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CHAPTER 117

ASSESSMENT OF PERFORMANCE AND OPTIMAL STRATEGIES FOR INSPECTION AND MAINTENANCE OF CONCRETE STRUCTURES USING RELIABILITY BASED EXPERT SYSTEMS[1]

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PREFACE

The present chapter is based on the "Final Technical Report" and other working reports produced within the EU supported research project P3091 (BREU Contract 0186-C) in the period from July 1, 1990, to December 31, 1993.

The project partners are:

- CSR, Aalborg, Denmark (project coordinator)
- University of Aberdeen, Aberdeen, UK / Sheffield Hallam University, Sheffield, UK
- Jahn Ingenieurbureau, Hellevoetsluis, Holland
- Instituto Superior Técnico, Lisboa, Portugal
- LABEIN, Bilbao, Spain.

INTRODUCTION

Most of the reinforced concrete bridges built in Europe in the past seventy years were designed on the basis of a general belief among engineers that the durability of the composite material could be taken for granted. Although a vast majority of reinforced concrete bridges have performed satisfactorily during their service life, numerous

[1] Technical Paper. EU project P3091. CSRconsult, Aalborg, Denmark, 2002.

instances of distress and deterioration have been observed in such structures in recent years. The causes of deterioration of reinforced concrete bridges are often related to durability problems of the composite material. One of the most important deterioration processes which may occur in reinforced concrete bridges is reinforcement corrosion, caused by chlorides present in de- icing salts and/or carbonation of the concrete cover zone.

Although the phenomenon of reinforcement corrosion is fairly understood, rational decisions about cost-effective bridge designs, optimum strategies for inspection, maintenance and repair are hampered by the absence of comprehensive data on the structural performance of deteriorated concrete elements. The research in the present project aimed to overcome this shortcoming by developing procedures for assessing the influence of reinforcement corrosion on the structural performance of reinforced concrete members. The experimental work was carried out on reinforced concrete beams and columns, which were subjected to accelerated reinforcement corrosion. Special emphasis was placed upon the evaluation of the bond strength at the steel/concrete interface. Different repair materials were examined from the viewpoint of performance under renewed corrosion attack. Structural analysis and reliability analysis techniques were applied to the results of the study, and simple models for predicting the residual strength of the corroded beams were produced. Such information was successfully incorporated in improved stochastic modelling of the deterioration to formulate optimal strategies for inspection and maintenance of deteriorated reinforced concrete bridges using a reliability-based expert system.
In general, the work in the present research project was divided into four subprojects.

In *Subproject 1*, all the relevant data from literature were collected, reviewed and analysed.

The stochastic modeling of deterioration was dealt with in *Subproject 2*. Experiments were carried out in order to estimate the residual flexural strength of corrosion-damaged beams and to evaluate the effect of reinforcement corrosion on the bond strength between reinforcement and concrete. Further experimental work was carried out to investigate the structural performance of corrosion-damaged beams after repair, and to investigate the performance of defective and repaired reinforced concrete columns. Further updating and reliability models for the structural performance of the reinforced concrete structural elements were developed on basis of existing software.

Optimal strategies for inspection and maintenance of reinforced concrete bridges were developed in *Subproject 3*. Existing inspection and maintenance techniques were reviewed and a correlation matrix between diagnosis methods and defects was defined. Inspection and maintenance strategies in several European countries were studied with the purpose of formulating reliability-based optimal strategies to be used in the project. In that respect, an inspection strategy submodule was defined and implemented. That includes three categories: current inspections, detailed inspections, and structural assessments. The decision system to be used in planning of maintenance and repair is divided into two subsystems: the maintenance subsystem and the repair subsystem. The expert systems were successfully applied on several bridges.

In *Subproject 4*, it was decided to divide the expert system in two modules, namely BRIDGE1 and BRIDGE2. BRIDGE1 is to be used by the bridge inspector during in-situ inspection of the bridge. BRIDGE2 is to be used in the office to analyse the data gathered from inspection and to perform reliability analysis of the bridge. During inspection, the expert system is designed to supply information to the bridge inspector on the causes of observed defects and on the appropriate diagnosis methods and related defects. Further, the inspector is asked to record the inspection results so that they can be used later in the assessment of reliability of the bridge and in the decision on whether a detailed structural assessment is needed. A detailed analysis of the state of the bridge is carried out in the office after each inspection. The output of the analysis includes estimation of the reliability of the bridge and recommendations, 'decisions', on whether a structural assessment is needed or immediate repair should take place. Recommendations on the relevant repair procedures and the time for repair are also given by the expert system. Expert knowledge is used extensively to improve the quality of the decisions.

According to the original technical annex the main objective of the project was to optimize strategies for inspection, maintenance and repair of reinforced and prestressed concrete structures by developing improved methods for monitoring and modeling the deterioration of existing as well as future structures using reliability based methods and expert systems.

At the beginning of the project it was decided that only reinforced concrete bridges damaged by reinforcement corrosion were to be investigated. However, the architecture of the final expert system was so designed that it would be possible to include other types of concrete structures and other mechanisms of deterioration.

The methodology used in each Subproject is described below.

Subproject 1: Review of Data and Models for Deterioration of Concrete

The first objective of this Subproject was to define deterministic as well as stochastic data and models on concrete performance. Data on concrete deterioration was gathered from research publications, and a preliminary selection of a deterioration model for reinforcement corrosion was made. According to this model., corrosion of steel reinforcement in concrete occurs in three successive stages, namely: initiation stage, propagation stage and final stage. For each of the three stages mathematical models were formulated. Data needed for the following FORTRAN modules was also identified: The structural analysis module, the updating analysis module, the reliability module, and the inspection module.

The second objective of this subproject was to analyze the data statistically. A purpose-made FORTRAN module for statistical analysis of data with missing information, called MISDAT, was developed and tested.

The third objective of this subproject was to identify the laboratory work which was needed in order to obtain relationships between the different parameters in the study of reinforcement corrosion, namely, the characteristics of the concrete cover zone, the chloride content and chloride and carbon dioxide diffusion coefficients in the concrete cover zone, and the thickness of the concrete cover zone.

Subproject 2: Stochastic Modeling of Deterioration of Concrete Structural Elements

The first objective of this Subproject was to establish a database containing all possible defects in concrete bridges and to define the correlation matrix between these defects and their causes. Likewise, a database of relevant repair techniques was established and a correlation matrix between corrosion-related defects and repair techniques was defined.

The second objective of this subproject was to design and execute an experimental programme to investigate the effect of reinforcement corrosion on the structural performance of concrete elements. That included the study of the residual flexural strength of corrosion- damaged concrete beams, the effect of reinforcement corrosion on bond strength between reinforcement and concrete, the residual strength of concrete columns with small corrosion- induced damages and the performance of corroded beams and corroded columns after repair. Further, experiments were performed to obtain the basic characteristics of concrete and the materials used in repair, such as compressive strength, elastic modulus, compressive creep, drying shrinkage and swelling deformations.

The third objective of this subproject was to develop stochastic models for deterioration of reinforced concrete structural elements and also for inspection, maintenance and repair. A FORTRAN program BETAEQ was developed so that these stochastic models can be updated when new information is available. A commercial software package RELIAB was modified and interfaced to the expert system modules.

Subproject 3: Optimal Strategies for Inspection and Maintenance of Concrete Structures

The first objective of this Subproject was to develop optimal strategies for inspection based on the available inspection and maintenance strategies. Inspection and maintenance strategies from France, Canada, USA, UK, Switzerland, Denmark, Sweden and Finland were studied, and it was decided to divide inspections in three categories: current inspections, detailed inspections and structural assessments.

The second objective of this subproject was concerned with the decision system used in planning the maintenance and repair activities. The decision produced by this system is basically an economic decision based on the total expected benefits minus costs in the remaining lifetime of the bridge. It was decided to divide the decision system into two subsystems: a maintenance subsystem, which deals with maintenance repair techniques and small repair, and a repair subsystem to be used in choosing the best option of structural repair. A FORTRAN program called INSPEC was developed to be used in the estimation of the optimal repair time and the number of repairs for a given repair method.

Finally, the expert system was tested on four bridges, two in Portugal and two in Denmark. The Bridges Department of the Grampian Regional Council in the U.K. also tested the expert system on some bridges in the Grampian region and forwarded valuable proposals for improvement.

Subproject 4: Expert System for Identification of Deterioration of Concrete Structures

The first objective of this Subproject was to investigate the market of available expert system shells to select one, which could satisfy the requirements of the present project. An important requirement was that the expert system should be able to call a number of FORTRAN programs such as RELIAB and INSPEC and several databases. It was concluded that a particular tool, ADS/PC, would satisfy all these requirements The choice was also due to the great variety of representational schemes as well as inference mechanisms that the software tool has.

As described earlier, it was decided that the expert system should be divided in two modules, BRIDGE 1 and BRIDGE2. The first prototype of BRIDGE 1 was presented as early as June 1991. It was based primarily on the correlation matrices between diagnosis methods and defects. Special emphasis was placed upon the definition of the databases, since the communication between BRIDGE 1 and BRIDGE2 works through these databases. A number of prototypes of BRIDGE2 were implemented and discussed between the partners. This process was very difficult but finally the specifications were completed. Expert knowledge was included based on interview of several experienced bridge engineers from several countries.

1. REVIEW OF DATA AND MODELS FOR DETERIORATION OF CONCRETE

The development of expert systems for structural assessment and for determining inspection and maintenance routines for reinforced concrete structures requires information on the basic mechanisms of deterioration in both concrete and reinforcement. Physical models to define the deterioration processes, and theoretical/empirical expressions which can be used to assess long-term performance, are also needed. Data are also required on the structural performance of reinforced concrete elements after they have suffered different degrees of deterioration and after repair has been carried out.

