the communication link in certain areas or to artificially inflate the grounds.
3. The environmental costs can be progressively quantified. There are acceptable levels for traffic-induced noise as a function of its volume, the distance from houses, the type of road, and the nature of the ground (AASHTO 1977). Similar levels can be created for each of the different items.

Social costs can also be quantified by comparing the costs of different designs for the itinerary layout, and then opting for a certain number of limiting criteria, which must be included in the definitive design (RRG 1973).

At the present stage of the algorithm COSTS, it has been decided not to include the environmental impact/social costs for three reasons:

1. their quantification and prediction is far from unanimous and even less accurate;
2. they do not usually affect the economic analysis of the bridge itself over time because they depend almost exclusively on the itinerary layout;
3. it is not the authors' intention to dwell on the vast range of problems concerning the optimization of itineraries layout.

Other Benefits

The enhancement or construction of a bridge can produce additional benefits not yet considered, which are difficult to quantify but should be considered in the economic analysis:

- possible increased tourism in certain regions;
- increase of value of real estate in the proximities of the bridge;
- development of industry in the region;
- others.

It may also contribute to a decrease in the functional failure costs of nearby bridges, in the sense that part of the normal traffic starts to use the bridge under analysis. This decrease can be quantified in terms of traffic that no longer must be detoured from its optimum route.

At this stage, it is interesting to compare the scales of several of the costs and benefits involved in an economic analysis of a bridge. To do so, one may refer to the results from the example presented in (de Brito 1992), which corresponds to a long-term analysis (ca. 30 years), in which all costs, even the initial costs, are predicted. Figure 12-35 (de Brito and Branco 1998b) clearly shows that inspection, maintenance, repair, and structural failure costs are much less important than the initial and functional failure costs and benefits (also connected with functional aspects). Even the initial costs themselves become diluted in a long-term analysis because they occur only in a very limited number of years.

The relative evolution of functional failure costs and benefits is explained in the following way: the benefits are proportional to the total annual traffic, with an annual percentage of growth that is lower than the interest rate, thus causing a decrease over time in present value prices. Functional failure costs are ruled by costs resulting from traffic delayed when crossing the bridge. After an initial period during which bridge capacity is not reached at the design speed of the itinerary (even during rush hours), costs decrease over time in a fashion similar to the benefits, delays start to register at the bridge. Bridge costs increase much more rapidly than the total traffic volume; therefore, as the bridge nears its saturation point, the functional failure costs approach the benefits, possibly even surpassing them.

This type of analysis leads to recognition of the functional service life concept, which covers the period from the end of construction, during which the bridge maintains a

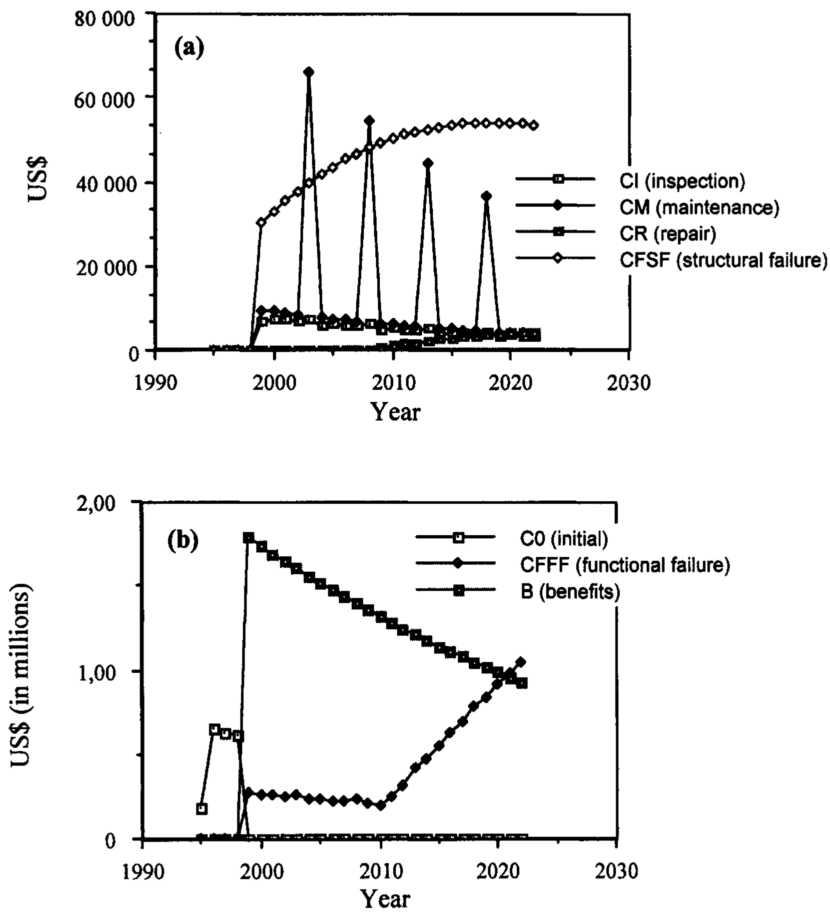


Figure 12-35. Comparative analysis of the evolution in time of the various costs and benefits of bridge 1 (see Section 12.3.4.) in present prices

predetermined level of functionality. In the situation described previously, the year in which, according to the economic analysis, the total costs exceed the benefits should coincide with one of the following events:

- replacement of the existing bridge with a new bridge with greater traffic capacity;
- opening to traffic of a new bridge that functions in parallel with the first bridge, thus easing the latter's traffic volume;
- widening the deck of the existing bridge.

In countries with great economic resources, the breakeven analysis permits determination of the year of construction of the bridge in which the accumulated costs during the bridge's predicted service life is equal to the accumulated benefits in the same period. However, in a country with limited resources, in which several enterprises are competing for existing funds, this analysis is not sufficient and what is important to maximize is the Net

Present Value (difference between the accumulated benefits for the service life of the bridge in present value prices and the accumulated costs for the same period), which is done through a net present value analysis (Bhandari and Sinha 1979). If the cost of opportunity of the capital is at least as high as the interest rate and if the annual benefits increase monotonically with time (untrue in the analysis represented in Figure 12-35) (de Brito and Branco 1998b), the Net Present Value is maximized if the bridge is discontinued in the first year during which the benefits are equal to the cost of opportunity of the capital invested (Bhandari and Sinha 1979).

As the benefits are directly dependent on the predicted traffic volume, it becomes obvious that it is only economically feasible to build a bridge if there is a minimum increase in that volume. The benefits obtained from its passage must compensate for the cost of the capital invested and the current operation costs. However, in situations close to saturation, the cost of the delays produced at the bridge because of a bottleneck effect may even surpass the benefits obtained by the users. Therefore, there is also a maximum volume of traffic above which the bridge must be either replaced or widened. The bridge must be designed so that its service life occurs between these two limits of traffic volume (Figure 12-36) (de Brito 1992). Since, in general, the predicted volume of traffic increases with time, to the traffic volume limits there corresponds a period of time that ideally marks the service life targeted for the bridge.

The more current concept of service life is the concept associated only with structural aspects that will be described in greater detail in Chapter 13. What is important to keep in mind is that, in the face of growing economic pressures, the number of bridges will increase and will be designed in terms of predicted functional service life, which the structural design must make feasible.

In Figure 12-37 (de Brito 1992), the various costs and benefits are presented in terms of percentages of the total, both at present value and at real prices. The relative importance of initial costs decreases considerably in a long-term real prices analysis because of inflation (in the example presented, considered constant and equal to 10%).

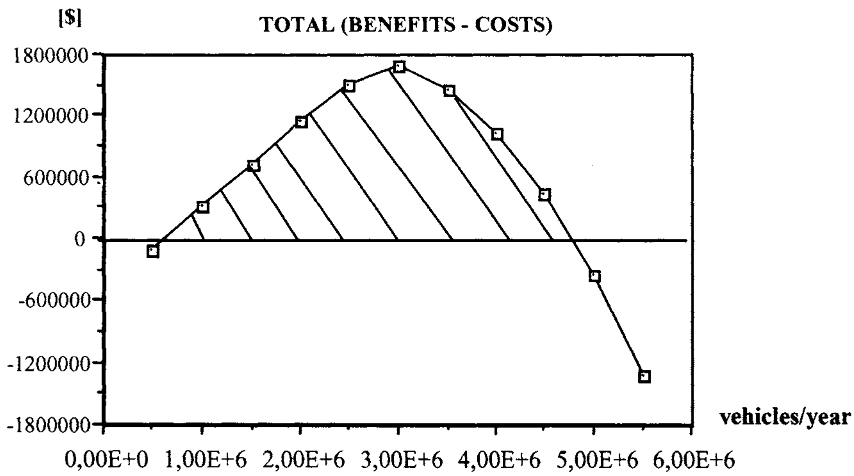


Figure 12-36. Evolution of the total annual ratio (benefits : costs) of a bridge in present prices with the predicted traffic volume

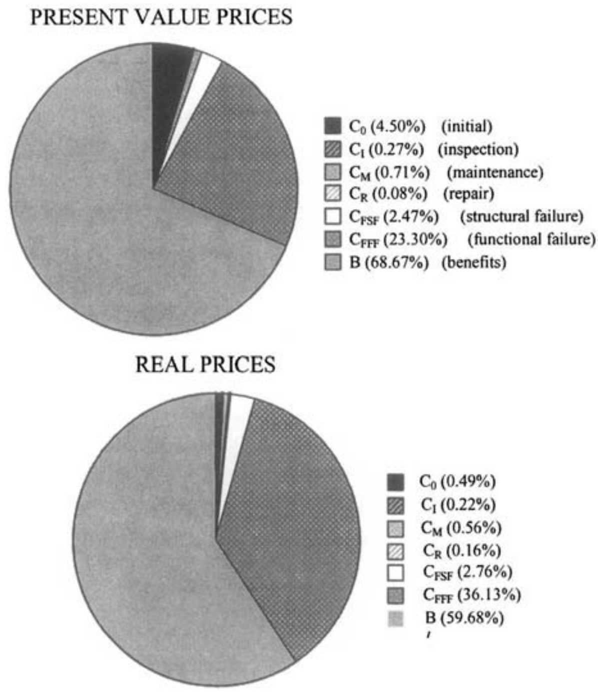


Figure 12-37. Total accumulated costs and benefits, percentages of bridge 1 (see Section 12.3.4.) in present value and real prices

12.9. General Architecture of the COSTS Algorithm

Figure 12-38 (de Brito and Branco 1998b) shows, through the use of a flowchart, the general architecture of the computer algorithm named COSTS, for quantification and prediction of costs in road concrete (reinforced and prestressed) bridges.

In the present chapter, the procedure adopted in the algorithm to determine the various costs is described, right after the presentation of calculation formulas. Next, a short description of the main computer subroutines within COSTS is given, with their main features and the type of data on which they rely (de Brito and Branco 1998b).

12.9.1. Short Description of the Subroutines

12.9.1.1. PRINCIPAL

This subroutine has the following main features:

- initialization of all parameters;
- calls the subroutine GENERAL;
- calls the subroutines CALC0, CALCI, CALCR, CALCF, and RESULT as many times as the number of bridges under analysis;
- calculates costs at present value prices from their real prices counterparts.

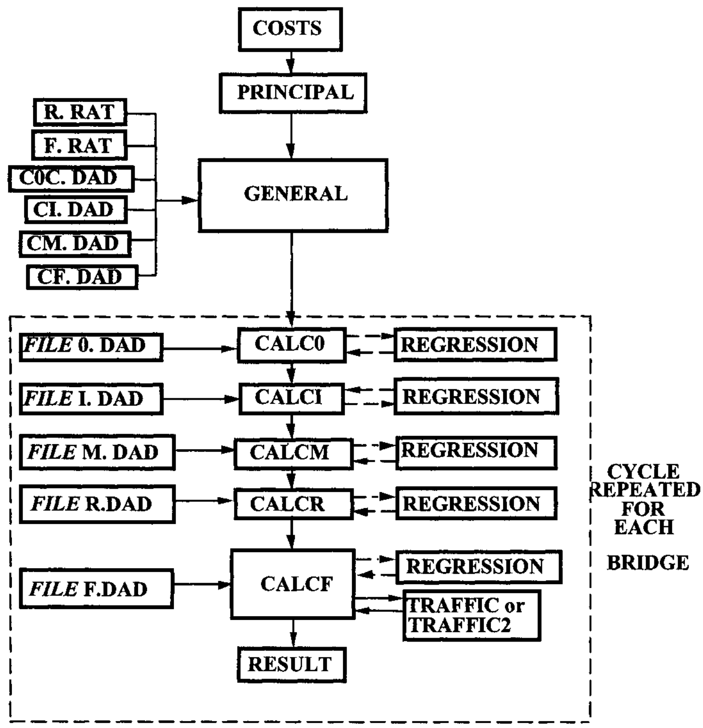


Figure 12-38. Computer algorithm COSTS

12.9.1.2. GENERAL

This subroutine has the following main features:

- reads general data and options of utilization of the program fed in by the user;
- reads the files R.RAT and F.RAT (discount and inflation rates already known) and estimates their future values;
- reads the files of general costs COC.DAD, CI.DAD, CM.DAD, and CF.DAD.

12.9.1.3. CALC0

In this subroutine, the file *FILE*0.DAD is read through subroutine LEITU0. This file contains the entire initial costs specific to each bridge under analysis as well as those of its road area of influence (one file for each bridge in which *FILE* is the code name). The initial costs for the current year as well as future costs are then estimated, when necessary.

12.9.1.4. CALCI

In this subroutine, the file *FILE*I.DAD is read through subroutine LEITUI. This file contains the entire inspection costs specific to each bridge under analysis as well as those of its road area of influence (one file for each bridge). The future inspection costs are then estimated. Depending on the user's choice, subroutine REGRESSION may be used.