Selection of Structure and Deterioration Process - Owing to the vast scope of the research field, only one type of structure and one deterioration process were selected for detailed analysis. Reinforced concrete bridges formed a suitable choice of structure owing to the large number of such structures, which were built after the war in the EEC countries. The deterioration process of reinforcement corrosion was selected for the present investigation since reinforcement corrosion may lead to serious consequences regarding the serviceability and safety of the concrete bridges, such as cracking and spalling of the concrete cover zone and the reduction in the load-carrying capacity of the concrete bridge. It was decided that only two mechanisms of corrosion would be investigated; reinforcement corrosion caused by chlorides present in de-icing salts and/or corrosion due to carbonation of the concrete cover zone.

Nature of the Problem - Corrosion of steel in concrete is considered to be one of the most serious problems that can lead to concrete distress. The protection of the steel reinforcement essentially depends upon the passivity provided by the alkalinity of the concrete environment. Gamma oxide film (Fe_2O_3), which rapidly forms on the steel surface in the presence of moisture and oxygen and water-soluble alkaline products formed during the hydration of cement provide the electrochemical protection needed against corrosion. Corrosion occurs when the environment closest to reinforcement has been changed to such an extent that the passive state has been counteracted. Corrosion-induced deterioration of a concrete structure usually becomes manifested when the concrete cover zone cracks, and eventually falls. This disruption occurs as a result of the bursting forces exerted by the accumulated corrosion products, which occupy larger volume than the steel.

1.1 Definition of Data Necessary for Subprojects 3 & 4

The two main causes of reinforcement corrosion in concrete were considered, namely corrosion induced by carbonation of the concrete cover zone and corrosion due to chloride attack on the reinforcement. The important environmental parameters (e.g. temperature, humidity, wet/dry cycles etc.) and material characteristics (e.g. w/c ratio, cement content, pH, CO_2 diffusion coefficient etc.), which govern carbonation, were identified. Similarly, the main parameters which control chloride induced corrosion were identified - e.g. chloride concentration threshold, exposure conditions such as marine environment, deicing salts on bridge decks etc., concrete mix proportion parameters such as w/c ratio, the diffusion coefficients for chloride ions and oxygen and moisture into concrete.

Corrosion Initiation Stage - An accurate determination of the initiation period requires extensive data on diffusion coefficients of CO_2 and chloride in concrete with different constituents and different mix proportions under various environmental conditions. Therefore, it was decided to develop general procedures of analysis of the corrosion initiation stage and rely on available data to determine relatively inaccurate assessments of the initiation period. Accuracy, however, will improve as more field data become available with time. Alternatively, it was decided to explore the possibility of the derivation of simpler empirical expressions based on laboratory data and on surveys of reinforced concrete bridge structures in the U.K., the Netherlands and Spain.

Corrosion Propagation Stage - The progress of corrosion after its initiation relies on the availability of oxygen and moisture. The nature of the concrete matrix, in addition to the exposure conditions, will govern the availability of these elements. Very little is known on the rates of reinforcement corrosion in concrete. Therefore, data available from field studies, such as those carried out under Concrete in the Oceans Programme in the U.K., was used to obtain relatively approximate values for reinforcement corrosion rates. These values of corrosion rate were to be used in the expressions proposed to determine the duration of the propagation stage of corrosion.

Corrosion Final Stage - On the basis of models/expressions derived for corrosion prediction, the deterioration in structural performance with time relationship was identified, together with the definition of the following stages:

- Degree of deterioration allowed before small or large repair is required,
- Degree of deterioration when structure is considered to have failed,
- The efficiency of the structure immediately after repairs have been carried out and the new structural performance-time relationship.

Data Gathered at Inspection - Data to be gathered during routine and major inspections of any bridge structure were carefully defined and classified. The incorporation of each group of these data in the future assessment of the strength of the structure was explained. For example, routine inspection could involve visual inspection of cracks, rust stains, excessive deflections etc. which could indicate if the structure is likely to be in distress and needs major inspection with the relevant concrete sampling. Similarly, careful interpretation of the data obtained from corrosion monitoring would assist in the process of prediction of the extent of corrosion and the rate of corrosion propagation. The updating of the analysis would be carried out after each inspection period so as to provide new data to be used for performance prediction.

Performance of Repaired Elements - It was decided that information would be required on the effect of different types of repair and of different repair materials on the long-term performance of the structural elements, so that their performance-time relationship could be established. This would allow the prediction, from the expert systems, of the new inspection, maintenance and repair routines of bridge structures after they were repaired and would also - allow the maximum life prediction of the structure.

Expert System - It was agreed that data are needed for the following FORTRAN modules:

- Structural Analysis Module.
- Updating Analysis Module.
- Reliability Analysis Module.
- Inspection Programme Module.

A *structural analysis program* (e.g. Finite Element program) was not developed in the project, but it was suggested that the effect of the structural system is modelled by a response surface. It was recommended that a limited number of standard bridges were selected. For each of these bridges, all the geometrical quantities and all parameters defining the cross-sections were defined.

In the *reliability analysis module,* two groups of data were identified: data for loads and data to be used in resistance models. Data for loads are divided in two categories, namely dead loads and live loads. The dead loads include weight of structural elements, weight of rails and weight of pavement. For each of these loads the following estimates were determined: expected value, standard deviation, distribution type and correlations. The live loads mainly are traffic loads. Wind, snow, temperature, and earthquake load are not taken into account. For the traffic load a probabilistic model is proposed based on the EUROCODE. Three failure models are considered, namely bending failure of beams and slabs, shear failure of beams and slabs, slab yielding and stability failure of columns. Probabilistic description of the resistance data and the corrosion processes is needed. For quantities modelled by stochastic variables used to describe the resistance and the corrosion process the following estimates are needed: expected value, standard deviation, distribution type and correlation with other stochastic variables.

In the *updating analysis module* two types of updating are foreseen, namely updating of probabilistic models for stochastic variables and updating based on observed events. Examples of stochastic variables for which the probabilistic models can be updated are: traffic load variables and yield strengths of concrete and/or reinforcement. Data needed for updating stochastic variables are future data. Observed events that can be taken into account are: the structure (or structural element) has not failed, inspection results and repair events. Only future data is needed. It should be possible to extract the necessary data from the database.

In the *inspection program module* data for the costs and data for the stochastic models modelling inspection and repair is needed. Data for the following costs are needed in the cost modelling: initial cost, inspection costs, repair costs and benefits. For the real rate of interest data of forecast models is needed. For the inspection techniques data to model a probabilistic description of the probability of detecting a defect for a given inspection effort should be given. Further, data to model the measurement uncertainty is needed for each inspection method. For each type of repair data to model the behaviour after repair is needed.

1.2 Review of Data

Considerable amount of information on corrosion rate, depth of carbonation and chloride diffusion were gathered from almost 25 highway bridges built in the Grampian region, Scotland. This information, together with data on bridges built in Spain, data on bridges built in the Netherlands and extensive laboratory data from literature were used to derive quantitative expressions and to illustrate relationships between the different parameters of the study of reinforcement corrosion in concrete. Actual values were used as long as it is possible; otherwise values for cement content and water/cement ratio were calculated according to British standard mix design procedures.

For plain concrete of moderate strength ($f_{cu} \approx$ 30 N/mm2) values of the chloride diffusion coefficient, D_C, in the range between 1×10^{-8} and 5×10^{-8} cm2/sec were reported, and values of the carbonation constant, K, in the range between 1 and 8 were suggested. Trends of decreasing chloride concentration in expressed pore fluid with increasing water/cement ratio were illustrated. The diffusivity of concrete to gases was shown to be influenced by the cement content as a result of the overall enhancement of the concrete strength with higher cement contents. The influence of cement replacement materials on concrete diffusivity and the subsequent corrosion risk was also discussed. The effect of each environmental parameter on the overall corrosion process was illustrated, such as the increase in the corrosion rate under conditions of concrete partially saturated with water. Data on the different inspection methodologies and their related errors were reviewed. That includes various destructive and non-destructive tests. Different repair materials and techniques were listed. Evaluation of the cost associated with each type of repair was made.

1.3 Statistical Analysis

In experiments, not all of the quantities in the model are necessarily observed. Therefore the data to be analyzed will in general include so-called missing values. Statistical software packages typically exclude experiments with missing values for any of the variables involved in the statistical analysis. This strategy is generally inappropriate, since the investigator is usually interested in making inferences about all the experiments made, and cases with missing values have some although not perfect information related to them.

Several methods to perform analyses of data with missing values were reviewed. The methods recommended estimating values of the missing parameters were the EM algorithm and Buck's method. The possibility of using the statistical software package SYSTAT in this task was examined and was found not to be very suitable for analysis of data with missing values. Therefore, a FORTRAN statistical program MISDAT was developed. MISDAT was used to estimate missing values in a data matrix according to the EM algorithm or Buck's method. (Buck's method can be used to estimate missing values when the amount of missing data is small and the missing data are missing completely at random, whereas the EM algorithm can provide consistent estimates when the missing data are missing at random) MISDAT has also the ability to perform linear regression analysis on the estimated complete data matrix by using a sweep operator. The output of MISDAT contains the estimated complete data matrix, expected values, covariance matrix and correlation matrix of the linear regression coefficients, regression coefficient and adjusted squared regression coefficient.

MISDAT was used to analyze data with missing values gathered during the course of the project. The conclusions of the analyses were as follows:

- Buck's method and the EM algorithm gave similar but different estimates of the missing values in the data matrix.
- The linear modelling considered in MISDAT was not satisfactory.
- More data were needed to make any conclusions of the statistical analyses.

1.4. Derivation of Models for Deterioration

Corrosion Model - A preliminary corrosion model was introduced. According to that model corrosion of steel reinforcement in concrete structures occurs in three successive stages, namely: initiation stage, propagation stage and final stage.

Corrosion Initiation Stage: Corrosion initiation period refers to the time during which the passivation of steel is destroyed and the reinforcement starts to corrode actively. Practical experience shows that the initiation stage is completely dominated by the carbonation of the concrete cover zone, and the excessively high chloride content around the embedded steel.

Corrosion Propagation Stage: Corrosion propagation period refers to the time, which follows corrosion initiation until failure occurs. The rate at which corrosion proceeds during the propagation period is believed to be governed mainly by the concrete characteristics and dimensions, and the exposure conditions.

Corrosion Final Stage: The concrete structure is considered to be within the final stage of corrosion when a certain amount of damage is believed to be inflicted upon it, however no universal criteria is available yet to define the state of failure.

Corrosion Initiation Stage due to Concrete Carbonation - Carbonation of concrete is caused mainly by the reaction of CO_2 in the atmosphere with $Ca(OH)_2$ of the cement hydration products in the presence of water. The result is loss in alkalinity in the concrete cover, approaching pH values of neutrality, and the passivation of the reinforcement is no longer maintained. The penetration of CO_2 into concrete pores tends to move as a front, which proceeds at a rate controlled mainly by the CO_2 diffusion coefficient. The depth of carbonation can be determined as follows:

$$d = K \cdot \sqrt{t}$$

where d is the carbonation depth in millimeters, t is the time elapsed in years and K is the carbonation constant.

Corrosion Initiation Stage due to Chloride Attack - It is widely reported that the presence of chloride ion in the vicinity of the steel reinforcement tends to depassivate the steel and promotes the metal corrosion, although the phenomenon has not yet been fully understood. Fick's law of diffusion can reasonably represent the rate of chloride penetration into concrete, as a function of depth from the concrete surface and time, as follows:

$$\frac{\delta c}{\delta t} = D_C \frac{\delta^2 c}{\delta x^2}$$

where c is the chloride ion concentration, as % of the weight of cement, at distance x cm from the concrete surface after t seconds of exposure to chloride source. D_C is the chloride diffusion coefficient expressed in cm^2/sec. The solution of that differential equation is as follows:

$$C(x,t) = C_0 \left[1 - \operatorname{erf}\left(\frac{x}{2\sqrt{D_C \cdot t}} \right) \right]$$

where C_0 is the equilibrium chloride concentration on the concrete surface, as % of the weight of cement, x is the distance from the concrete surface in cm, t is the time in sec, erf is the error function, D_C is the diffusion coefficient in cm^2/sec and $C(x,t)$ is the chloride concentration at any position x at time t. In a real structure, if $C(x,t)$ is assumed to be the chloride corrosion threshold and x is the thickness of concrete cover, then the corrosion initiation period, t, can be calculated based on a knowledge of the parameters C_0 and D_C. The following table suggests values for C_O, as % of cement weight, for different types of reinforced concrete structures under different types of exposure.

Type of Structure	Exposure Condition	C_o
Offshore platform	Air zone	1.6

Offshore platform	Splash zone	2.4
Bridge deck	De-icing salt	1.6
Bridge column	De-icing salt	5.0
Jetty	Upper tidal zone	6.1

For plain concrete of moderate strength ($f_{cu} \approx 30$ N/mm^2) reported values of D_C are in the range between 1×10^{-8} and 5×10^{-8} cm^2/sec. Based on vast experimental results on chloride diffusion into concrete samples, it was concluded that D_C is time dependent. The relationship between chloride diffusion coefficient, D_C , and time, t, could be approximated by the following empirical equation:-

$$D_C = D_1\sqrt{t}$$

where D_C is the chloride diffusion coefficient (cm^2/sec), t is the elapsed time (sec) and D_1 is a coefficient (cm^2/sec) that represents a D_C value at time equal to one second. A modified diffusion equation taking into account the time dependence nature of D_C has been produced as follows:

$$C_{(x,t)} = C_o\,\{1 - erf\,\frac{x}{2\sqrt{2D_1}\sqrt[4]{t}}\}$$

where C_O is the equilibrium chloride concentration on the concrete surface, as % of the weight of cement, x is the distance from the concrete surface in cm, t is the time in sec, erf is the error function, D_C is the diffusion coefficient in cm2/sec and $C(x,t)$ is the chloride concentration at any position x at time t.

Experimental Work - The diffusion coefficient of the chloride ion was experimentally obtained for samples of concrete and of cement pastes. Samples with different w/c ratio (0.4, 0.5 & 0.6) were tested. The concrete samples (350 kg/m^3) were cured for one week in a saturated humidity chamber, whereas the pastes were cured for two months in a saturated chamber or in a $Ca(OH)_2$ solution. After cure all the samples were exposed to 3% NaCl solution for chloride penetration determinations. Results obtained for chloride ion diffusion showed that the diffusion coefficient in concrete samples linearly increases with the w/c ratio being constant with time. Values obtained for D_C were between $1\cdot10^{-9}$ and $1\cdot10^{-8}$ cm^2/s. These results also showed a linear correlation between w/c ratio and chloride diffusion coefficient as follows:

$$D_C\ (\text{cm}^2/\text{s}) = -2.0\times10^{-9} + 7.5\times10^{-9}(\frac{w}{c})$$

In another set of experiments the carbonation depth on a concrete sample having a w/c ratio of 0.6 and exposed to a controlled environment (CO_2 = 5% by volume of air) was determined. Results showed an increase of carbonation depth with time and a linear relationship between the carbonated thickness x and the time t square root:

$$x\ (\text{cm}) = 0.002 + 0.0013\sqrt{t} \qquad \text{(time in seconds)}$$

1.5. Main Results and Conclusions

Owing to the vast scope of the research field, only one type of structure and one deterioration process were selected for detailed analysis. Reinforced concrete bridges formed a suitable choice of structure owing to the large number of such structures, which were built after the war in the EEC countries. The deterioration process of reinforcement corrosion was selected for the present investigation since reinforcement corrosion may lead to serious consequences regarding the serviceability and safety of the concrete bridges, such as cracking and spalling of the concrete cover zone and the reduction in the load-carrying capacity of the concrete bridge. It was decided that only two mechanisms of corrosion would be investigated; reinforcement corrosion caused by chlorides present in de-icing salts and/or corrosion due to carbonation of the concrete cover zone. Data on corrosion-induced deterioration in reinforced concrete bridges was gathered from research publications and field inspections, and a preliminary selection of a deterioration model for reinforcement corrosion was made. According to this model, corrosion of steel reinforcement in concrete occurs in three successive stages, namely: initiation stage, propagation stage and final stage. For each of the three stages mathematical models were formulated. Data needed for the following FORTRAN modules was also identified: The structural analysis module, the updating analysis module, the reliability module, and the inspection module. All the data were analyzed statistically, and a purpose-made FORTRAN module for statistical analysis of data with missing information, called MISDAT, was developed and tested. Empirical relationships between the different parameters in the study of reinforcement corrosion, namely, the characteristics of the concrete cover zone, the chloride content and chloride and carbon dioxide diffusion coefficients in the concrete cover zone, and the thickness of the concrete cover zone were obtained.

2. STOCHASTIC MODELLING OF DETERIORATION OF CONCRETE STRUCTURAL ELEMENTS

The following were the objectives of this subproject:

1- to review of the currently available criteria of corrosion-induced deterioration levels of reinforced concrete structures in which, the different deterioration features and mechanisms are identified, and performance-over-time functions are determined.
2- to define a database containing correlations between damages verified in bridge structural elements and their possible causes.
3- to investigate the mechanical behaviour and the failure mechanism of corrosion damaged reinforced concrete beams to clarify how the different factors contribute to the action of failure once corrosion has started. Such information is of great importance in design inspection and maintenance strategies for reinforced concrete structures where corrosion is considered to be a potential problem.
4- to study load-strain characteristics of short columns with spalled concrete cover and also of repaired one.
5- to develop a data file in repair techniques to be implemented in the expert system; and to complement the existing results with experimental results.
6- to develop stochastic models for deterioration of reinforced concrete structural elements. Stochastic models for inspection, maintenance and repair will also be considered.

2.1. Estimation of the Deterioration of Concrete Structural Elements

Corrosion-induced distress in concrete is visually evident by three general features; cracking, spalling and decomposition. It is possible for these three effects to be present simultaneously or to develop in succession. They may also be present in each zone of deterioration in varying and different degrees of development. The following is a review of the currently available criteria of corrosion-induced deterioration levels of reinforced concrete structures in which, the different deterioration features and mechanisms are identified, and performance-over-time functions are proposed. In this task a database containing correlations between damages verified in bridge structural elements and their possible causes is defined.

Behaviour of Deteriorated Components: The concrete structure is considered to be within the final stage of corrosion when a certain amount of damage is inflicted upon it. Since very little research has been carried out to assess the behaviour of deteriorated reinforced concrete structures, the stress analysis of such structures may result in the strength being very roughly estimated or, in some cases, completely ignored. This often produces a repair work, which is unnecessarily strong and uneconomical. If a proportion of the original member strength could be allocated to the defective member a more accurate assessment of the required repair could be made. Therefore, the need for an agreement on the parameters capable of defining the degree of deterioration of any reinforced concrete structure is obvious.