12.9.1.5. CALCM

In this subroutine, the file *FILE*M.DAD is read through subroutine LEITUM. This file contains the entire maintenance costs specific to each bridge under analysis as well as those of its road area of influence (one file for each bridge). The future maintenance costs are then estimated. Depending on the user's choice, subroutine REGRESSION may be used.

12.9.1.6. CALCR

In this subroutine, the file *FILE*R.DAD is read through subroutine LEITUR. This file contains the entire inspection costs specific to each bridge under analysis (one file for each bridge). The future repair costs are then estimated. Depending on the user's choice, subroutine REGRESSION may be used.

12.9.1.7. CALCF

In this subroutine, the file *FILE*F.DAD is read through subroutine LEITUF. This file contains the entire failure costs and benefits specific to each bridge under analysis as well as those of its road area of influence (one file for each bridge). The future failure costs and benefits are then estimated. Depending on the user's choice, subroutine REGRESSION may be used. To estimate the functional failure costs and benefits, subroutine TRAFFIC is used or, alternatively, subroutine TRAFFIC2 is used.

12.9.1.8. RESULT

This subroutine's main function is to write the results of the economic analysis. These can be presented with three alternative levels of detail:

- the condensed global annual costs for each bridge;
- the condensed partial annual costs for each bridge and the results of the sensitivity analysis only for global costs, in addition to the information mentioned in the previous level;
- the partial annual costs for each bridge divided into their separate items and results of the sensitivity analysis for partial costs, plus the information mentioned in the previous level.

12.9.1.9. REGRESSION

This subroutine performs a linear regression for any parameter given, based on all the known results prior to the current year. Even if more results are available, only those that are related to the previous 10 years are considered relevant and taken into account.

12.9.1.10. TRAFFIC/TRAFFIC2

The function of these subroutines is the calculation of traffic delays when crossing the bridge during an average working day and their respective average delay time. To do that, the daily traffic flow distribution and the maximum traffic capacity of each bridge at the road design speed must be known. While in the subroutine TRAFFIC it is assumed that excess traffic during rush hours is necessarily moved to sometime after those periods, subroutine TRAFFIC2 takes into account that part of the traffic may choose to anticipate its journey (Figure 12-29) (de Brito and Branco 1994). However, the latter is more limiting in terms of daily traffic distribution.

12.9.2. Data Files

The data files used by the algorithm COSTS are fundamentally of two types:

- of a general nature, with a national scale range of application or, at least, valid for all the bridges under analysis;
- of a particular nature for each bridge, which implies the existence of one file with a certain type of information for each bridge.

The following files are included in the first type of data file:

- R.RAT—discount rates;
- F.RAT—inflation rates;
- C0C.DAD—construction costs;
- CI.DAD—inspection costs;
- CM.DAD—maintenance costs;
- CF.DAD—failure costs and benefits.

The following files are included in the second type of data file:

- *FILE*0.DAD—initial costs;
- *FILE*I.DAD—inspection costs;
- *FILE*M.DAD—maintenance costs;
- *FILER*.DAD—repair costs;
- *FILE*F.DAD—failure costs and benefits.

Further details about the organization of the algorithm COSTS and its routines and data files can be obtained from the technical annexes of (de Brito 1992).

12.9.3. Sensitivity Analysis

For the economic analysis referred to previously, it is necessary to estimate a set of parameters for which it is impossible to guarantee great precision. It is therefore of interest to know to which of the final results are most sensitive, so that extra effort can be put into calibrating them. To do so, certain parameters are made to vary by a fixed percentage up and down relative to the estimated value and the results thus obtained are compared. In order to avoid the influence of interaction, the sensitivity analysis must be made one parameter at a time.

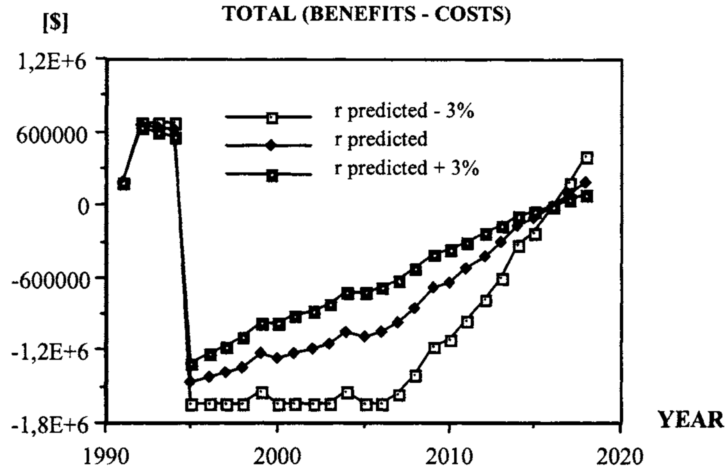


Figure 12-39. Sensitivity of the global cost function to the future interest rates (±3%) in present value prices analysis of bridge 1 (see Section 12.3.4.)

The algorithm allows evaluation of the resultant sensitivity with nine different parameters (de Brito and Branco 1994):

- future values of interest rates (this parameters affects only the present value prices economic analyses; Figure 12-39);
- future values of inflation rates (this parameter affects only the real prices economic analyses—Figure 12-40 (de Brito 1992)—because the construction and general inflation rates are considered equal);
- future values of initial costs (as shown in Figure 12-37 (de Brito 1992), the sensitivity of the global cost function to this parameter fades away with the increase of

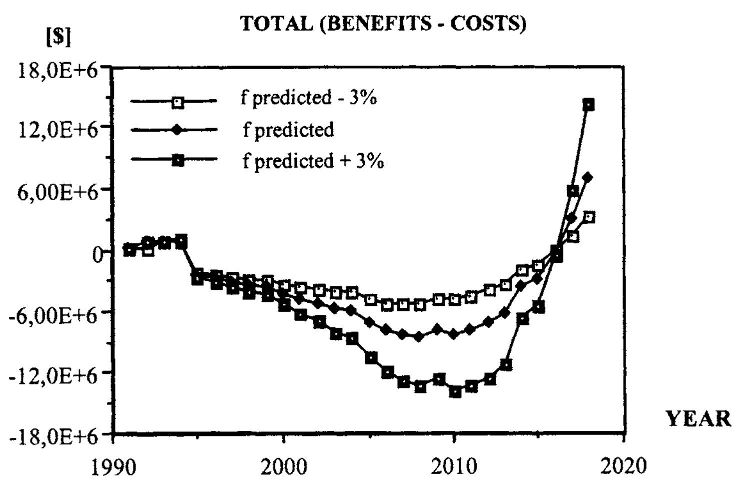


Figure 12-40. Sensitivity of the global cost function to the future inflation rates (±3%) in a real prices analysis of bridge 1 (see Section 12.3.4.)

the period of time covered by the economic analysis, particularly in real prices analyses);

- future values of inspection costs (this parameter has a negligible influence on the global cost function; see Figure 12-37) (de Brito 1992);
- future values of maintenance costs (what has been stated about the previous parameter applies);
- future values of repair costs (what has been stated about the previous parameter applies);
- future values of the structural collapse probability (only the structural failure costs are significantly sensitive to this parameter; its influence in the global cost function is minor; see Figure 12-37) (de Brito 1992);
- future values of traffic volume (this parameter significantly affects the functional failure costs and the benefits that, as shown before, represent the greater part of the costs involved in a bridge's long-term economic analyses; see Figure 12-41) (de Brito 1992);
- future values of detoured traffic volume (this parameter significantly affects only the costs due to detoured traffic in terms of volume; they generally represent by themselves only a relatively insignificant percentage of the functional failure costs; see Figure 12-34) (de Brito and Branco 1994).

In Figure 12-39 (de Brito 1992), the main results of a present value prices sensitivity analysis to future interest rates are presented. The predicted value used is 4% considered constant over all the analysis period, and the variation imposed on the interest rate of every future year is ±3%. As expected, a higher interest rate corresponds to a decrease in the present value costs, especially those that occur later.

In Figure 12-40 (de Brito 1992), the main results of a real prices sensitivity analysis to the future inflation rates are presented. The predicted value used is 10%, considered con-

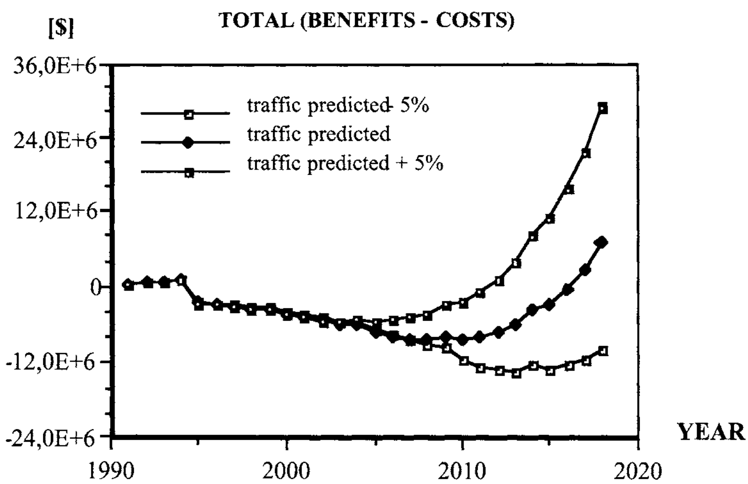


Figure 12-41. Sensitivity of the global cost function to the future traffic volumes (±5%) in a present value prices analysis of bridge 1 (see Section 12.3.4.)

stant over all the analysis period, and the variation imposed on the inflation rate for every future year is ±3%. An increase in the inflation rate causes an increase in the real prices, and the more so the later they are paid.

In Figure 12-41 (de Brito 1992), the main results of a present value prices sensitivity analysis to the future traffic volumes are presented. A prediction of the evolution of total traffic volume throughout the analysis period is made, and the variations imposed on traffic volume for every future year is ±5%. It can be verified that costs due to delays in traffic crossing the bridge are very sensitive to the predicted values of traffic volume, especially when they approach the bridge's saturation point.

An example of the application of the algorithm to two bridges (real prices economic analysis) and an analysis of the sensitivity to the future inflation rates is presented in (de Brito 1992). The complete listing in FORTRAN of the algorithm COSTS is also presented in this reference.

CHAPTER 13

Repair Strategies

13.1. Introduction

In Chapter 11, the division of the decision system into maintenance/small repair and rehabilitation/replacement (Figure 8-3) was described. The criteria used to distinguish which work performed within the scope of bridge management should be included in each of these sub-systems were also described. Specifically, it was stated that, whenever there is a need to perform a structural assessment (as defined in Chapter 10), the elimination of the defects that originated the assessment should be included in the rehabilitation/replacement sub-system (described in this chapter as repair sub-system).

However, the criteria that allow a decision as to whether there is a need to perform a structural assessment and when to do so has not yet been discussed. This decision making immediately follows a periodic inspection (current or detailed) and constitutes the first part of the decision sub-system.

If the decision is to go forward with the structural assessment, the inspection must be planned to provide answers to existing uncertainties about the bridge's structural capacity, as well as elements that enable an evaluation of the necessity of a structural repair (repair techniques recommended and estimates of the quantities of work required). The possibility of enhancement of the bridge's structural capacity (strengthening of the superstructure or the infrastructure), its functionality (widening of the deck or increase of the maximum live loads allowed), and even its replacement by another bridge that better answers questions about the necessities of the system, must be studied and comparatively analyzed. This analysis is fundamentally economic, even though most of its data are characterized as technical. Its content constitutes the second part of this decision sub-system.

13.2. Submodule of Inspection Strategy

The bridge inspection module presented considers the existence of two types of inspection: periodic and nonperiodic. The latter cannot be planned in the long term since the need only arises when, during a periodic inspection, an important structural or functional defect is detected. Therefore, it is necessary to define the criteria that allow a decision to be made concerning this subject. The submodule must be used immediately after every periodic inspection (from hereon referred to as inspection strategy submodule). Independent of the results of the inspection, it is necessary to know whether the bridge inspires enough confidence to do without any structural repair work until the next periodic inspection. Even though a structural assessment is expensive, its cost is substantially lower than the repair itself and, whenever a structural assessment is needed, it is tacitly accepted that it will be performed independent of any budget limitation.

Two possible decision systems that concern the necessity and planning of structural assessments are presented next. The first system is based on the defect rating, as described in Chapter 11 and, even though less "precise", it is much easier to apply. The second system is based on an analysis of the bridge's structural reliability, but it requires a vast range of information that is not always available. It also forces the use of a computer algorithm to estimate the evolution with time of the bridge's reliability and its updating based on the results of the periodic inspections.

13.2.1. Decision by Rating of the Structural Defects

The defect rating system presented in Chapter 11 is based on three fundamental parameters: (1) urgency of rehabilitation, (2) importance to the structure's stability, and (3) the volume of traffic affected by the defect.

The third parameter has no contribution to the decision to perform the structural assessment since it is related only to functional aspects of the bridge. Furthermore, only the defects of type A according to the importance to the structure's stability criterion (eminently structural defects associated with main structural elements—bridge deck, beams, columns, abutments, and foundations) may lead to a structural assessment being proposed.

The decision to call for a structural assessment depends on the rating according to the urgency of rehabilitation criterion. Type 3 defects dispense with any additional investigation until the next periodic inspection, at which time they will be reanalyzed and eventually rated again based on the inspection form. Defects of types 0 and 1 demand a structural assessment as soon as the planning and logistic difficulties allow it. As for type 2 defects, a structural assessment must be planned and carried out with enough time beforehand to perform the repair work concerning them before the next periodic inspection (15 months hence). Figure 13-1 (de Brito 1992) schematically represents the decision flowchart presented here.

13.2.2. Decisions Based on Structural Reliability Analysis

13.2.2.1. Decision Criteria

A structural reliability analysis of the bridge should not be exceedingly complex to meet the objective that it is supposed to help attain: to evaluate the necessity of a structural assessment after a periodic inspection. It is based on quantification and prediction of the reliability index β, a function of the bridge's structural collapse probability P_f (through the normal distribution function), itself a function of time (Thoft-Christensen 1992):

$$\beta(t) = -\phi^{-1}\,(P_f(t)) \tag{13-1}$$

This type of analysis has been developed within the scope of the Brite/Euram EC project designated as BREU-0186-C, "Assessment of Performance and Optimal Strategies for Inspection and Maintenance of Concrete Structures Using Reliability Based Expert Systems." The analysis has been applied only to degradation mechanisms associated with the corrosion of steel bars (initialized by carbonation or chloride penetration).