When general corrosion has taken place, it is possible to classify component failure in to two classes as follow:

The first class of failure:

The first class of failure occurs when either the steel or the concrete is degraded by corrosion or cracking to a point where the materials can no longer support the stresses imposed on them. This class of failure is usually a serviceability rather than safety concern, since visual signs of corrosion can be expected before the strength of materials become critical. As cracking proceeds, the component will become less stiff, leading in turn to continued deflection in beams, or bowing in slender columns.

The second class of failure:

The second class of failure occurs when disruption due to cracking alters the geometry of the section significantly. It is not always easy to assess the second class of failure, since relatively little corrosion cracking may be required to alter the component's behaviour radically.

Performance over Time Functions: several workers defining the deterioration levels of reinforced concrete structures due to steel corrosion have put a number of proposals and studies forward. One of the proposals suggests 10 to 25% reduction in steel bar section as an end of the residual life of the structure. Another proposal defines levels of deterioration based on the loss in safety margin in concrete members due to section loss. Some workers are defining levels of deterioration based on visual indications, such as rust stains. However, colour modifications are not always present and cannot be considered as prequisite for damage classification. In a different way, several authors consider the presence of longitudinal cracking in concrete as a sign of ultimate state of the structure. However, flexural testing of members in this condition gave results with small yield strength reduction and small loss in ultimate flexural strength. However, as corrosion of reinforcement has already been initiated, the results obtained from the experimental work in the present project suggest the deterioration process starts to proceed slowly in the early stages of the corrosion propagation period. As cracking, spalling and the loss in the bond strength start to take place a sharp increase in the deterioration process will occur. This behaviour can be described by the following expression:

$$B = 1 - \sin^2(90\ t/T)$$

where B is a deterioration indicator defining the residual capacity of a deteriorated structure as a percentage of the design capacity, t is the actual time elapsed and T is the life span less the corrosion initiation period.

Defects in concrete structures have been classified with geographical / functional criteria: the foundations / abutments / embankments are referred in one chapter, the joints in another, the bearings in yet another, etc. The need for a special chapter on global deficiencies in the behaviour of the superstructure was felt and it was solved with the introduction of chapter “A-A. Superstructure Global Behaviour”, right at the beginning. Chapters *“A-C. Concrete Elements”* and *“A-D. Reinforcement / Strands”* have a very wide scope: they are used to classify each defect, respectively in concrete and in conventional or prestressing steel, regardless of where they may occur. Thus, repeating such defects for specific elements that are covered in other chapters (sidewalks, foundations, edge beams, etc.) is avoided.

The causes of defects classification was developed, following a chronological criteria: design errors preceded construction errors which themselves proceeded in service accidental / environmental actions as well as aggressive factors.

Within the project it was decided to analyse only corrosion related defects. To correlate defects related to corrosion in concrete bridges and their causes it was felt necessary to first create a systematic classification of both defects and causes. In order to clarify the use of this correlation lest, it was decided that the possible causes were to be divided into near causes and primary causes. The near causes are the ones that immediately preceded the coming in sight of the defect. The primary causes can be quite distant from the defect and their causability is sometimes very indirect.

It was felt then, that a correlation matrix was needed in order to simplify the data that is to be fed into the computer database. A pseudo-quantitative classification of the type no correlation, low and high correlation was proposed.

The matrix was organized so that each line represents a defect and each column a possible cause. In the intersection of each line and column a number representing the correlation between defect and possible cause is to be introduced. The criteria adopted for that number is:

0 *No correlation* - no correlation whatever (direct or indirect) between the defect and the cause.

1 *Low correlation* - indirect cause of the defect connected only with the very 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; when the cause occurs, it is one of the main causes of the deterioration process and is indispensable to its development.

After corrosion of reinforcement has already initiated, it is suggested that the performance of the structure concerned may be described by the following empirical model; which is based on an experimental study of this subproject.

$$B\% = 100 - 1600\frac{R \cdot T}{D}$$

where

$B\%$ is the percentage residual strength,
R is the corrosion rate (mm/year),
T is the time elapsed (years), and

D is the rebar diameter (mm).

According to this model, the beam would have 20% residual strength when reinforcement corroded by 10%.

A correlation matrix between corrosion related defects and defects was developed and implemented in the system to help to identify associated corrosion related defects and to report the causes in the provisional defect report.

2.2. Determination of Residual Strength and Deformation

Corrosion of the embedded steel reinforcement is believed to affect the strength of concrete beams in two ways. Firstly, by reducing the rebar cross sectional area leading to premature steel yielding. Secondly, by changing the conditions at the steel / concrete interface, mainly loss of surface roughness with development of flaky layer of corrosion products and reduction in the degree of bar confinement caused by an opening of longitudinal cracks along the reinforcement, leading to severe deterioration of bond strength between concrete and the reinforcing steel with subsequent inadequate transmission of stresses between the two materials. The immediate objective of this experimental programme was to investigate the mechanical behaviour and the failure mechanism of corrosion damaged reinforced concrete beams to clarify how the different factors contribute to the action of failure once corrosion has started. The work was also concentrated on load-strain characteristics between the short columns affected by reinforcement corrosion and subsequent spalling of concrete and repaired one. Such information is of great importance in design inspection and maintenance strategies for reinforced concrete structures where corrosion is considered to be a potential problem.

The experimental programme comprised four phases:

1. Determination of the residual flexural strength of corrosion damaged beams,
2. Evaluation of the effect of reinforcement corrosion on the bond strength between the two materials,
3. Investigation of the corrosion damaged beams after repair, and
4. Performance of defective and repaired reinforced concrete columns.

Phase 1. Determination of the Residual Flexural Strength of Corrosion Damaged Beams: A total of 87 beam specimens made with ordinary Portland cement were subjected to accelerated corrosion damage and tested in flexure. The levels of corrosion were achieved by impressing direct current of different intensities on the reinforcing bars. After the application of corrosion, the specimens were then tested as simply supported beams in a two point loading machine to determine the ultimate flexural strength and the maximum deflection at failure. The investigation variables were: the degree of corrosion, the rate of corrosion and the age of the concrete.

The results indicated clear trend of decreasing residual strength with increasing corrosion degree. For the same degree of corrosion, the corrosion-induced damage appeared to be more pronounced with high corrosion rates. No evidence could be found to suggest any significant effect of concrete age on the overall rate of corrosion-induced concrete deterioration. Based on the experimental results, as corrosion of reinforcement has already initiated, it is suggested that the following empirical model may describe the performance of the structure concerned:

$$B\% = 100 - 1600\frac{R \cdot T}{D}$$

where

$B\%$ is the percentage residual strength,

R is the corrosion rate (mm/year),
T is the time elapsed (years), and
D is the rebar diameter (mm).

Phase 2. Effect of Reinforcement Corrosion on Bond Strength: A total of 17 beam specimens were employed in the investigation. Bond behaviour was studied at different levels of reinforcement corrosion. These levels of corrosion were achieved by impressing direct current of different intensities on the reinforcing bars. After the application of corrosion, the specimens were then tested as simply supported beams in a two point loading machine. Four linear Variable Displacement Transducers (LVDT) were employed to reflect the free-end slip at both ends of each corresponding load was constantly recorded.

The results provided evidence to suggest some increase in bond strength as a result of small amounts of rebar corrosion (less than 0.4%). However, beyond 0.4% corrosion a trend of decreasing bond strength with increasing corrosion level was clearly indicated, although no quantitative relationship could be provided to describe that trend.

Phase 3. Performance of Corroded Beams After Repair: A total of 78 beam specimens made with ordinary Portland cement were subjected to accelerated corrosion damage and tested in flexure. The beams were tested in a two point loading machine to determine the ultimate flexural strength and the maximum deflection at failure. The investigation variables were the repair material characteristics and the degree of corrosion before and after the application of repair work.

Three different repair materials were employed in the investigation; Concrete, Repair material M1 (which is a cement based repair mortar, supplied by JAHN, Netherlands, as a ready to use powder to be mixed with water), and M2 (which is a high performance non-shrink repair material, supplied by FOSROC, UK, as a blend of Portland cement, graded aggregates, and additives to control expansion on both the plastic and hardened states).

Phase 4. Performance of Defective and Repaired Reinforced Concrete Columns: All the column specimens used in this phase were reinforced longitudinally with four 12 mm diameter high yield steel bars, the centroid of each bar was located at 26 mm from adjacent faces of the specimens. Mild steel bars of 6 mm diameter were used as link.

It is well known that the product of steel corrosion has a volume several times greater than the original rebar, causing concrete to spall and delaminate. To represent this real situation, a short reinforced column with:

(a) a single void (with two 12 mm longitudinal bars), and
(b) two opposite side voids (with all four 12 mm longitudinal bars);

were cast with about 10% loss of cross-section due to corrosion, centrally positioned with a length of 200 mm.

The voided sections, therefore, simulate the spalling of concrete due to local reinforced corrosion. The experimental work looked at the strain distribution across a voided and repaired short columns when loaded to failure. The failure modes were also carefully studied, and together the results give the required knowledge for structural assessment.

Three different repair materials were employed in this investigation; Material M1 and M2 (as described in phase 3), and M3 (single component cementitious mortar which incorporates most advanced cement chemistry, microsilica, fiber and styrene acrylic copolymer technology).

All the specimens were cast in horizontal position in three layers, each layer being adequately compacted. The fresh concrete was covered with a polythene sheet. After 24 hours the specimens were removed from their moulds and allowed to cure in water at 20°C for 14 days, then, columns with void were repaired. Subsequently cured in water for 14 days prior to test.

The results clearly indicated that the degree of initial corrosion damage significantly affects the immediate gain in the load carrying capacity of the repaired beams and columns. Furthermore, beams repaired with different repair materials exhibited different performances under renewed corrosion attacks.

On the light of these findings it may be concluded that, to differentiate between the repair materials only on the basis of the immediate gain in the load carrying capacity of the repaired element is misleading. A sound repair strategy requires the provision of sufficient information on both the short-term and the long-term performance of the repaired elements under the expected service conditions and its ability to minimize corrosion rates should corrosion become active.

Further**,** the presence of a void in the reinforced concrete column due to corrosion (10% loss of cross section) of reinforcement and subsequent spalling reduces strength for:

(a) single side void by 46%, and
(b) double sided void approximately by 79% compare to the control column.

Repair of the double voids caused by spalling using repair material M1, M2 and M3 restore the strength of column by 72, 94 and 72 percent respectively. Also load-strain relationship for each repaired column highly depends on the Young's Modulus, E, and compressive strength of the repair materials.

2.3. Development of Data file on Repair Technique

2.3.1 Introduction

The increase in durability problems in recent years has been accompanied by a rapid growth in repair industry worldwide. However a prospective client continue***s*** to be faced with difficulties in the selection of a suitable repair system. This situation results mainly from a lack of information on the likely long-term effectiveness of repair treatments. The investigation programme comprises two phases:

Phase I: In order to establish the probable long-term performance of the repair materials with the parent concrete, it was decided to concentrate on some basic mechanical properties with long-term deformations of three different repair materials (namely A, B and C):

Material A, which is a high performance non-shrink repair material, supplied by FOSROC, UK, as a blend of Portland cement, graded aggregates, and additives to control expansion on both the plastic and hardened states.
Material B, which is a cement-based repair mortar, supplied by JAHN, Netherlands, as a ready to use powder to be mixed with water.
Material C, which is a single component cementitious mortar which incorporates most advanced cement chemistry, microsilica, fibre and styrene acrylic copolymer technology, supplied by FLEXCRETE, UK.

Phase II: The purpose is:

- to develop a data file in repair techniques to be implemented in the expert system; and
- to complement the existing results with experimental results.

Technical Developments and Results:

Phase I.

Strength: The compressive strength of (each repair material) 100 mm cubes at 3, 7, 14 and 28 days was determined. The flexural strength of $100 \times 100 \times 500$ mm prism specimens was determined under four point bending. The curing condition for the strength and elastic modulus test was at 20°C in water.

Elastic Modulus (static): The elastic modulus of each material was tested in accordance with BS 1881: Part 121. Strain readings were taken at each load increment with the 200 mm Demec gauge on opposite sides of 100 mm square $\times 500$ mm long prisms.

Compressive creep: As no formal creep tests were specified, a creep test method was adopted from that used traditionally for concrete. The specimen size used was 100 mm square by 500 mm long. Which were cured at 20°C in water for 28 days prior to loading. In order to calculate total deformation, shrinkage has been measured on separate control specimen and deducted from the total measured creep strain.

Drying Shrinkage and Swelling deformation: Unrestrained prisms of mortar 100 mm square by 500 mm long were used to measured shrinkage and swelling deformation from an age of 24 hours. The effect of varying the cure environment on such deformation was also examined. For shrinkage deformation following curing regime were adopted :

- 20°C and 30 percent R.H.
- 20°C and 45 percent R.H.
- 20°C and 55 percent R.H.
- cured in water for 28 days, then stored in air at 20°C and 55 percent R.H.

The table below gives the results of the various strength tests associated with each material.

Material	Age	Compressive strength (N/mm^2)	Flexural Strength (N/mm^2)	Elastic Modulus (kN/mm^2)
A	- 3 days - 7 days - 14 days - 28 days	36.20 46.30 56.75 63.70	7.74	31.98
B	- 3 days - 7 days - 14 days - 28 days	25.10 29.90 31.30 33.00	4.21	19.10
C	- 3 days - 7 days - 14 days - 28 days	31.00 39.70 41.95 44.00	3.69	18.30

From the test results of creep and shrinkage deformations after 90 days:

- Material C, the single component mortar with enhanced fiber and polymer properties, has a very high creep strain (1138 microstrain and 1819 microstrain at 30% and 45% stress/strength ratio respectively).
- Among the three repair materials, the lowest creep strains were exhibited by mineral porous material with no additives, material B.
- Curing in water the swelling deformation was 252 microstrain; while curing at 55, 45 and 30 percent relative humidities (at 20°C constant temperature) the shrinkage was 962, 1238 and

1474 microstrain respectively for repair material C. These results show the marked influence of moisture conditions of storage on length change of repair materials.

Phase II.

- A classification of repair techniques in concrete structures was developed;
- Tests were performed using some repair techniques in columns, to obtain the main characteristics of their structural behaviour.
- The main parameters to characterize the repair techniques were described and flow charts for correct procedures were presented;
- A correlation matrix between defects and repair techniques was developed and implemented in the system to help to choose the best repair.

Conclusion/Discussion:

Phase I: From experimental work carried out in this task, the following conclusions can be made:

- The results of the tests applied to three different repair materials revealed wide range of values for each property.
- Long-term deformation properties of repair material***s***, which may be of significance to the subsequent structural behaviour of the repaired member, have been presented.
- Material C, which is a single component cementitious mortar, which incorporates the most advanced cement chemistry, micro silica, fiber and styrene acrylic copolymer technology shows high deformations compared with other two repair materials.
- It is also found that condition of humidity of surrounding air during the cure period has a significant effect on the shrinkage deformation of repair materials**.**

Phase II. Correlation between Defects and Repair Techniques: Following the decision taken in the project of considering only the corrosion related defects, a data file and a correlation matrix was developed and implemented in the expert system. There are a wide range of publications concerning repair / rehabilitation of concrete structures available. Classifications of such repairs are not so easy to find. When they are available, they do not apply to the specific case of bridges and do not take into account the multitude of works that need to be done to keep them functional as well as structurally safe. As a matter of fact, most of the work done on a bridge after it has been built concerns maintenance and not rehabilitation.

Taking these factors into consideration, the classification that was developed does not concern only repair work but it also includes most maintenance works. Although very often the repair has to take into account the causes of defect, it is the latter that concerns whoever manages the bridge. So, it was decided that there should be a very close parallelism between the classification of the repairs and the classification of the defects, which were described in task 2.1. A matrix of correlation between corrosion related defects in concrete bridges and repair techniques were then developed. The matrix is organized so that each line represents a defect and each column a repair technique. In the intersection of each line and column a number representing the correlation between defect and repair technique is to be introduced. The criteria adopted for that number was:

0 *No correlation* no correlation whatever (direct or indirect) between the defect and the repair technique.

1 *Low correlation* preventive repair aiming at eliminating the cause or causes of the defect but not the deterioration.

2 *High correlation* defect repair aiming at eliminating the deterioration of the area in which the defect was detected.

Detailed Characteristics of Repair Techniques: An extensive description of the application of all repair techniques was prepared. A study was done of the decision parameters for selecting the effective repair techniques what lead to flow charts to be implemented in the expert system. The parameters were defined in order to describe the defect degree and to quantify the repairs for every defect and repair technique, a model for cost estimation for actual repair was made techniques can be estimated according to this model.

Study of Repair Techniques: The analysis of repair techniques has been experimentally studied for the case of several concrete columns and analyzing four types of repairs. The column design considered the original load capacity and in the experimental tests a discontinuity at intermediate height of them was prepared. The program was based on testing 15 close to real size column. Methods based on polymeric concrete jacket or steel plate jacket with injection of voids present high efficiency at a low or moderate cost. Angle bars with prestressed bolts have a very good behaviour and seem very reliable although are expensive and fastidious to apply, may be of interest for emergency strengthening. It was found useless, because of low strengths and strong scatter, the strengthening based on steel plate jacket made by adhesion of plates in the column faces and connecting corner steel angles adhered to the plates. The characteristics of these results in terms of calculations, costs, failure mode, and resistance efficiency were compared to choose the best repair technique.

2.3.2 Corrosion related defects

The following corrosion related defects in reinforced concrete bridges were considered. However, only those defects indicated with * are implemented in the prototype software modules BRIDGE1 and BRIDGE2.

A_C Concrete elements
- 1 Rust stain*
- 7 Delamination/spalling*
- 13 Crack over/under bar*

A_D Reinforcement
- 1 exposed bar*
- 4 Corroded bar*
- 5 Bar with reduced cross-section*
- 6 Broken bar*

A_E Bearings
- 2 Obstruction due to rust*
- 3 Broken retainer-bars*
- 6 Corrosion*
- 7 Deteriorated base plate/pot*
- 8 Detachment/failure of anchor bolts/pins*

A_F Joints
- 5 Obstruction due to rust*
- 6 Corrosion*
- 7 Detachment/failure of anchorages*
- 8 Loosening/failure of bolts/pins*

A_I Secondary Elements
- 2 Deteriorated traffic signs
- 4 Damaged curbs/traffic barrier wall
- 5 Damaged hand railing
- 6 Defective coating
- 7 Corrosion
- 8 Loosening/failure of bolts/pins
- 10 Damaged sidewalks
- 14 Deteriorated edge beams*
- 15 Damaged acroterium

2.3.3 Diagnosis methods

The following diagnosis methods correlated with corrosion related defects were considered and implemented in the prototype software modules BRIDGE1 and BRIDGE2.