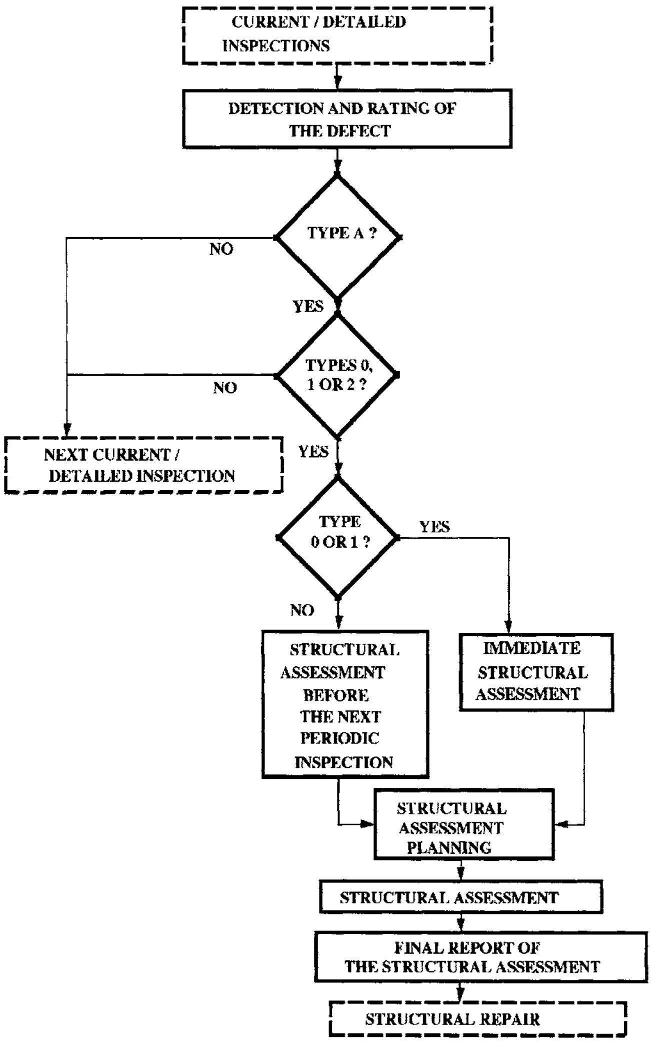


Figure 13-1. Inspection strategy submodule based on the rating of the structural defects (see Figure 11-3)

Two computer-based expert knowledge systems/modules (de Brito et al. 1997) were developed: the interactive system Bridge-1, mentioned in Chapter 10, to be used as an inspection aid at the bridge site; the Bridge-2 module for decision making, containing the inspection strategy, maintenance, and repair submodules, to be used at the management authorities headquarters after collecting the necessary data during inspection.

The global functionality of the Bridge-1 and Bridge-2 decision systems, which are illustrated in Figure 13-2 (de Brito et al. 1997), is used to perform the inspection, maintenance, and repair activities. After a periodic inspection i (current or detailed, C/D), performed at time t_i, with the help of Bridge-1 (B1), the inspector classifies the defects leading to maintenance activities. The maintenance Bridge-2 B2(M) submodule is then used to rate the defects and maintenance work (M) is implemented. After every inspection, a reliability

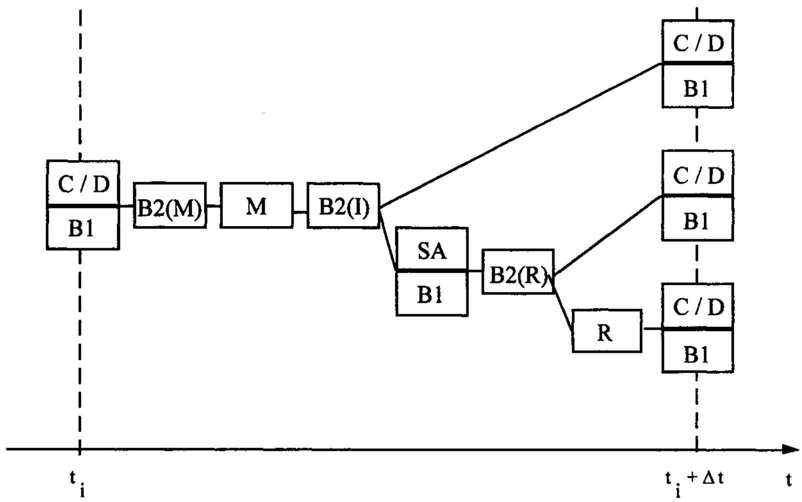


Figure 13-2. Reliability-based expert system general procedure

analysis B2(I) (inspection strategy submodule) of the bridge is performed. Based on the predicted evolution of safety, calculated by updating the reliability index, this module indicates whether a structural assessment should be performed before the next scheduled periodic inspection. The assessment will only occur if an important defect is detected on the bridge. After the structural assessment (SA) is completed, the repair submodule B2(R) is used to perform an economic and reliability analysis and to decide the type of repair work and the schedule for performing the work (R). At time $t_i + \Delta t$, the process starts again (Δt is the interval between periodic inspections).

In this expert system, reliability estimates are obtained by using first order reliability methods (FORM) (Thoft-Christensen and Baker 1982). For each failure mode or limit state (ultimate or serviceability), it is assumed that it is possible to formulate a failure function $g(x, t) = g(x_1, x_2, \ldots, x_n, t)$, where

t = time

x = a realization of the basic stochastic variables X modeling the uncertain quantities (e.g. load and yield strength)

The failure function is defined so that a positive value of g indicates a safe set of basic variables, whereas a negative value of g indicates a failure set of basic variables. For time-invariant reliability problems, the probability of failure P_f of one failure mode in the time interval $[0, t]$ is then estimated by:

$$P_f(t) = P(g(X, t) \leq 0) \approx \Phi(-\beta(t)) \tag{13-2}$$

where

Φ = distribution function for a standardized normal distributed stochastic variable

$\beta(t)$ = reliability index estimated by the techniques described in (Thoft-Christensen and Baker 1982)

Different types of failure modes are considered in the expert system related to a bending failure of the deck and to compression failure of columns, associated with reduction of the reinforcement cross-section due to corrosion. These failure modes are modeled as elements in a series system such that the structure is assumed to fail if any of these failure modes is reached. To estimate the deterioration of the reinforcement cross section with time, analyses concerning chloride penetration, carbonation initiation, and active corrosion evolution are performed.

An analysis of the probability of failure considers the uncertainty connected to the inspection results modeled by the following 24 stochastic variables: strength of concrete and reinforcement bars (4 var.); load (3 var.); chloride diffusion coefficient (1 var.); carbonation coefficient (1 var.); corrosion rate (1 var.); geometrical properties (5 var.); diameter of reinforcement bars (4 var.); external chloride content (1 var.); and inspection measurement uncertainties (4 var.) (de Brito et al. 1997).

It is obvious from this description that the determination of β evolution with time (Figure 13-3) (de Brito 1992) is rather complex and generally implies the use of specific computer software (Sørensen and Thoft-Christensen 1992). For each bridge, it is necessary to define a discrete number of cross sections to be analyzed as well as the ultimate limit states considered. The failure mechanisms (in series or in parallel) must also be identified. The reliability index β is supposedly affected by a certain set of parameters, function of the degradation mechanism (e.g., steel corrosion initialized by chloride ions), which must be identified: chloride content at given depths (cores); depth of the carbonation front by indicator spray; "half-cell potential" tests; and covermeter measurements. Furthermore, the real characteristics of the defect observed during the inspection can be taken into account to update the model. An important question that must be tested before the implementation of the system is the sensitivity of the β index to each of these parameters. It is necessary to find practical ways to update the value of each parameter based on the inspection results.

Information concerning the structural design (i.e., the live loads considered) must be collected to determine the intended values of resistance for each cross section. Another possibility consists of directly providing the system with these values based on the design (design action-effects). This last solution has the advantage of dispensing with the use of finite element software, which is not easy to use by the expert system without important backup from a structural engineer who was directly involved in the design of each bridge under

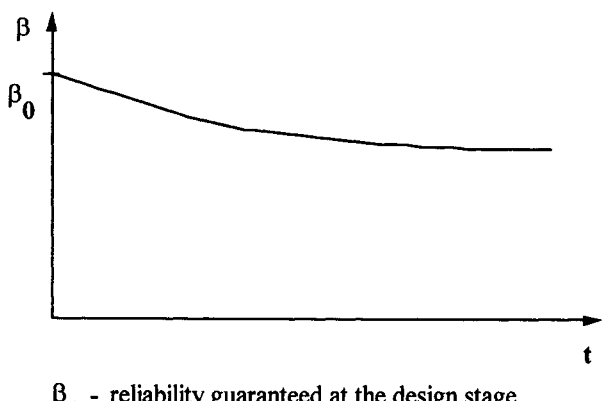


Figure 13-3. Theoretical evolution of the reliability index with time

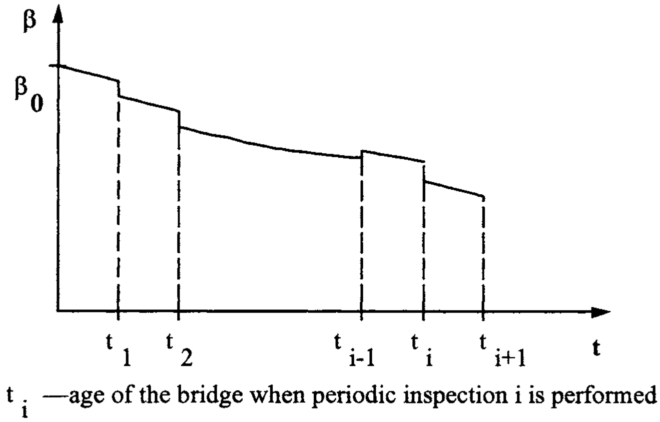


Figure 13-4. Evolution of the reliability index with time as a function of the results of the periodic inspections

analysis. Finally, the evolution of β with time is modeled by degradation mechanisms that need to be generated and calibrated.

After each periodic inspection, the reliability index β is updated, taking into account the information collected, specifically that which is relative to the parameters that condition the degradation mechanisms. As a result, a new curve for β is generated with time that will be valid at least until the next periodic inspection (Figure 13-4) (de Brito 1992).

Even though it is possible to mathematically model the evolution of the β coefficient without using the information collected during the inspection, it is useful and advisable that the system use this information to update its predicted evolution curve (Thoft-Christensen 1992). As discussed previously, it is necessary to identify the parameters that provide a model of the most important structural degradation mechanisms designated here by type 5 parameters (see Chapter 11).

The main problem with these parameters is that the greatest number of them is not available within the scope of periodic inspections (current and detailed). It is therefore proposed that, in terms of the inspection form, they be divided into two parts (de Brito 1992):

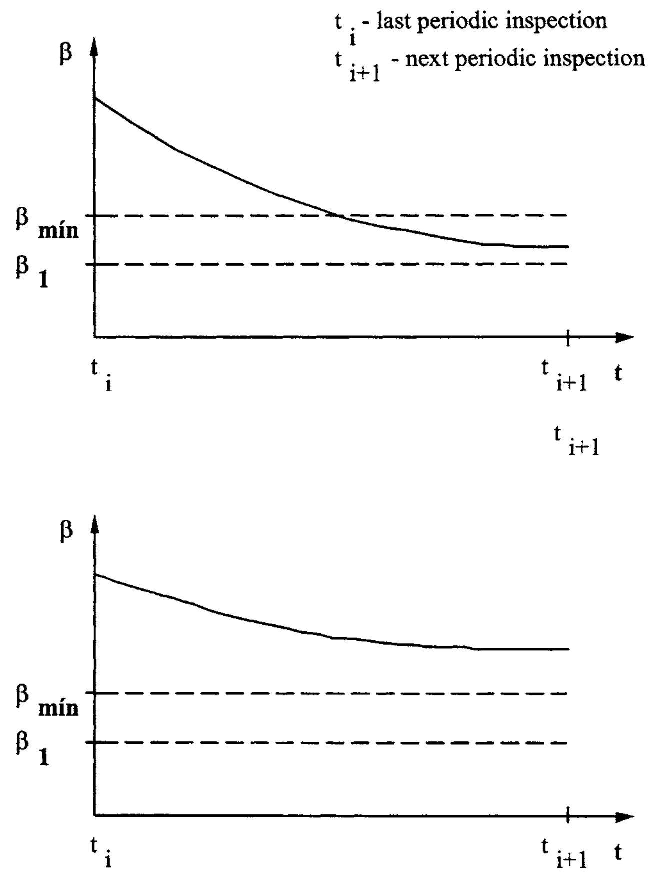
- The parameters that are usually measured during a current or detailed inspection and should be part of the main body of the inspection form; for the reinforcement corrosion mechanisms, these parameters are, among others, the depth of the carbonation measured with phenolphthalein, the existence of rust stains and their location, the fact that no signs of corrosion are detected in locations where they were expected, according to the mathematical model, bars with cover loss with or without transverse cross-section loss, etc.;

- The parameters and measurements that generally are performed only in a structural assessment and that must be included in the form following the general information concerning the structural assessment (see Chapter 9); for the same degradation mechanisms, these parameters are exemplified by measurements of the reinforcement cover with a magnetometer, measurements of the surface and in-depth chloride ion content, measurements of the potential field using a galvanic half-cell, etc.

Another problem is that the location of the measurements does not usually coincide with the discrete conditioning cross sections defined for the analysis of the ultimate limit states. The fact that corrosion has been detected in a certain location within the deck, and that it has been measured, does not give any indication about whether the same is to be expected in the lower reinforcement of the mid-span cross section or in the upper reinforcement over the supports.

Further considerations of the advantages and limitations of risk-based bridge management systems are made in Rubakantha et al. (1996).

Notwithstanding these limiting factors, a decision criterion has been defined that consists of the following: if the β value, estimated according to the last periodic inspection, is expected to fall below a predetermined value β_{min} in the period that ends at the beginning of the next periodic inspection (Figure 13-5, **a.**) (de Brito et al. 1997), the performance of a structural assessment must be proposed. The value of 3.72, corresponding to the probability of structural collapse in a 1-year period of 10^{-4}, has been proposed for β_{min} (Thoft-Christensen 1992). In principle, the structural assessment must be performed as soon as possible, even though two levels of urgency might well exist: immediate structural assessment



a.

b.

Figure 13-5. Inspection strategy submodule criteria based on a structural reliability analysis: **a.**, structural assessment must be performed before; **b.**, one must wait until the next periodic inspection without performing a structural assessment

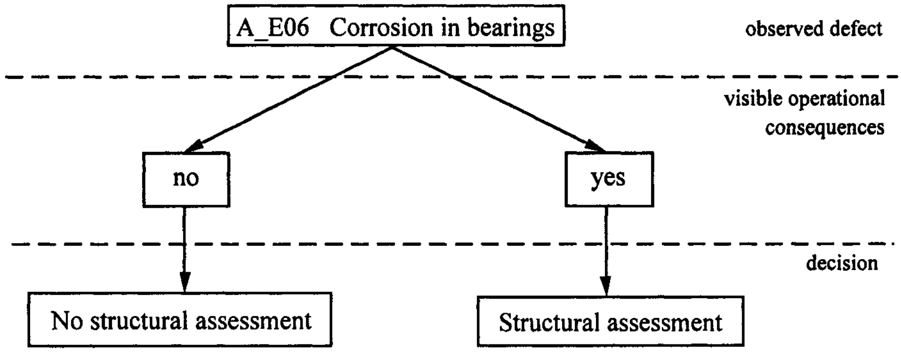


Figure 13-6. Example of a knowledge-based rule concerning corrosion in bearings

if β falls below a value $\beta_1 < \beta_{min}$ in the period discussed (the value initially proposed for β_1 is 3.0); structural assessment before the next periodic inspection if $\beta < \beta_{min}$ but $\beta \geq \beta_1$ during the same period. If, during the period mentioned, β never falls below β_{min}, no structural assessment is performed until the next periodic inspection, when the β index is updated again (Figure 13-5, **b.**) (de Brito et al. 1997).

It is reasonable to state that the decision to perform or not to perform a structural assessment is not uniquely based on reliability estimates. The knowledge of experts and good sense can and should be considered. This could be done by considering a "gray" area of values for β (e.g., from 3.0 to 5.0) in which the expert system would demand outer direction in order to reach a definite decision (de Brito 1992).

In addition to the rules related to the value of β, the system contains knowledge rules used to decide whether a structural assessment should be performed. These rules are simple and are implemented for all significant corrosion-related structural defects (e.g., in Figure 13-6) (de Brito et al. 1997).

13.2.2.2. Inspection Strategy Submodule Procedures

Submodule Input

The computer-based use of the structural reliability analysis within this submodule demands a set of data that includes (de Brito 1992):

- the inspection forms described in Chapter 9 with results of the measurements of all the parameters that may influence the determination of the β index in the critical cross-sections;
- the definition of each bridge's critical cross sections, the ultimate limit states considered. and the plausible collapse mechanisms;
- the definition of the parameters considered in each degradation mechanism, its probability modeling, the sensitivity of the model to each parameter, and the procedure of updating of the values;
- the definition of the information needed for the structural analysis (structural model and design actions or, alternatively, the resistance design action-effects for each critical cross section and ultimate limit state considered);
- the definition of how the parameters selected affect the evolution of β with time;

- the computer software for calculation, updating, and prediction of the evolution of β with time.

Submodule Output

The output of this module includes (de Brito 1992):

- a prediction of the evolution of β in all the critical cross sections and ultimate limit states in the period that goes from the last periodic inspection to the next one;
- a prediction of the evolution of the β index that models the bridge global collapse during the same period;
- a proposal about the performance or postponement of a structural assessment during the same period for each bridge with, if possible, a deadline for its implementation;
- a list of defects that must be analyzed in the structural assessment with their location and extension; another useful piece of information that could be included is the diagnostic methods recommended (correlation matrix (de Brito 1992)), equipment and personnel necessary, and special means of access (this information is provided by the user after consulting the inspection forms of previous periodic inspections and structural assessments of the same bridge).

Submodule Flowchart

Figure 13-7 (de Brito 1992) presents a flowchart of the inspection strategy submodule based on a structural reliability analysis.

This option for the inspection strategy submodule has the advantage of evaluating the necessity of a structural assessment from a global point of view (the bridge and not the defect) and of being quantitative rather than qualitative.

In Chapter 12, some considerations were expressed concerning the concept of functional service life and its interconnection with the concept of service life usually associated with structural aspects. When conceiving and designing a concrete structure, one does not have a very clear idea of what its service life will be. The common assumption for the service life of current concrete structures is approximately 50 years, which is increased to about 100 to 120 years for special structures (bridges, dams). However, this term often is not satisfied and much older structures continue to fulfill their roles. On the other hand, it has become quite common to find structures that degrade well before the predicted service life and can be considered obsolete.

The structural reliability analysis presented here is a promising path in the sense that it is possible to define very precisely the end of the bridges structural service life. In fact, as long as they are periodically surveyed by current and detailed inspections, it is possible to make predictions of the coefficient β with time (Figure 13-4) (de Brito 1992). These predictions will be used not only to make a decision concerning the need to promote a structural assessment but also to predict the end of the bridge service life. To do so, it is necessary to define a value of β that corresponds to a minimum acceptable safety level, below which society must not run the risk of keeping the bridge in service. This level will necessarily be much lower than the values of β_1 and β_{min} (Figure 13-5) (de Brito et al. 1997), since the latter correspond to situations in which the risk of structural failure is still very low and there is sufficient time to take measures that will return the bridge to its initial safety levels.

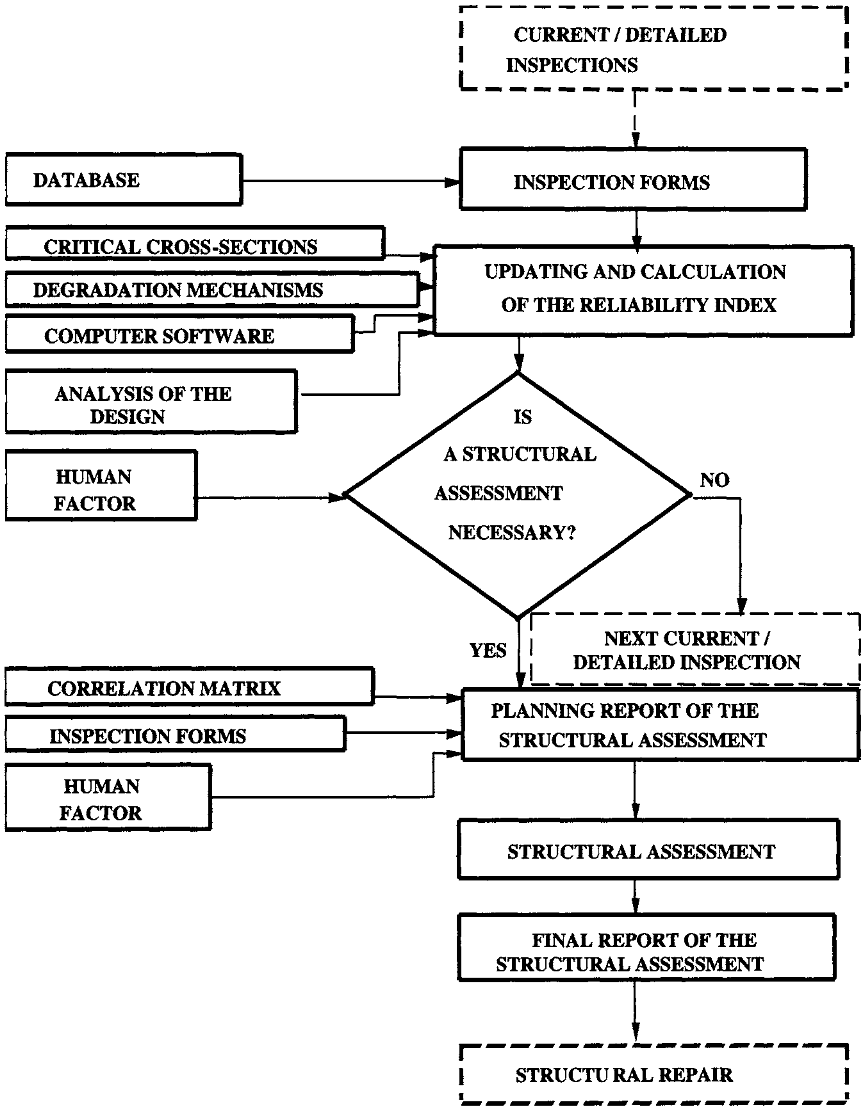


Figure 13-7. Inspection strategy submodule based on a structural reliability analysis (de Brito 1992)

If the bridge authorities wish to increase structural service life of a bridge, it should intervene (repair/rehabilitation) by increasing the value of β and thus obtain a new curve of its evolution. If, on the contrary, the predicted functional service life is equal to or less than the structural life, the best option would be not to invest any money or effort in the existing bridge, and to just let the safety level degrade progressively, as long as it never falls below the minimum acceptable level.

As long as there are no reliable degradation mechanisms, the option just described can, as an alternative, be significantly simplified by resorting to pseudo-quantitative methods of structural evaluation of existing bridges, such as those proposed by AASHTO (AASHTO 1989) and summarized in Section 6.1.4.4. In this case, the safety evaluation is made to depend on a series of factors (Table 6-13): degradation condition of the structure and the road pavement, in-situ tests performed, degree of traffic police control, periodicity and efficiency of current inspection and maintenance, refinement of the structural model adopted in the design, percentage registered of infractions to posting, and so forth.

13.3. Submodule of Repair Work Selection

As discussed previously, the bridge management system used in Pennsylvania (McClure and Hoffman 1990) is the one that more clearly defines the decision system and divides it into maintenance and repair/replacement. Even though the repair system presented next differs significantly from the Pennsylvania system, some of the basic ideas are common and attention therefore is drawn to Section 6.1.4.3.

The second part of the rehabilitation/replacement manages the situations in which the hypothesis for performing important structural repair work on the bridge is under consideration, and the options for enhancement of its capacity (deck widening or strengthening of the structure), vehicles loads posting, and even straight replacement are also admissible. The repair work includes both structural repair and semistructural repairs even though there may be consequences in terms of the bridge's functionality.

This submodule must be used whenever a structural assessment is performed and is ignored when, in the inspection strategy just described, it is concluded that no structural assessment is necessary until the next periodic inspection (Figures 8-3 and 8-7). Even when it is decided that it is more economical not to perform any structural work on the bridge related to the defects detected, this decision must be made using the present submodule and must be based on an economic analysis.

13.3.1. Decision Criteria

To know the budget constraints is fundamental to the decision because it is practically impossible to predict the number of bridges that will require repair each year or the work that will be needed. A bridge may go without repair for 10 or more years and, in the following year, undergo rehabilitation that costs almost as much as the initial construction. At the beginning of each year, the budget for those entities that manage all work that goes beyond maintenance and small repair, and the funds allocated to every bridge in the network, is disclosed. The background for decision making follows:

1. There is a network of bridges that are regularly inspected;
2. Some of them have important structural or functional defects. These may result from a degradation of their initial characteristics after the construction or from a change/increase in the requirements demanded of them (heavier traffic volume or loads);
3. Ideally, each of these bridges has been the object of a structural assessment in which all the defects have been not only identified but also quantified. This is not always possible, which makes the decision more difficult and more error-prone. The structural assessment provides a list of work needed (the item "Repair Work Needed" of the respective inspection form in the database, as described in Chapter 9) to restore the original characteristics of the bridge, which strictly involves repair work. This list must be made in accordance with the repair forms presented in Chapter 10;
4. For each of these bridges, present and future service inadequacies must be identified (e.g., the bridge has only two lanes in each direction and three lanes are

needed or will be in the short term). Solutions must be studied to eliminate these inadequacies, which leads to the preparation of traffic and structural preliminary studies;

5. Ideally, when the decision has to be made, the following information about the bridges included in the analysis must be available:
   - an estimate of the repair work needed in accordance with the repair forms; by multiplying each task by the respective unit price, an estimate of operational costs is obtained;
   - an estimate of the enhancement work required and an approximate budget prepared by the management department;
   - a proposal for replacement of the existing bridge (if that is a possibility under analysis) with approximate dimensions and a rough budget for the new bridge;
   - more than one solution may be studied for each of these options; if there is enough time, studying more solutions will, in principle, lead to a better decision.

It is necessary to make a decision based on all the information described. This is basically the result of an economic analysis that can be made at three different levels according to what is being compared (de Brito 1992):

- Level 1—the elimination of the defect;
- Level 2 —the bridge repair;
- Level 3—the management of the bridge network.

In the description of each of the decision levels that follows, the option under analysis is to repair the bridge. The less common cases, which involve enhancement of the bridge's capacity and replacement of the bridge, are discussed later in this chapter.

13.3.1.1. Level 1

At level 1, several solutions for repair of a certain defect are proposed. The various solutions have different costs but also different degrees of efficiency. At the defect level, the efficiency can be measured by the repair life cycle (i.e., the time spent from completion of the repair until a new repair is needed).