M_A Direct visual observation 1 Unaided (except for rulers, calibrated wedges, scales, calipers, a watch and other day-to-day equipment) 2 Using telescopes, binoculars, micrometer, camera or video equipment 4 Using special means of aerial access 5 Underwater/on water M_B Mechanical techniques 1 Surface hammering/chain dragging 2 Sclerometer 1 Schmidt/Rebound/lmpact/Swiss hammer test 2 Williams testing pistol 3 Frank spring hammer 4 Einbeck pendulum hammer 4 Pull-out test *1 Conventional* 2 Capo test (or Lok-test) 3 Internal fracture method (or BRE test) 4 Expanding sleeve concrete test (ESCOT) 5 Anchorage system pull-out 6 Pull-off test 7 Pull-out after penetration	M_C Potential differences measurement 1 Galvanic cell test 1 Copper-copper sulfate half-cell 2 Multi-cell equipment M_D Magnetic techniques 1 agnetometer/Covermeter/Pachometer. M_F Ultrasonic techniques 1 Ultrasonic pulse velocity test 3 Pulse echo method 5 Radar M_G Radioactive methods 1 X-Ray propagation 2 Gamma Ray propagation 3 Radiation attenuation method (tomographic system) M_I Thermic methods 2 Infrared thermography M_K Chemical indicators 1 Phenolphtalein 2 Silver nitrate 3 Rapid chloride test 4 Rapid alkali test

2.3.4 Repair techniques

R_B Foundations/abutments/embankments
- 3 Foundation consolidation (jacking up and compaction)*

R_C Concrete elements
- 1 Cosmetic repair*
- 2 Concrete patching (with deteriorated concrete removal)*

R_D Reinforcement
- 1 Concrete patching (with reinforcement cleaning)*
- 2 Concrete patching (with reinforcement splicing/replacement)*
- 3 Concrete encasing (with reinforcement splicing/replacement)*

R_E Bearings
- 1 Removal of debris/moisture/trapped water/vegetation growth*
- 2 Replacement of the retainer-bars*
- 3 Replacement of the roller*
- 4 Blast cleaning/coating*
- 5 Replacement of the base plate/pot*
- 6 Replacement of the anchor bolts/pins*
- 9 Concrete patching of the bearing seat*
- 10 Repositioning of the bearing*
- 11 Replacement of the bearing*

R_F Joints
- 1 Removal of debris/moisture/trapped water/vegetation growth*
- 2 Blast cleaning/coating.*
- 3 Replacement of the anchorages*
- 5 Replacement of the filler/sealant*
- 6 Replacement of the joint*

R_H Water drainage
- 1 Removal of debris/obstructing asphalt from deck drain or gutter*
- 2 Gutter joint repair*
- 3 Deck drain extension downwards/upwards*
- 4 Diversion of point of discharge of deck drain
- 6 Replacement of drain/gutter/void tubes

R_I Secondary elements
- 3 Replacement of hand railing
- 4 Blast cleaning/coating
- 5 Replacement/tightening of bolts/pins
- 6 Welding repair
- 7 Replacement of sidewalks
- 8 Replacement of utilities
- 10 Replacement of edge beams*
- 11 Replacement of acroterium
- 12 Removal of vegetation growth

The following repair techniques correlated with corrosion related defects were considered. However, only those repair techniques indicated with * are implemented in the prototype software modules BRIDGE1 and BRIDGE2.

Repair techniques is divided in maintenance techniques (section 2.3.5), which are small and in expensive repairs and in structural repair techniques (section 2.3.6), which are big and expensive repairs.

2.3.5 Maintenance techniques

The following maintenance techniques correlated with corrosion related defects were considered and implemented in the prototype software modules BRIDGE1 and BRIDGE2.

R_C Concrete elements
- 1 Cosmetic repair

R_E Bearings
- 1 Removal of debris/moisture/trapped water/vegetation growth
- 4 Blast cleaning/coating
- 5 Replacement of the base plate/pot

R_F Joints
- 1 Removal of debris/moisture/trapped water/vegetation growth
- 2 Blast cleaning/coating

R_H Water drainage
- 2 Gutter joint repair
- 3 Deck drain extension downwards/upwards
- 4 Diversion of point of discharge of deck drain

R_I Secondary elements
- 10 Replacement of edge beams

2.3.6 Structural repair techniques

The following structural repair techniques correlated with corrosion related defects were considered and implemented in the prototype software modules BRIDGE1 and BRIDGE2.

R_C Concrete elements
- 2 Concrete patching (with deteriorated concrete removal)

R_D Reinforcement
- 1 Concrete patching (with reinforcement cleaning)
- 2 Concrete patching (with reinforcement splicing/replacement)
- 3 Concrete encasing (with reinforcement splicing/replacement)

R_E Bearings
- 2 Replacement of the retainer-bars
- 5 Replacement of the base plate/pot
- 6 Replacement of the anchor bolts/pins
- 9 Concrete patching of the bearing seat
- 11 Replacement of the bearing

R_F Joints
- 3 Replacement of the anchorages
- 6 Replacement of the joint

2.3.7 Causes correlated to corrosion related defects

The following causes correlated with corrosion related defects were considered. However, only those causes indicated with * are implemented in the prototype software modules BRIDGE1 and BRIDGE2.

C_A Design errors
- 1 Deficient layout of the bridge or its approaches*
- 3 Wrong choice of materials*
- 6 Missing temperature effects on long or skewed decks*
- 14 Insufficient reinforcement design cover*
- 15 Inadequate reinforcement spacing*
- 16 Other reinforcement detailing errors*
- 17 Deficient metallic connections design/detailing*
- 18 Deficient bearings design/positioning*
- 19 Deficient joints design/positioning*
- 20 Excessive exposed areas in structural elements/faulty geometry design*
- 21 Inability to predict the replacement of heavily deteriorated elements*
  of parts of the structure*
- 22 Difficulty/impossibility of inspection of parts of the structure*
- 23 Non-prevision of a minimum inclination in quasi-horizontal surfaces*

24 Drainage directly over concrete, a joint,

26 Lack of waterproofing membrane*
27 Deficient building specifications*
28 Incomplete/contradictory/over compact drawings*

C_B Construction errors
1 Wrong interpretation of the drawings*
2 Inexperienced personal*
3 Deficient soil compaction/stabilization*
4 Deficient materials transport/storing*
5 Changes in materials mixing proportions*
6 Use of inappropriate materials (contaminated water, over-reactive aggregates)*
8 Overuse of formwork/faulty formwork*
9 Deficient concrete compaction/curing*
11Inaccurate reinforcement positioning/detailing*
14 Early/faulty demoulding*
16 Faulty patching*
17 Faulty placing of waterproofing membrane*
18 Deficient asphalt paving/repaving of the deck*
19 Faulty asphalt patching*
20 Obstruction of drains with asphalt*
21 Faulty bolt/pin tightening*
22 Defective welding
23 Faulty coating*
24 Faulty construction/placing of joints*
25 Deficient placing of bearings*
26 Insufficient quality control*
1 Earthquake*
2 Fires*
4 Floods*
5 Earth-fall*
6 Snow-slip*
8 Tsunamis*
9 Thunderbolts*
10 Volcano eruptions*

C_D Man caused accidental actions
1 Fire*
2 Collision/traffic accidents*

C_C Natural accidental actions
3 Explosion/bombing*
4 Overloads*
5 Heavy objects dropped*
6 Vandalism*

C_E Environmental actions
1 Temperature*
2 Humidity (wet/dry cycles)*
3 Rains*
4 Snow*
5 Ice (freeze/thaw cycles)*
6 Winds
7 Direct solar radiation

C_F Natural aggressive factors
1 Water (wet/dry cycles)*
2 Carbon dioxide*
3 Salt/salty water (chlorides)*
6 Sulphates*
7 Alkali-aggregate reactions*
8 Abrasion (wind, sand, heavy objects suspended in a stream)
10 Biological action (algae, lichen, roots)

C_G Man-caused aggressive factors
1 Water*
2 Carbon dioxide*
3 Deicing salts.*
4 Pollution*
6 Abrasion (traffic, transport of materials)

C_H Lack of maintenance
1 Accumulation of rust/debris in the bearings*
2 Bearings (or components of) not functioning properly still in service*
3 Accumulation of rust/debris in the joints*
4 Joints (or components of) not functioning properly still in service*
5 Gutter/drains obstructed by debris*
6 Lack/loosening of pins/bolts*
7 Defective metallic coatings*
8 Heavy vegetation growth/burrows*

C_I Changes from initially planned normal use
5 Excessive traffic speed*
8 Foundations settlement*
11 Abnormal functioning of the bearings*

2.3.8 Correlation between defects and diagnosis methods

A so-called correlation matrix between the corrosion related defects (see section 2.3.2) and the diagnosis methods (see section 2.3.3) have been produced on basis of expert knowledge.

A “blank element” in the matrix means that there are no correlation whatever (direct or indirect) between the defect and the diagnosis method.