For each location of a defect, a different degradation mechanism can be applied. This mechanism is calibrated as time passes by integrating the information collected during the periodic inspections. Every time it is necessary to make a decision, a residual service life can be predicted. For each repair technique, a life cycle must be predicted with the help of a deterioration model and also by taking into account the intrinsic aggressiveness of the location. Until sufficiently accurate mathematical models of degradation are developed, it is possible to use tables such as those presented in the annex of the SFAM (Reel and Muruganandam 1990a), which contains average service life periods for different materials, elements, and repair techniques.

At this level, and in view of the time at which the decision is made, the bridge initial costs (C_0), inspection costs (C_I), and maintenance costs (C_M) (defined in Chapter 12) should be considered as irrelevant to the decision, because they are not altered regardless of what decision is made relative to the defect repair. The decisions must be made in accordance with the cost efficiency index (CEI) of each option (Aylon 1990).

The cost efficiency index gives a comparative indication of the actions planned versus the option of doing nothing. A CEI value higher than 1 indicates that, from an economic point of view, the actions planned are better than the option of doing nothing and vice versa. An option with a CEI greater than that obtained from another option must be preferred because it constitutes a better investment.

The concept of return of investment (ROI) (Aylon 1990) can also be used. The ROI index is equal to the CEI index multiplied by the interest rate r. A value of ROI higher than the interest rate identifies an efficient plan from an economic point of view.

The most important concept is that the decision not only must be based on the immediate costs of the option (i.e., lower costs $\Rightarrow$ better solution) but also must take into account the benefits (Chapter 12) that can be obtained from it. The objective is to maximize benefits−costs (or to minimize costs−benefits) and not simply to minimize costs.

The value of CEI must be determined for each option (Branco and de Brito 1995):

$$\mathrm{CEI} = \frac{(C_R + C_F - B)_{\mathrm{repair}}}{(C_R + C_F - B)_{\mathrm{no-action}}} \tag{13-3}$$

where

C_R = repair costs [$]

C_F = failure costs [$]

B = benefits [$]

$$C_R = C_{RSA} + C_{RSR} \tag{13-4}$$

where

C_{RSA} = structural assessment costs [$]

C_{RSR} = structural repair costs [$]

Next, some indications are given on how to calculate the different terms in Equations 13-3 and 13-4. For the no-action option, $C_{RSR} = 0$. For the repair option C_{RSR} is the predicted cost of the repair, including all labor, materials, equipment, administration, and quality control costs. As for the cost C_{RSA}, ideally, it should be the same for both options since it is recommended that, whatever the final decision, a structural assessment be performed to identify and quantify the necessities. However, if the decision is made before the structural assessment, $C_{RSA} = 0$ for the no-action option. On the other hand, the uncertainties will be greater and the failure costs must be increased for this option (by increasing the collapse probability P_f). In the determination of C_{RSR}, the fixed costs associated with the repair of each defect must be shared whenever more defects of the same type occur on the same bridge.

The failure costs and benefits also should be predicted; $B = 0$ for the no-action option. If the maintenance costs are substantially reduced by the option of repairing (which is unlikely in most cases), this decrease can also be included in the respective benefits. To quantify the failure costs and benefits, an economic analysis in a certain period (preferably the residual service life RL_1 of the element in which the defect was detected, in the case of doing nothing to eliminate it) must be made. After that period, repair or replacement of the element is paramount. For the repair option, there is still residual service life RL_2 after the period RL_1 elapses that must be evaluated (de Brito 1992):

$$RL_2 = SL_2 - RL_1 \tag{13-5}$$

where

RL_2 = residual service life of the element after RL_1 is through, in case the repair option is chosen when decision time comes [year]

SL_2 = expected service life of the element in case the repair option is chosen [year]

RL_1 = residual life of the element in case nothing is done [year]

The residual value RV_2 of the repair option after time RL_1 must be quantified (by using, for example, Equation 12-51 or 12-52) and include the corresponding benefits.

$$C_F = C_{FSF} + C_{FFF} \tag{13-6}$$

where

C_{FSF} = structural failure costs [$]

C_{FFF} = functional failure costs [$]

If the fact that the defect exists is not a jeopardizing factor of the bridge's functionality, the costs C_{FFF} must be ignored in this analysis because they are the same for both options. The cost of the actual structural collapse of the bridge C_{FF} (Equation 12-45) is also basically the same. However, it is possible that the collapse probability P_f is greater for the no-action option, as well as the costs C_{FSF} and C_{FF}. The quantification of this increase is a difficult problem that must be solved through simple formulas.

By using the preceding equations, it is possible to obtain a CEI index for each repair technique considered plausible for a particular type of defect found on the bridge. The option with the maximum CEI value is, in principle, the best choice.

13.3.1.2. Level 2

At level 2 of the decision, a list of the different types of defects detected at the bridge has already been prepared. For each one, a level 1 analysis has been prepared and the repair technique with the maximum CEI has been chosen. A new list of the defect types in terms of the optimal repair techniques together with the respective values of CEI_{max} can then be prepared (Table 13-1) (de Brito 1992).

The budget for repair assigned to each bridge is limited and not all defects can be repaired. The decision at level 2 consists of first repairing the type of defect with the highest value of CEI_{max} (CEI_1) and so on. From the budget assigned to the bridge under analysis the costs C_1, C_2, etc. are consecutively deducted until the budget is exhausted. If the bridge budget depends on the general network budget, level 3 decisions must be used.

When deducting the costs C_i from the bridge budget, a small reduction of all the costs C_j ($j = 1$ to i) can be made to take into account that the fixed costs of each repair technique can be shared among the number of works planned.

Table 13-1. Defects and their optimal repair techniques according to the respective CEI_{max} value (individual costs)

Type of defect	Optimal repair technique	CEI_{max}	Cost [$]
....	....	CEI_1	C_1
....	....	CEI_2	C_2
....	....	....	....
....	....	CEI_i	C_i
....	....	....	....

$CEI_{i+i} \leq CEI_i \leq CEI_{i-i}$

13.3.1.3. Level 3

At level 3, the most important decision level, the options are made from a global point of view (i.e., how the option to repair a certain bridge affects all the other bridges included in the network. The great options for future use of each bridge (enhancement of the capacity or replacement) must be made at this decision level.

When the decision system reaches level 3, it is assumed that all bridges with significant structural or functional defects have been the object of an economic study at level 2. For each of these bridges, a new list of the types of defects detected must be prepared. Instead of having individual values of CEI_{max} and costs for each repair technique, the accumulated costs and indexes ACEI are used in the list (Table 13-2) (de Brito 1992).

$$\mathrm{ACEI}_i = \frac{\sum_{j=1}^{i} C_j \mathrm{CEI}_j}{\sum_{j=1}^{i} C_j} \tag{13-7}$$

$$\mathrm{AC}_i = \sum_{j=1}^{i} C_j \tag{13-8}$$

Table 13-2. Defects and their optimal repair techniques according to the respective ACEI value (accumulated costs)

Type of anomalia	Optimal repair technique	Accumulated CEI (ACEI)	Accumulated cost (AC) [$]
....	....	$ACEI_1$	AC_1
....	....	$ACEI_2$	AC_2
....	....	....	....
....	....	$ACEI_i$	AC_i
....	....	....	....

$ACEI_{i+i} \leq ACEI_i \leq ACEI_{i-i}$

The value $ACEI_i$ represents the cost efficiency index for performing all the repairs necessary to eliminate defects of types 1 to i and, logically, the value decreases with the number of types of defect repaired. When determining the value of the accumulated costs, the correction mentioned previously (concerning the reduction of unit costs with an increase of the number of defects repaired) must be taken into account.

The decision at level 3 consists of first performing a repair (or a group of repairs) with the highest ACEI index (from all bridges included in the network) and so on. From the global budget the accumulated cost of the repair with the highest ACEI value is deducted and so on. Obviously, whenever a group of techniques for a certain bridge that includes n techniques is included in the list, the group of repairs in the same bridge with $(n - 1)$ techniques is eliminated from the list and the available budget is corrected by the respective cost.

13.3.1.4. Particular Cases

The description of the three decision levels was made based on an option for repair of the bridge (i.e., of restoring its original characteristics). However, the underlying concept can be generalized to situations in which the possibility of enhancing the existing bridge capacity or even replacing it is under consideration.

Structural Strengthening/Deck Widening

When the possibility of enhancing the capacity of a bridge is under consideration, several options may be chosen: to do nothing and accept the structural inadequacy until a specific budget is made available; to increase the bridge capacity (by widening the deck, strengthening the structural elements, and/or changing the structural behavior and redistributing the internal action-effects); to build a new bridge (that replaces the existing bridge or runs parallel to it). For each situation, the economic implications are the most important considerations for decision making and, together with political and architecturally based issues, greatly reduce the impact of technical criteria (structural engineering or others). Therefore, it is not wise to create a decision-making system for enhancing existing bridges based only on structural aspects.

Let us suppose that several structural strengthening and/or deck widening options are under analysis. For each option, it is possible to know the costs of implementation (including the structural assessment costs and probably some structural repair costs, since it does not make sense to enhance a bridge without repairing at least some of the defects). These costs replace the repair costs in the determination of the CEI index (Equation 13-3). It is also possible to estimate failure costs and benefits of the option.

The benefits arise from an increase of the maximum axle load allowed on the bridge, Q_0, and/or from an increase of traffic flow allowed at the design speed, TF_0. The estimate can be made based on the criteria and equations presented in Chapter 12.

To quantify the functional failure costs, the reference situation must be the bridge with the present design functionality after it has been repaired (but not enhanced). The option of doing nothing involves only functional failure costs if the defect (or defects) detected affect the bridge service level. The functional failure costs of the bridge enhancement option are nil, because it is assumed that its functionality increases. This increase relative to the initial design must be considered as a benefit derived from this option.

The structural collapse cost C_{FF} increases with enhancement of the bridge because it now represents a higher investment and replacement with a similar bridge is more expensive. However, the collapse probability P_f during the economic analysis reference period should decrease significantly, and therefore some reduction in the structural failure costs C_{FSF} is expected after the enhancement.

To compare the option of capacity enhancement with the other options, the value of the CEI index must be related to the residual service life RL_1 of the bridge (or the element) in the no-action option. Since the expected service life RL_2 of the enhanced bridge is greater than RL_1, its residual value RV_2 after RL_1 must be taken into account for the benefits through the use of Equation 12-51 or 12-52.

The value of the CEI index for each enhancement option must be compared with the corresponding values of CEI_{max} in a level 2 analysis (if the budget for the bridge under analysis is independent of the network budget) or with the values of ACEI in a level 3 analysis. The decision criterion is the same.

Replacement

The context associated with the hypothesis of replacing the bridge is very similar to the proposal for the enhancement option. The main difference is that the repair costs (or of enhancement) are replaced by the costs of replacement.

It is necessary to undertake an economic study to compare the rehabilitation costs and the residual service life of the existing bridge with the respective value of building a new bridge. A present value analysis that takes into account initial costs, life cycle, residual service life, user costs, probabilities of occurrence, and a sensitivity analysis (Reel and Muruganandan 1990a) (i.e., by varying the interest rate) must be used. The inflation effect, when construction prices progress with an inflation rate different from the general prices, must also be included. The initial costs of replacement include design costs, construction costs, and assorted expenses such as the existing bridge demolition, approaches, utilities, creek diversion, traffic detours, and others (Reel and Muruganandan 1990a).

Two different options may be used to determine the type of bridge replacement (de Brito 1992):

- rebuilding a bridge identical to the existing bridge—in this case, the functional costs and benefits are the same as for a straight repair; however, the residual value of the new bridge after the economic analysis reference period is probably greater than the corresponding value for the existing repaired bridge, since the life cycle of a new bridge is almost surely greater than the life cycle for a repair technique;
- building a better bridge (wider or with a greater load-bearing capacity)—in this case, the functional costs and benefits must be estimated and compared with the additional cost of the enhancement to know which of the two options has the greater CEI index.

The values of the CEI index for the various options of replacement are then used for a level 2 or 3 analysis as explained previously.

13.3.2. Repair Work Selection Submodule Procedures

13.3.2.1. Submodule Input

This submodule input includes (de Brito 1992):

- the forms described in Chapter 9 with the identification, degree, and extension of all the structural defects detected in the structural assessment;

- the repair work needed forms prepared by the inspector with relatively accurate estimates of the work quantities;
- a correlation between each structural defect detected and at least one repair technique that will eliminate it;
- a list of unit prices for each repair technique considered within the repair scope (preferably in man hours/unit);
- economic data for the current year (cost per man hour, cost of traffic delay, etc.);
- interest and inflation rates predicted for the future years;
- data about the present and future years (average daily traffic, distribution of the traffic flow throughout each day, distribution of traffic in terms of weight, detour lengths, etc.);
- present and expected future bridge capacity for live loads in terms of detected deterioration;
- reliable mathematical models for the deterioration mechanisms or, alternatively, simplified tables to predict the total or residual service life for each repair technique and for the no-action option;
- current year repair budget.

13.3.2.2. Submodule Output

This submodule output includes (de Brito 1992):

- a list of repair work to be performed during the current year based on the urgency of rehabilitation; the list identifies the tasks required by the bridge and the standard repair technique; it provides material quantities and costs and should also identify the structural defects that the technique supposedly eliminates, with the same reference as that in the inspection forms;
- a list of repair work needed that will not be performed in the current year because of budget constraints, with the same information included as on the list previously described; this list can be used to justify a request for an increase in the repair budget and to select the work that cannot wait until next year.

In the present section, as well as in the two previous sections, the word *repair* must be seen in its broader sense to include capacity enhancement and bridge replacement solutions.

Figure 13-8 (de Brito 1992) presents a flowchart of the second part of the rehabilitation/replacement subsystem (the repair work selection submodule).

13.3.3. Submodule Implementation Strategy

The implementation of this submodule is very complex and involves a wide range of information that, at least at an initial stage, will not be available. Therefore, it is useful to define a strategy of development that allows, on the one hand, to start obtaining practical results from the beginning of its implementation and, on the other hand, to calibrate the models and parameters used to obtain results that are realistic.