A “1” in the matrix means that the correlation is low. The diagnosis method may be useful as a second choice to a high correlated method when the high correlated method cannot be performed or gives inconclusive results. It may also be useful to give some secondary information on the extend or the cause off the defect.
A “2” in the matrix means the diagnosis method is, in principle, indispensable to the

	A_C			A_D				A_E					A_F				A_I
	1	7	13	1	4	5	6	2	3	6	7	8	5	6	7	8	14
M_A1	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
M_A2	1	1	1	1	1	1	1										1
M_A4	1	1	1	1	1	1	1	1	1	1	1	1					1
M_A5		1	1	1	1	1	1										1
M_B1	1	2	1														1
M_B2		1	1														
M_B4																	1
M_C1	1	1	1		2	2	1										1
M_D1	1		1														1
M_F1		1	1														1
M_F3		1															
M_F5		2															
M_G1							1										
M_G2							1										
M_G3		1															
M_I2		1	1														
M_K1	2	1	1	1	2	2											1
M_K2	1	1	1	1	1	1	1										1
M_K3	1	1	1	1	1	1	1										1
M_K4		1	1														1

inspection of the defect. It may provide essential information on the extent, degree and cause of the defect. It may be replaced by a low correlated method if for some reason (lack of equipment, workmanship, time, etc.) it can not be performed. Its use does not invalidate the use of other methods if more detailed information is thought necessary.

2.3.9 Correlation between defects and repair techniques

A so-called correlation matrix between the corrosion related defects (see section 2.3.2) and the repair techniques (see section 2.3.4) have been produced on basis of expert knowledge.

A “blank element” in the matrix means that there are no correlation whatever (direct or indirect) between the defect and the diagnosis method.

A “1” in the matrix means that the correlation is low. The repair technique gives a preventive repair aiming at eliminating the cause or causes of the defect but not the deterioration.

A “2” means high correlation. The repair technique gives a defect repair aiming at eliminating the deterioration of the area in which the defect was detected.

2.3.10 Correlation between defects and maintenance techniques

A so-called correlation matrix between the corrosion related defects (see section 2.3.2) and the maintenance techniques (see section 2.3.5) have been produced on basis of expert knowledge.

A "blank element" in the matrix means that there are no correlation whatever (direct or indirect) between the defect and the diagnosis method.

A "×" means high or low correlation. The maintenance technique aims at eliminating the deterioration of the area in which the defect was detected or eliminating the cause(s) of the defect.

	A_C			A_D				A_E					A_F				A_I
	1	7	13	1	4	5	6	2	3	6	7	8	5	6	7	8	14
R_B3 (R)															1		
R_C1 (M)	2																2
R_C2 (R)	2	2		2											2	2	2
R_D1 (R)	2	2	2	2	2	2											2
R_D2 (R)	2	2	2	2	2	2	2										
R_D3 (R)							2										
R_E1 (M)								2	1		1				1		
R_E2 (R)									2						1		
R_E3 (R)															1		
R_E4 (M)								2		2							
R_E5 (R)											2				1		
R_E6 (R)												2			1		
R_E9 (R)												2					
R_E10 (R)								1	1	1	1	1			1		
R_E11 (R)									2	2	2	2					
R_F1 (M)													2				
R_F2 (M)													2	2			
R_F3 (R)															2	2	
R_F5 (R)								1	1	1	1		1	1	1	1	
R_F6 (R)														2	2	2	
R_H2 (M)	1				1	1											
R_H3 (M)																	1
R_I10 (M)								1	1	1	1	1	1	1	1	1	1

	A_C			A_D				A_E					A_F				A_I
	1	7	13	1	4	5	6	2	3	6	7	8	5	6	7	8	14
R_C1	X																X
R_E1								X	X		X				X		
R_E4								X		X							
R_F1													X				
R_F2													X	X			
R_H2	X				X	X											
R_H3																	X
R_H4								X	X	X	X	X	X	X	X	X	X
R_I10																	X

2.3.11 Correlation between defects and structural repair techniques

A so-called correlation matrix between the corrosion related defects (see section 2.3.2) and the structural repair techniques (see section 2.3.5) have been produced on basis of expert knowledge.

A "blank element" in the matrix means that there are no correlation whatever (direct or indirect) between the defect and the diagnosis method.

A "2" means high correlation. The structural repair technique aims at eliminating the deterioration of the area in which the defect was detected.

	A_C			A_D				A_E					A_F				A_I
	1	7	13	1	4	5	6	2	3	6	7	8	5	6	7	8	14
R_C2	2	2		2											2	2	2
R_D1	2	2	2	2	2	2											2
R_D2	2	2	2	2	2	2	2										
R_D3							2										
R_E2									2								
R_E5											2						
R_E6												2					
R_E9												2					
R_E11									2	2	2	2					
R_F3															2	2	
R_F6														2	2	2	

2.3.12 Correlation between defects and causes

A so-called correlation matrix between the corrosion related defects (see section 2.3.2) and the causes (see section 2.3.7) have been produced on basis of expert knowledge.

	A_C			A_D				A_E					A_F				A_I
	1	7	13	1	4	5	6	2	3	6	7	8	5	6	7	8	14
C_A1									1		1	1			1	1	
C_A6									1			1			1		
C_A14	2	2		1	2	2						1			1		2
C_A15			2														
C_A16																	2
C_A17												2				2	
C_A18								1	2	1	1	2					
C_A19		1		1				1		1	1		1	1	2	1	
C_A20	1				1	1											1
C_A21																	1
C_A22								1					1				
C_A23	1				1	1											
C_A24	2				2	2		2	1	2	2	1	2	2	1	1	2
C_A25	1				1	1											1
C_A26	1	1			1	1											
C_A27		1	1	1													1
C_A28	1	1	1	1	1	1		1	1	1	1	1	1	1	1	1	1
C_B1	1	1	1	1	1	1		1	1	1	1	1	1	1	1	1	1
C_B2	1	1	1	1	1	1		1	1	1	1	1	1	1	1	1	1
C_B3															1		
C_B4		1		1													1
C_B5	1		2		1	1											
C_B6		1		1													1
C_B8			1														
C_B9	2	1	2		2	2											1
C_B11	2	2	2	1	2	2						1			1		2
C_B14		2		2													2

A “blank element” in the matrix means that there are no correlation whatever (direct or indirect) between the defect and the causes.

A “1” in the matrix means that the correlation is low. Indirect cause of the defect connected only with the very early stages of the deterioration process; secondary cause of the deterioration process and not necessary for its development.

A “2” in the matrix means that the correlation is high. Direct cause of the defect associated with the final stages of the deterioration process; when the cause occurs, it is one of the main causes of the deterioration process and is indispensable to its development.

	A_C			A_D				A_E					A_F				A_I
	1	7	13	1	4	5	6	2	3	6	7	8	5	6	7	8	14
C_E4	1				1	1											
C_E5		2		2													2
C_F1	2	1	1	1	2	2											1
C_F2	2	1	1	1	2	2											1
C_F3	2	2	2	1	2	2	1	2	1	2	2	1	2	2	1	1	2
C_F6		2		2													2
C_F7		2		2													2
C_G1	2	1	1	1	2	2											1
C_G2	2	1	1	1	2	2											1
C_G3	2	2	2	1	2	2	1	2	1	2	2	1	2	2	1	1	2
C_G4																	1
C_H1								2	1		1				1		
C_H2									2		2	2			2		
C_H3													2				
C_H4								1	1	1	1		2	2	1	1	
C_H5	1				1	1											
C_H6												2				2	
C_H7								2		2	2		2	2		1	
C_H8									1		1						
C_I5																1	
C_I8															2		
C_I11									2		2				2		

2.4. Development of Stochastic Models

The purpose of this task is to develop stochastic models for deterioration of reinforced concrete structural elements. Stochastic models for inspection, maintenance and repair will also be considered.

Some simple models of the constant K in the model of depth of carbonation front, the chloride diffusion coefficient D_C and the rate of corrosion i_{corr} were investigated based on the data collected in task 1.3. The data matrices are completed by filling-in values by the FORTRAN program MISDAT developed in task 1.3. Final conclusions regarding the investigated models cannot be drawn because of the very limited number of data matrices available. For the three parameters mentioned above there are too many variables in the models (6, 6 and 7 variables) compared with the number of available cases (18, 20 and 18 cases). Furthermore, some of the cases have missing values (7, 8 and 7 cases).

Models for failure, inspection and repair have been formulated. In the models of failure of reinforced concrete structural elements the models for deterioration developed in task 1.4 are used.

Two different failure modes are considered, namely bending failure of the main beam of a bridge and compression failure of a column. For bending failure both 'positive' and 'negative' bending failure are considered. For compression failure two models for deterioration of the column are considered, namely one model where the concrete deteriorates on all four sides of the column and one model where the deterioration is concentrated on one side. In the models the diameter of the reinforcement is assumed to decrease with time due to corrosion. In the failure modes both chloride and carbonate initiated corrosion are considered.

Two types of uncertainty in the models for inspections are considered. The first type of uncertainty is related to the uncertainty (reliability) of an inspection method, i.e., how good is an inspection technique to detect a defect if a defect is present and what is the risk that the inspection method indicates a defect when there is no defect (false alarm). The second type of uncertainty is related to the measurement uncertainty when a detected defect is being quantified. Stochastic models are derived for the following inspection methods:

- 'Half-cell potential' test for detection of corrosion in reinforcement steel
- Measurement of depth of concrete cover
- Measurement of the carbonation depth by indicator spray
- Titration based estimates of chloride content.

A repair implies that new and/or modified values of parameters are needed to model the behaviour of the bridge after the repair. In relation to stochastic modelling of repair the quantities can be divided in the following groups:

- Quantities (deterministic or stochastic), which are the same before and after repair.
- Quantities, which can be modelled by deterministic variables. The values for these quantities are known rather precisely after the repair.
- Quantities, which can be considered as new outcomes of the old stochastic variables used before the repair. A variable of this type is modelled by introducing a new stochastic variable with the same distribution function but statistically independent with the old stochastic variable.
- Quantities modelled by new stochastic variables correlated or not correlated with the old stochastic variables.