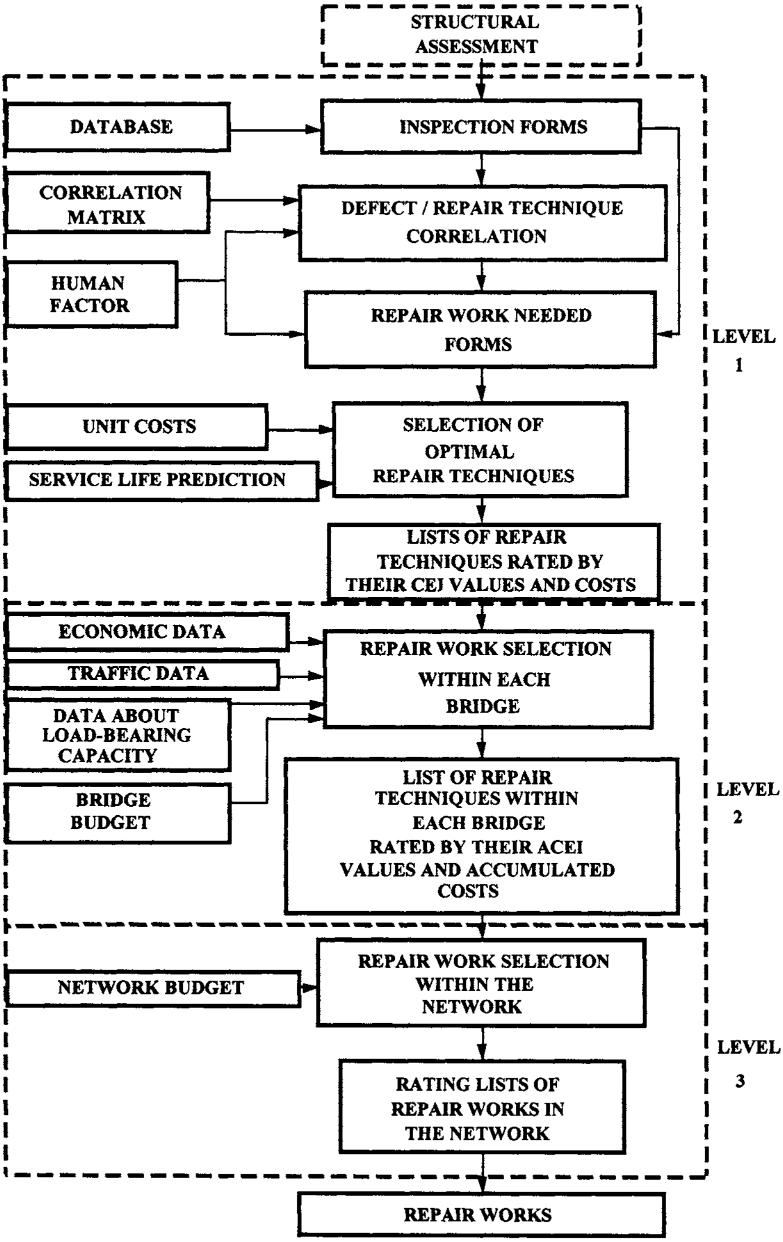


Figure 13-8. Repair work selection submodule of the rehabilitation/replacement subsystem

For these reasons, it is proposed that the submodule initially be prepared to take into account only the repair of the bridges, setting aside capacity enhancement or replacement. It is also proposed that initially only the economic analysis decision of levels 1 and 2 are implemented, thus leaving level 3 for a subsequent stage.

During periodic inspections within the scope of the maintenance/small repair subsystem, some typical structural defects are also detected that must be eliminated within the

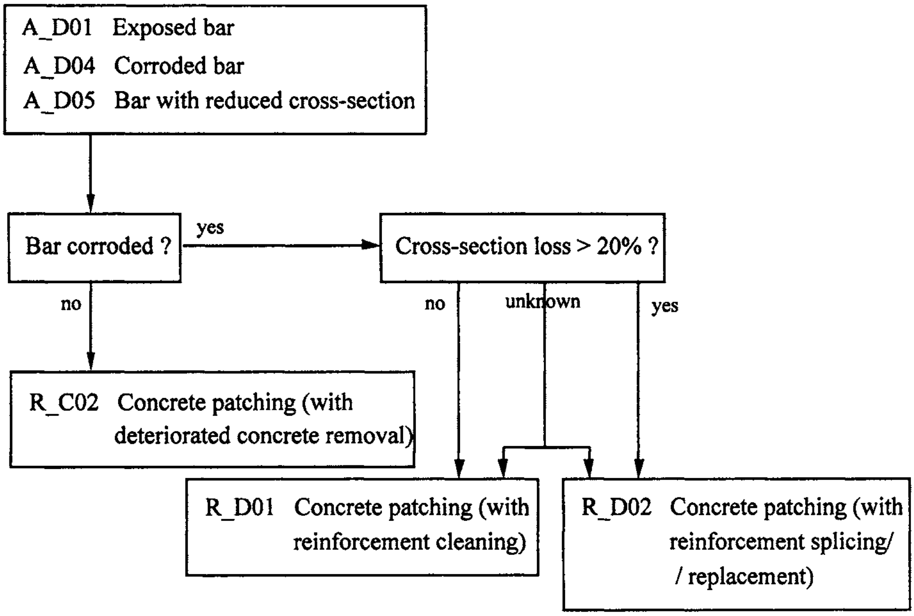


Figure 13-9. Example of repair technique selection

scope of the rehabilitation/replacement subsystem. Nevertheless, these defects must also be classified according to the same type of criterion (defined in Chapter 11) and the same type of information must be recorded in the inspection form. A similar system is presented in Matsui and Muto (1993), where deteriorated reinforced concrete slabs of highway bridges are evaluated and rated. The damage patterns are then related to residual load-carrying capacity and lifetime, thus allowing the decision on whether to make repairs and when to do so.

Type 1 parameters are used to rate the defect, type 2 parameters are inserted in the inspection form, type 3 parameters are determined and included in the provisional form "Repair Work Needed," and type 4 parameters are used to eliminate repair techniques not appropriate in view of the particular characteristics of the defect detected. For the same purpose of type 4 parameters, decision-making flowcharts (Figure 13-9) (Branco and de Brito 1995) can be used.

However, there are several differences relative to the description that appears in Chapter 11. The most important is the fact that all repair work is always preceded by a structural assessment. During that inspection, all the characteristics related to the structural defects detected are to be investigated in order to provide definitive answers regarding the repair techniques to be selected and the respective type 3 parameters necessary to quantify costs. Only then can there be a definitive version of the "Repair Work Needed" form. It is proposed that a rating for structural defects investigated during the structural assessment be made. This can be done by the use of the number of points correspond-ing to the defect rating (e.g., only defects to which a number of points higher than 50 has been assigned in the two first criteria—urgency of rehabilitation and importance to structural stability—would be investigated) or by identifying the defects that have been mainly responsible for the decrease in the predicted value of the β index during the period under analysis.

Some simplifications are proposed in the calculation of the values of Equation 13-3. Costs for the repair option can be divided into two parts: direct costs of the repair C_{R_0} and functional costs of the repair C_{R_F} (de Brito 1992).

$$C_R = C_{R_0} + C_{R_F} \tag{13-9}$$

Direct costs for repair are the expenses incurred for the design and execution of the repair itself (Equation 13-4), the latter of which can be divided into fixed costs and variable costs (Equation 11-1). Their global value can be estimated by using the repair forms and can be inserted in the "Repair Work Needed" form together with other information that is considered useful.

Functional costs of the repair are the values attributed to hindrances to normal traffic flow across the bridge during the execution of repairs. As explained later in this chapter, these costs are approximately proportional to the average daily traffic on the bridge TF_d [vehicle/day] (Equation 12-78) (de Brito 1992).

$$C_{R_F} = \mathrm{ERT}\,\frac{n_{l_r}}{n_l}\,k_1\,TF_d \tag{13-10}$$

where

ERT = estimate of the time for repairs [day(s)]

n_{lr} = average number of lanes affected simultaneously during the repair

n_l = total number of lanes on the bridge

k_1 = proportionality coefficient provided by the user (at least at an initial stage this coefficient should be kept constant at present value prices) [$/vehicle]

Quantification of the functional failure costs and benefits is a long and intricate process, as described in Chapter 12, in which a very high number of variables are involved: annual traffic at the bridge, structural collapse probability, service life period, value attributed to the user's time, average detour length, structural and functional capacity, accident rate on the bridge's itinerary (as defined in Chapter 12) and alternative detours, vehicles maintenance costs, and so on. It is proposed that, at an initial stage, the determination of these costs and benefits be simplified according to the following equations (de Brito 1992). However, it will become necessary in the middle term to include more accurate calculations of these values for this submodule because they represent more than 95% of the global costs and, therefore, the results of the economic analysis would tend not to be significant without the more accurate calculations.

Based on practical examples used to test the computer algorithm COSTS and specialized references (Sinha 1986), it has been concluded that it is possible to obtain acceptable estimates of failure costs and benefits by making them proportional to the average daily traffic on the bridge TF_d. However, the proportionality coefficients are different from one bridge to another and have to be estimated by the user at this stage in the development of this submodule.

$$C_F = C_{FSF} + C_{FFF} \tag{13-11}$$

where

C_{FSF} = structural failure costs [$]

C_{FFF} = functional failure costs [$]

$$C_{FSF} = P_f\,[N_r\,k_0\,TF_d + C_0] \tag{13-12}$$

where

N_r = number of days estimated for the replacement of the collapsed bridge [day]

P_f = probability of structural collapse of the bridge

k_0 = proportionality coefficient provided by the user [$/vehicle]

C_0 = bridge replacement cost; at present value prices, it can be considered equal to the initial cost of the collapsed bridge; it is assumed that the old bridge is replaced by an identical bridge (see Chapter 12)

$$C_{FFF} = N\,k_1\,TF_d \tag{13-13}$$

where

N = number of days of the period under analysis [day]

k_1 = the same coefficient as in Equation 13-10 [$/vehicle]

The average daily traffic and the probability of structural collapse are the only time-dependent variables. The collapse probability is calculated simultaneously with the β index, but it could be made to depend on the repair technique used (and could be higher for the no-action option). At an initial stage, the daily traffic can be made to vary linearly with time (de Brito 1992):

$$TF_{d_i} = TF_{d_0} + \alpha\, i \tag{13-14}$$

where

TF_{d_i} = average daily traffic in year i [vehicle/day]

TF_{d_0} = average daily traffic in the last year from which there are results (year 0) [vehicle/day]

α = annual increment of average daily traffic [vehicle/day year]

The benefits can be quantified in an identical way (de Brito 1992):

$$B = N\,k_2\,TF_d \tag{13-15}$$

where

k_2 = proportionality coefficient provided by the user for the specific bridge under analysis [$/vehicle]; this coefficient can be considered equal to the coefficient k_0 mentioned previously

All of the proportionality coefficients are assumed to be constant at present value prices, which, even though not exact, is an acceptable approximation.

The flowcharts shown in Figures 11-2 to 11-4, which are valid for maintenance, can also be applied to repair with the following adaptations:

- the selection of the repair technique becomes the selection of the appropriate options for the repair technique to be analyzed comparatively;
- the form "Maintenance Work Needed" becomes the provisional version of the form "Repair Work Needed" after the periodic inspection and the definitive version of the form "Repair Work Needed" after the structural assessment;
- the rating of repair techniques in terms of priority of action is made at two levels: level 1, in which the best repair technique for a specific defect is selected (based on the CEI index); and level 2, in which the most urgent repair techniques in a certain bridge are selected (based on the CEI_{max} index).

13.3.4. Examples of Application of the Algorithm COSTS to the Submodule

The computer algorithm COSTS, as described in Chapter 12, can be used to perform some relatively simple economic analyses related to the present decision submodule.

The objective of this section is to illustrate the use of the decision criteria described in Chapter 13 with two case studies: the first within the repair scope and the second within the capacity enhancement scope (deck widening).

13.3.4.1. Wearing Surface Repair

In this example (de Brito and Branco 1994), bridge 2 (see Section 12.3.4.) from the examples for the use of the algorithm COSTS, presented fully in de Brito (1992), is used. It concerns a bridge in service for 4 years, during which time it was possible to collect the necessary data to predict all future costs and benefits by using linear regression techniques. The end of its service life was predicted for the year 2030. Inflation and discount rates were considered constant over the entire period and equal to 10% and 4%, respectively. All analyses were based on the present value prices of 1991.

Under normal circumstances, the bridge deck is repaved every 5 years; the first time is predicted for the analysis reference year. During that operation, every attempt is made to avoid disturbing normal traffic flow. Nevertheless, specialized personnel and equipment and better quality materials are usually not used (option 1). To reduce the functional costs associated with this operation, two alternative options are proposed:

- option 2—to use extended working hours and more personnel in order to reduce repair time and the average number of lanes closed to traffic during that period;
- option 3—to use better materials and more advanced technology to increase the service life of the road surface and further decrease the disturbance of traffic while the repairs are being performed; in this option, however, there is an increase in repair time.

Table 13-3. Bridge repavement options under analysis

Option	C_{Ri}	T_{VU}	E_{RT}	n_{lr}
1	10,000	5	4	1.0
2	15,000	5	3	0.8
3	25,000	10	7	0.6

The three options are described in Table 13-3 (de Brito and Branco 1994) in which the abbreviations have the following meanings:

CR_i = total estimated repavement cost (considered constant throughout the service life of the bridge for each repavement option) [$]

TVU = expected service life of the road repavement (i.e., time between repairs) [year]

ERT = estimated repair time [day]

n_{lr} = average number of lanes closed to traffic during repair time

Future traffic volume was predicted using data collected during the bridge's years in service. It is considered that only 20% of the traffic volume impaired by the repavement works will choose other road alternatives (Equation 12-78). The others will endure delays when crossing the bridge. Therefore, the comparison of the repavement operations will only be related to functional failure costs of two types (see Chapter 12):

- costs due to traffic delayed C_{FFFD};
- costs due to traffic detoured in terms of volume C_{FFFV}.

To apply Equation 13-3, the reference option (usually the no-action option) is option 1 described previously, because it makes no sense to even consider the hypothesis of not repaving the bridge. Benefits are determined as discussed in Chapter 12), and they are the same for all options considered; what is going to change is the functional failure costs and, obviously, the repair costs. How various relevant costs vary with time for each option is represented graphically in Figures 13-10 to 13-13.

Figure 13-10 (de Brito and Branco 1994) shows the direct costs for each alternative. They are nil except in the years when the wearing surface is replaced, according to the calendar and costs described in Table 13-3 (de Brito and Branco 1994).

The delayed traffic costs during repavement C_{FFFD} are shown for each alternative in Figure 13-11 (de Brito and Branco 1994). Like the latter, they only occur in the years in which the surface is to be replaced. The way that they initially increase (at present value prices) followed by a subsequent decrease, is explained by the fact that the annual volume of traffic has been considered equal to a fixed number of vehicles during the entire service life of the bridge. This fixed number corresponds to an annual rate of increase in the total delay time (which is not proportional to traffic volume) that is higher than 4% (the discount rate) in the first few years, but less than that value as the total annual volume of traffic increases.

On the contrary, the light traffic detoured costs C_{FFFV}, shown in Figure 13-12 (de Brito and Branco 1994), start decreasing with time from the beginning of the analysis because the percentage of the annual traffic increase is always lower than the discount rate.

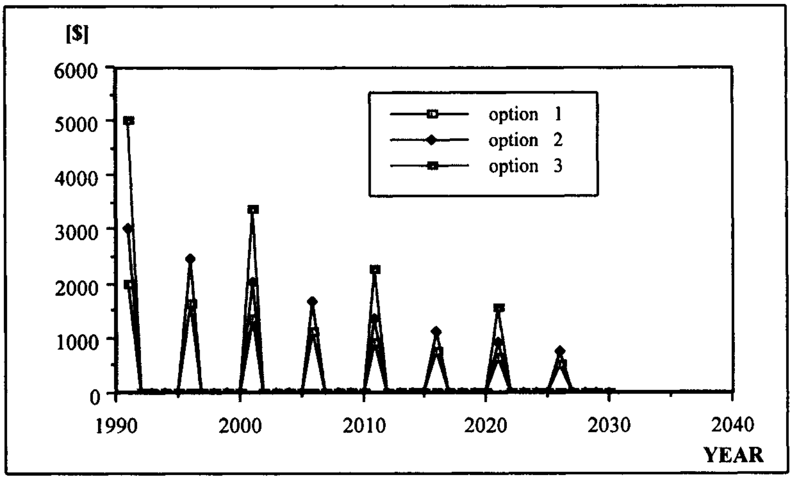


Figure 13-10. Comparison of the direct costs C_R for each repavement option (see Table 13-3)

Figure 13-13 (de Brito and Branco 1994) shows the total number of vehicles detoured during repavement operations is proportional to the total number of vehicles that use the bridge, whose constant annual increase shows up clearly.

In Table 13-4 (de Brito and Branco 1994), the main results of the economic analysis are presented. The costs and benefits in the table are in $1,000. The main conclusion from this analysis is that the best option is the third option, followed by the second option. The fact that the coefficients CEI are very close is due to the benefits being the same for all options and also having a great influence on the global results (if only the failure costs are measured relative to option 1 and the sign of the factor CEI is inverted, the results for options 2

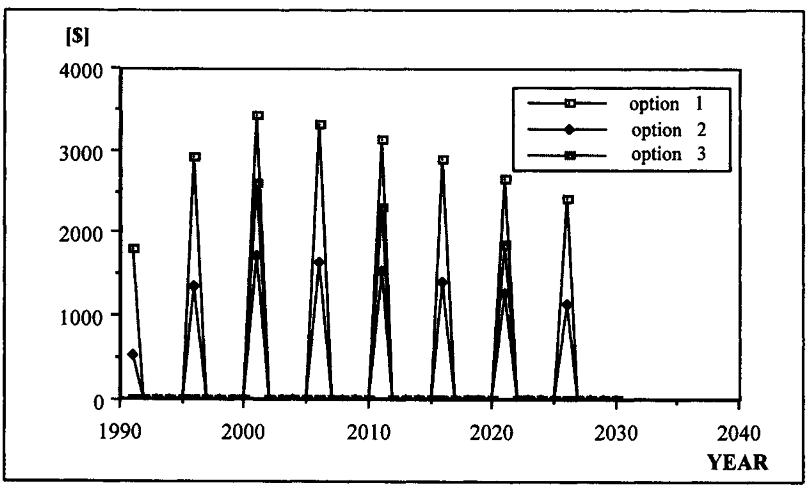


Figure 13-11. Comparison of the functional failure costs due to traffic delay C_{FFFD} directly caused by each repavement option (see Table 13-3)

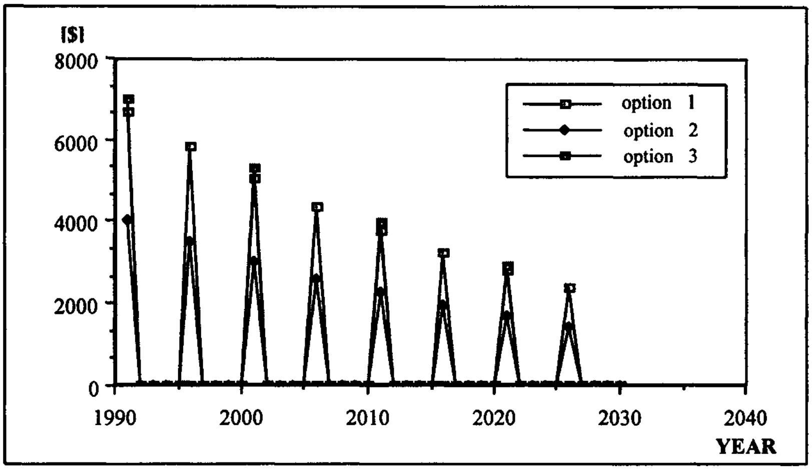


Figure 13-12. Comparison of the functional failure costs due to traffic detoured in terms of volume C_{FFFV} directly caused by each repavement option (see Table 13-3)

and 3 would be increased to 1,380 and 2,078, respectively, values that show the right decision to be made much more clearly).

A sensitivity analysis was performed for the following two parameters:

- future discount rates—their reference value was 4%; the variation introduced was ±3%;
- future annual traffic volumes—their basic values were predicted using linear regression techniques and the results obtained from the first years that the bridge was in service; the variation introduced was ±15%.

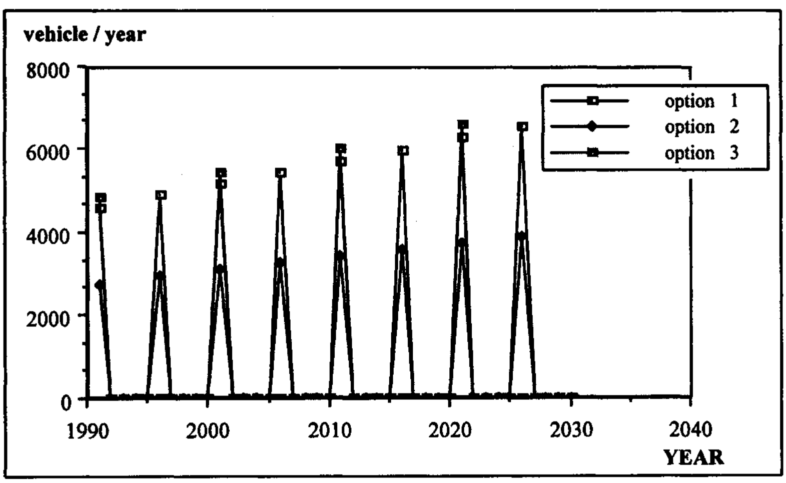


Figure 13-13. Comparison of the detoured traffic volumes directly caused by each repavement option (see Table 13-3)

Table 13-4. Economic analysis results for all the repavement options

Option	ΣC_R	ΣC_F	ΣB	CEI
1	44.46	49955.97	91833.67	1.0000
2	66.69	49827.89	91833.67	1.0025
3	61.01	49802.56	91833.67	1.0033

The main results (the CEI coefficients) of the sensitivity analysis are presented in Table 13-5 (de Brito and Branco 1994).

Changes in the future discount rates do not seem to seriously affect the previous results. On the contrary, the sensitivity analysis of future traffic volume yiels some interesting results:

- If the real volume of traffic is significantly lower than the initial prediction, the alternative options (2 and 3) lose some of their advantage in relation to option 1, because they involve higher direct costs and their main advantage is the reduction of functional failure costs; option 3 is the most penalized because it is also the most sensitive to this effect;
- If, on the other hand, the real traffic volume is much higher than the predicted volume, the initial conclusions are significantly emphasized; however, the delays of traffic trying to cross the bridge increase in such a way that the global costs over the service life of the bridge become bigger than the global benefits over the same period (the coefficients presented are the inverse of Equation 13-3 in order to be compared with the other values of Table 13-6) (Branco and de Brito 1995);
- The final conclusion is that option 3 is the best, even though the predictions of future traffic volume must be as accurate as possible.

13.3.4.2. Deck Widening

In the previous example, it was concluded that, in accordance with the predicted rate of evolution of traffic volume, the increase in the traffic delayed costs C_{FFFD} is so significant that, at the end of the bridge's predicted service life, the functional failure costs C_{FF} exceed the annual global benefits B (Figure 13-14). If the prediction of future traffic volumes significantly fails by a defect, it is concluded that the sum of benefits is exceeded by the cumulative failure costs at the end of the bridge's service life.

Table 13-5. Sensitivity analysis in terms of the CEI index to discount rates and traffic volumes

Option	$r - 3\%$	$r + 3\%$	$T - 15\%$	$T + 15\%$
1	1.0000	1.0000	1.0000	1.0000
2	1.0042	1.0019	1.0009	1.0034
3	1.0058	1.0023	1.0012	1.0042

Table 13-6. Results of the economic analysis (CEI values) for the deck widening options

Option	ΣC_R	ΣC_F	ΣB	CEI
0	0.0	59 607.9	110 200.4	1.0000
1	51.3	14 113.8	110 200.4	1.8982
2	42.2	14 328.2	110 200.4	1.8942
3	34.6	16 533.0	110 200.4	1.8507
4	28.5	21 376.1	110 200.4	1.7551

Figure 13-14 also gives an indication of the duration of functional service life of the bridge if no funds are made available to increase its traffic capacity. In fact, to maximize the net present value of the bridge, it should be replaced by a new bridge in the year in which the total annual costs are equal to the total annual benefits (year 2017), assuming that the cost of opportunity of capital equals the discount rate (Bhandari and Sinha 1979). This analysis is made more complex if there are alternatives concerning other bridges within the system.

Based on these conclusions, it has been decided to increase bridge traffic capacity. A preliminary structural analysis revealed that it was feasible to widen the deck from two lanes to three lanes (the extra lane will be used alternately, according to the rush hours), as long as the infrastructure undergoes strengthening. Four possible dates to implement this solution have been studied: 1995, 2000, 2005, and 2010 (options 1 to 4). After the last of these dates, it has been considered that it is not worth significantly enhancing the bridge at a high cost, because the expected end of its service life is too near (2030). These four options have been compared with the no-action option (option 0).

It has been assumed that the capacity of the bridge in terms of maximum allowable live load is not affected by widening the deck. Therefore, the costs C_{FFL} (heavy traffic detoured) are the same for all the options. Nevertheless, if the enhancement of the bridge were to

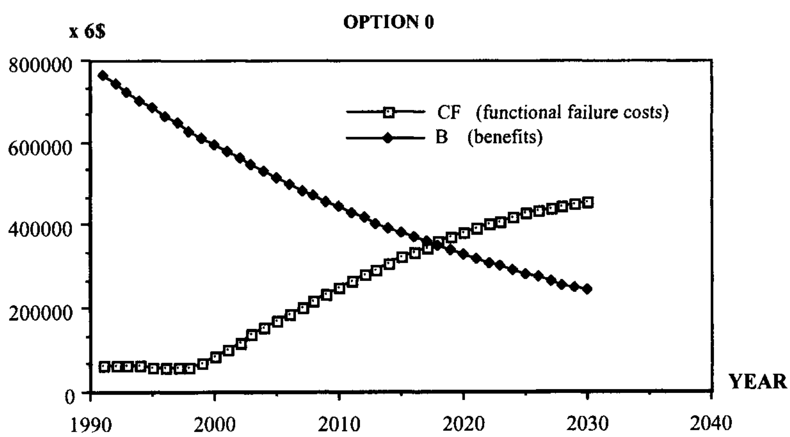


Figure 13-14. Comparison between the present value prices functional failure costs and the benefits throughout the bridge service life for the option of not widening the bridge deck (option 0)

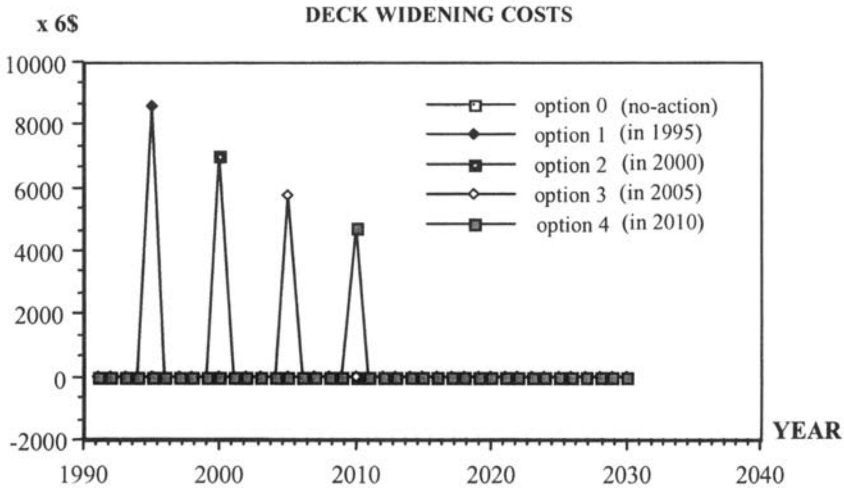


Figure 13-15. Comparison of the direct (repair) costs *CR* for each deck widening option

include an increase in the bridge's structural capacity, it would be easy to consider that in the analysis.

The structural failure costs C_{FSF} are affected by deck widening, because they depend on the functional failure costs in the period during which the bridge is being replaced in case it collapses. However, to simplify the analysis, it has been considered, that the decrease in the probability of failure compensates for the increase in the cost of reconstruction, so the structural failure costs are almost the same for all the options.

The deck widening operation takes 9 months to be performed. During that period, one lane on average is closed to traffic. The traffic evolution prediction is the same as in the previous case study. The direct costs of deck widening have been computed at present value prices of 1991 (the analysis reference year) as $60,000. As shown in Figure 13-15, these costs will be different for each option because they occur at different times. The deck widening work is analyzed, considering also two types of functional costs:

- costs due to delayed traffic C_{FFD} (during the period in which the work is being performed and, under normal circumstances, due to insufficient traffic capacity of the bridge, which is affected by the number of lanes that are open);
- costs due to light detoured traffic in terms of volume C_{FFV}.

In this analysis, the only repair costs considered are the differential costs related to option 0 (not performing the widening), equivalent to the no-action option in Equation 13-3. The benefits will be the same because the number of traffic lanes in working conditions is the same for all options.

The influence of the repair date on the traffic delayed is shown in Figure 13-16, where it can be seen that the sooner the widening is performed, the better in terms of functional costs. Peak values correspond to local traffic delay costs during the construction periods.

The influence of the repair date on the light detoured traffic is shown in Figure 13-17. Peak values relate to the fraction of the total in excess traffic detoured during the construction period.

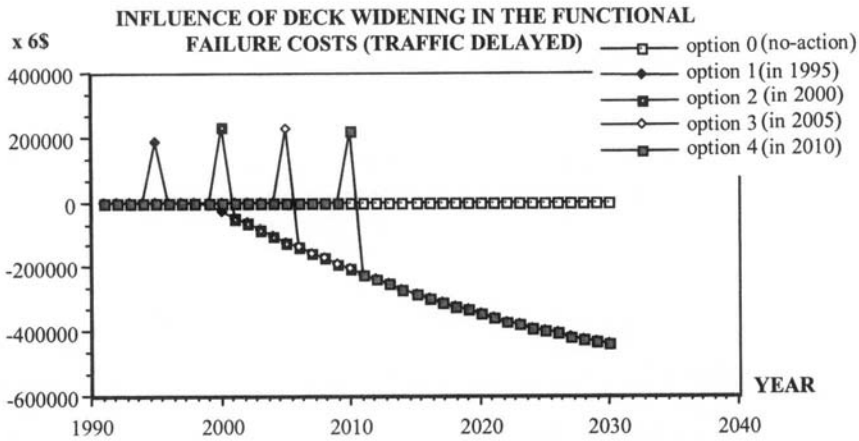


Figure 13-16. Comparison of the functional failure traffic delayed costs C_{FFFD} for each deck widening option

The main results of the economic analysis are presented in Table 13-6 (Branco and de Brito 1995), where the benefits and costs are presented in thousands of U.S. dollars, and it can be seen that:

- all the options that consider widening the deck are preferable to the no-action option;
- the best option is to widen the deck in 1995 or sometime between 1995 and 2000, because the first two options have very similar results; a more localized analysis would be necessary to reach a definite conclusion;
- deck widening will significantly increase functional service life, a situation that is particularly interesting because the end of the bridge's structural life will probably not occur in 2017.

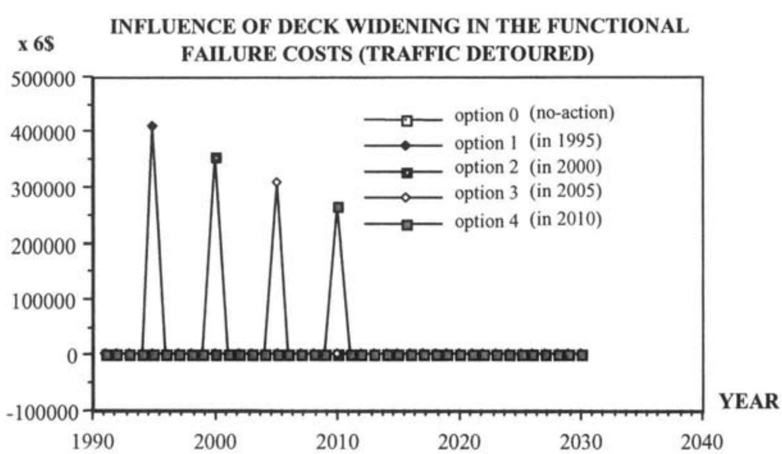


Figure 13-17. Comparison of the functional failure traffic flow detoured costs C_{FFFV} for each deck widening option

Table 13-7. Results of the sensitivity analysis (CEI values) to discount rates and traffic volumes

Option	$r - 3\%$	$r + 3\%$	$T - 15\%$	$T + 15\%$
0	1.0000	1.0000	1.0000	1.0000
1	3.1391	1.4352	1.0468	3.0319
2	3.1216	1.4412	1.0484	2.8057
3	3.0465	1.4145	1.0487	2.4868
4	2.8771	1.3540	1.0489	2.0867

As in the previous case study, a sensitivity analysis has been performed using this example with different discount rates (variation of ±3%) and traffic volume (variation of ±15%). The main results, in terms of the CEI index, are shown in Table 13-7 (de Brito and Branco 1998b), from which it can be seen that:

- the advantage of all options that consider widening the deck increases significantly with a reduction in future discount rates and vice versa; this is due to the fact that the functional failure costs, which increase with increased traffic, occur during the service life of the bridge, and the direct costs occur as soon as the work is done; deck widening options decrease failure costs, but have direct costs; therefore, the reduction in the discount rates gives a higher relative importance to the failure costs in a present value economic analysis;
- for identical reasons, an increase in the future discount rates delays the optimum date for widening the deck (option 2 becomes the best option);
- functional failure costs decrease with a reduction in traffic volume, thus leading to a tendency to under-evaluate all the action options and to delay the optimum date for action; if the real future traffic volume is 15% less, the optimum date for widening will be after the year 2005;
- on the contrary, if there is an excess of traffic, all the action options will be over-evaluated, and the deck must be widened as soon as it is feasible; in that case, an interesting situation occurs, because option 0 corresponds to a global deficit in opposition to all the other options;
- the final conclusion indicates that the deck must be widened even though the optimum date (after 1995) can be decided later, which will allow for obtaining more accurate estimates of discount rates, especially of traffic volume.

13.3.5. Reliability-Based Decision Making

As mentioned previously, the expert system developed within the Brite 3091 Project consisted of a reliability-based decision-making submodule, Bridge-2(R), that offers the user the following standard main menu (de Brito et al. 1997):

- relevant structural repair techniques;
- optimization of repair plan.

Table 13-8. Parameters used to estimate repair (or maintenance) techniques costs

General Parameters
- Defect location (on structure) factor (f_1)
- Coefficient modeling the cost from the headquarters (k_{F1})
- Cost to set up a roadblock of one lane during an 8-hour period (R_1) [$]
- Number of 8-hour periods during which a roadblock will be necessary (t_1)
- Repair time factor (f_t)

Specific Parameters
- Area to repair [m_2]
- Joint length to repair [m]
- Gutter joint length to repair [m]
- Number of deck drain extensions
- Number of diversions

After a structural assessment is performed at time T_0, a decision must be made about repairing the bridge and the time necessary to do it. For this analysis, the repair Bridge-2 submodule has expert knowledge (in terms of flowcharts [Figure 13-9 (Branco and de Brito 1995)] prepared using know-how from designers and contractors) to eliminate repair techniques that are inappropriate for the defect found. The system will ask a set of questions to determine the parameters that characterize the defect (Table 13-8) (de Brito et al. 1997), and, knowing these parameters, the possible repair methods will be pointed out. If more than one technique is considered possible, the system uses an optimization procedure with the following assumptions (Sørensen and Thoft-Christensen 1991):

- After each structural assessment, the total expected benefits minus expected repair and failure costs for the remaining life time of the bridge are maximized by considering only the repair events during the lifetime of the bridge;
- It is assumed that N_r repairs of the same type I_r are performed within the remaining lifetime of the bridge. The first repair is performed at time T_{r_1} and the remaining repairs are performed at equidistant times with a time interval $t_r = (T_l - T_{r_1})/N_r$, where T_l is the year corresponding to the expected lifetime of the bridge.

To decide which repair type (including no repair) to choose and the repair time necessary after a structural assessment, the following optimization problem is considered with optimization variables (Sørensen and Thoft-Christensen 1991):

- type of repair I_r (including no repair) to be selected;
- time T_{r_1} of the first repair [year];
- total number of repairs N_r in the remaining lifetime of the bridge.

$$\max C_T(I_R, T_{R_1}, N_R) = B(I_R, T_{R_1}, N_R) - C_R(I_R, T_{R_1}, N_R) - C_F(I_R, T_{R_1}, N_R)$$
$$\text{subject to } \beta^U(T_L, I_R, T_{R_1}, N_R) \geq \beta^{\min} \qquad (13\text{-}16)$$

where

C_T = total expected benefits minus costs in the remaining lifetime of the bridge

B = expected benefits in the same period

C_R = expected repair cost in the same period

C_F = expected failure cost in the same period

T_L = year corresponding to the expected lifetime of the bridge

β^U = updated reliability index (estimated by taking into account the structural assessment results)

$\beta^{\min}$ = minimum acceptable reliability index for the bridge (related to critical elements or to the global system)

The long-term present-value economic analysis that must be performed to make a decision should use a global cost function that includes the following: initial, inspection, maintenance, repair, and failure costs and benefits. In the analysis adopted in the prototype actually implemented, some of these items (initial, inspection and maintenance costs) are omitted for the sake of simplification, because they are not relevant to the comparative analysis between repairs and their contribution to the global costs is negligible. Also, the repair and failure costs and the benefits concerning each option are quantified with simplified formulas, based on traffic predictions prepared by the user (see Section 13.3.3.).

In order to estimate direct repair costs, they are divided into fixed costs and unit costs (Equation 13-9). The fixed direct costs include displacement costs from headquarters and roadblock costs. The direct unit costs are obtained by multiplying the repair volume by its unit cost, which depends on many factors (location, repair time, labor, material/equipment, etc.; Table 13-8) (de Brito et al. 1997).

The functional costs of the repair depend on, among other things, traffic volume, the degree of disruption caused by the work, the time to accomplish the repair, and the detour length. The estimation of the user costs must consider aspects such as time wasted by the drivers, energy costs, vehicle maintenance costs, and costs resulting from an increase in traffic accidents. They can be computed using all the traffic data usually available to the authorities, but at the present stage, the system uses coefficients, estimated for each bridge, which allow the direct estimation of the failure costs and benefits proportional to the traffic volume on the bridge (see Section 13.3.3.).

There has been considerable investigation into reliability-based structural modeling and assessment, structural service life prediction, maintenance, repair, strengthening prioritization, and cost predictions. Examples of just such proposals can be found in Cosyn (1993), Faber and Lauridsen (1996), Henriques et al. (1996), Shetty et al. (1996), Ng and Moses (1996), Cropper et al. (1999), Shetty et al. (1999), Frangopol and Estes (1999), Stewart (1999), Jensen et al. (2000), Das and Onoufriou (2000), Rubakantha and Parke (2000), Frangopol et al. (2000), Kaschner et al. (2000), Enevoldsen and Pup (2000), and Das (2000).

13.3.6. Decision-Making and Structural Assessment

There are many interesting references concerning bridge assessment from a structural point of view and the conclusions that can be drawn from it in terms of decisions concerning repair/strengthening.

For example, in Dawe (1993) an assessment program for old bridges based on a specific code is described. Bridges that passed the code but not the national standard were allowed to continue in use, but were considered substandard. A further development of that code allows the use of "worst credible strengths" and lower partial material factors to take into account the findings of the inspection, in terms of deterioration of the materials. Flint and Huband (1993) deal with the same subject but point out specific differences between assessment of existing bridges and design of new bridges.

In Jackson (1993), attention is drawn to the fact that analytical methods for assessment of the load-carrying capacity of bridges are often over-conservative and that many bridges can remain in service if consideration is given to more realistic assessment methods (such as conventional analyses with modified section properties, plastic or nonlinear analyses, or other methods).

Clark (1993) presents analytical models for assessing the effects of alkali silica reaction on element strength, which allow for their detrimental effect on concrete strength and the beneficial effect derived from induced prestress. This approach could be used in a reliability-based decision system, such as the system described in Sections 13.2.2 and 13.3.5.

In Miyamoto et al. (1993), a bridge rating expert system is presented, which is a knowledge-based expert system for structural safety assessment of concrete elements and is based on a knowledge database, an inference engine, and a concept of the fuzzy set theory. By application of fuzzy mapping, a prediction of the remaining life of existing bridges can be obtained.

Meadowcroft and Whitbread (1993) present a procedure for assessing the risk of scour at bridges based on characteristics of the bridge and river. The results are used to prioritize further action.

Nowak and Szerszen (1999) present a practical way of estimating the remaining service life of reinforced concrete beams exposed to repeated dynamic loading, thus allowing the modeling of a fatigue-controlled limit state.

Lark and Mawson (2000) propose a serviceability approach to assessment instead of the classic ultimate limit state approach, based on techniques intended for calculating, gauging, and monitoring "in-service" reliability of bridges.

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