In addition to the above models it can be relevant to update the distribution functions of the stochastic variables when observations are obtained in connection with the repair. The following structural repair types are modelled: concrete patching (with deteriorated concrete removal), concrete patching (with reinforcement cleaning), concrete patching (with reinforcement splicing/replacement) and concrete encasing (with reinforcement splicing/replacement).

Models of the following parameters have been investigated: the constant K in the model of the depth of carbonation front, the D_C chloride diffusion coefficient and the rate of corrosion i_{corr}. Conclusions regarding the usefulness of the investigated models cannot be drawn because of the very limited number of data. Therefore, these models will not be used further in the project. The parameters K, D_C and i_{corr} will be given as input to the expert system by the user.

Stochastic models are developed for:

- Two different failure modes
- Four inspection methods
- Four repair techniques.

2.5. Development of Updating Techniques

The purpose of this task is to develop methods by which the stochastic models can be updated on basis of new information e.g. from inspections, repairs and new measurements of environmental parameters. Three types of information are considered, namely samples of basic variables, events and linguistic information.

When new information becomes available the estimates of the probability of failure (and the reliability) of structures can be updated. New information is divided in three types:

- Sample information on basic variables
- General information on stochastic variables
- Linguistic information.

When new information is available as samples of one or more stochastic basic variables Bayesian statistical methods are used to obtain updated (predictive) distribution functions of the stochastic variables. An uncertain quantity modelled by a stochastic variable X is considered. $\vec{\Theta}$ is a vector of parameters defining the distribution for X. One or more of the parameters in $\vec{\Theta}$ are considered as stochastic variables. The initial joint density function of the stochastic variables in $\vec{\Theta}$ is denoted the prior density function. It is assumed that an experiment or inspection is performed. Realizations of the stochastic variable X are obtained and denoted $\vec{x}^*$. The measurements are assumed to be independent. The updated joint density function of the stochastic variables in $\vec{\Theta}$ is denoted the posterior density function. The updated density function of the stochastic variable X taking into account the realizations $\vec{x}^*$ is denoted the predictive density function.

An updating FORTRAN program DISTUP is developed which is used to estimate updated statistical parameters in posterior and predictive functions based on the parameters in the prior distribution and the observations. The following distribution types are implemented: normal distribution with unknown mean or/and unknown standard deviation, lognormal distribution with unknown location parameter, Gumbel distribution with unknown location parameter, Weibull distribution with unknown location parameter, Frechet distribution with unknown location parameter and exponential distribution with unknown location parameter.

In some cases the information obtained by measurements is not directly related to a basic stochastic variable. The information is generally modelled by using a stochastic variable, which is a function of the basic stochastic variables. Models of inequality and equality events are formulated. The event margin is a stochastic variable and it is therefore possible to estimate the probability that the event occurs. Further, this type of information can be used to update the probability of failure of a structural element. The updating based on information on basic variables can be considered as a special case of the general updating methods.

An updating FORTRAN program BETAEQ is developed which is used to estimate the probability of failure of the following models:

- Parallel system model of equality and inequality events
- Parallel system model of equality and inequality events conditioned on equality and inequality events
- Parallel system model of equality and inequality events in parallel with a series system of inequality events conditioned on equality and inequality events.

Linguistic information cannot directly be used to update the estimates of the probability of failure. Possible techniques/methods to be used are:

- 'Translation' of the information into events modelled by stochastic variables. The estimates can then be updated as described above.
- Application of expert system techniques.
- Application of Fuzzy set techniques.

Two FORTRAN programs DISTUP and BETAEQ are developed to update stochastic models on basis of new information related to samples of basic variables and events. When possible the linguistic information is translated into events modelled by stochastic variables.

2.6. Development of Reliability Models

The purpose of this task is to develop reliability methods to estimate the time-dependent reliability of reinforced concrete bridges taking into account deterioration of the reinforcement.

Both time-invariant and time-variant reliability analysis are performed for an example where the following effects are modelled stochastically:

- Deteriorating strength of a reinforced concrete cross-section. Corrosion due to chloride and/or carbonation penetration is considered.
- A stochastic process model describes the traffic load.

In the time-invariant reliability analysis the estimate of the probability of failure is obtained by considering the extreme load in the lifetime T_L and the strength at time T_L. The time-variant reliability analysis is a so-called first-passage problem; an approximate solution based on out crossing rates is used. The time-invariant reliability analysis gives a higher reliability index than the time-variant reliability analysis. The results of the analysis indicate that for small values of the expected value of the coefficient, which determines the corrosion, rate a reasonable estimate of the reliability can be obtained using a time-invariant reliability method. The other parameters investigated (number of traffic loads per year, magnitude of traffic load and a coefficient which determines the time when corrosion starts) do not influence significantly on the difference between the time-invariant and time-variant reliability estimates. Further, the calculation time of a time-variant reliability index calculation is approximately 80 times the calculation time of a time-invariant reliability index calculation. The results of the investigation of influence of the parameters and the large difference in computation time indicate that a time-variant reliability analysis should only be performed if it is absolutely necessary.

To estimate the time-independent reliability of reinforced concrete bridges taking into account deterioration of the reinforcement a number of FORTRAN reliability modules are developed. Further, the databases used by BRIDGE2 are implemented. The FORTRAN reliability programs are shortly described below. They are implemented in a form independent of the expert system so that they can be used independent of BRIDGE1 and BRIDGE2. Thereby the developed reliability programs can be used to assess the reliability of any type of structure. The main FORTRAN reliability program RELIAB in BRIDGE2 only contains the most important FORTRAN modules in a short version as problems with the memory handling occurred during the integration of RELIAB with the expert system.

BETAEL: *First-order reliability analysis for a single failure element*. The reliability index is estimated by approximating the failure surface by its tangent hyper-plane at the design point in the standardized space. Two first-order reliability methods are implemented, namely an iteration using the Rackwitz-Fiessler algorithm and optimization using the NLPQL algorithm. Sensitivities of the estimated reliability index with regard to statistical parameters for stochastic variables and deterministic parameters in the failure function are evaluated.

DISTRB: *Distribution library*. In order to estimate the reliability index of a single failure element a transformation is defined of the generally correlated and non-normally distributed variables into uncorrelated, standardized and normally distributed variables. The transformations and the following distribution functions are implemented: deterministic, uniform, normal, logarithmic normal, exponential, Gamma, Gumbel, Frechet, Weibull, Rayleigh, Normal with unknown mean or/and unknown standard deviation, Lognormal with unknown location parameter, Gumbel with unknown location parameter, Weibull with unknown location parameter, Frechet with unknown location parameter and exponential with unknown location parameter.

FAILEL: *Failure element library*. The failure modes are described by failure functions, where uncertain quantities are modelled by stochastic variables. The failure functions are implemented in this module.

GAUSSD: *Gaussian distribution functions*. Several times during a reliability analysis evaluation of the Gaussian (normal) distribution functions implemented in this module are needed. The following Gaussian distribution functions are implemented: one-dimensional Gaussian density function, one-dimensional Gaussian distribution function, one-dimensional inverse Gaussian distribution function and multi-dimensional Gaussian distribution function. The multi-dimensional Gaussian distribution function is estimated by the Hohenbichler approximation.

BETASY: *System reliability analysis*. If a failure state of a structural system requires that more than one element fails then the set of elements defining the failure mode is modelled by a parallel system. The parallel systems are again combined as a series system whereby the structural system fails when the weakest parallel system (sequence of failure modes) fails. The system reliability index can be estimated for general series systems, parallel systems and series systems of parallel systems. Bounds for the reliability index are calculated and sensitivity analysis is performed.

BETASO: *Second-order reliability analysis for a single failure element*. A three-term approximation formula is implemented. The second-order reliability index is based on the main curvatures of the failure surface at the design point in the standardized space.

BETASI: *Reliability analysis for a single failure element using simulation*. No approximation of the failure surface in the standardized space is used. Two Monte-Carlo simulation techniques are implemented: hit-or-miss Monte-Carlo simulation and Monte-Carlo simulation with importance sampling.

BETAZIP: *Beta-unzipping analysis of structural systems*. The total number of failure modes for a structural system is usually too high to include all possible failure modes in the estimation of the reliability. The majority of the possible failure modes will in general have a relatively small probability of occurrence. The β-unzipping technique is used to identify all relevant failure modes of the structural system. When a failure mode is violated the structural model is modified in accordance with the failed element type. The identified failure modes are modelled as parallel systems in a series system. In the finite element module in BETAZIP truss elements and two- and three-dimensional beam elements are included. Failure is defined by failure of a given number of failure elements or formation of a mechanism. The implemented failure elements are: two- and three-dimensional tubular elements with yielding failure, punching failure and stability failure.

RELIAB: *Main reliability program in BRIDGE2*. This program is used to estimate the reliability of a reinforced concrete bridge. The failure functions modelling the failure modes developed in task 2.4 are implemented in the failure element module FAILEL. The failure functions are described by stochastic variables with the distribution functions implemented in DISTRB. The reliability index for each failure mode is determined using BETAEL. The failure modes are modelled as elements in a series system and a system reliability index is determined using BETASY. The Gaussian distribution functions are evaluated using GAUSSD. When inspection results are obtained the reliability indices for single failure modes and for the system are updated using the updating module BETAEQ developed in task 2.5. The inspection results are modelled as inequality events or equality events by the models developed in task 2.4. Two inspection parameters of the observed defects are used to update the stochastic models, namely the depth of the defect and the observed bar diameter.

The stochastic variables used in RELIAB are listed in the following table, where the description, distribution type and coefficient of variation are shown for each variable.

The correlation structure for the stochastic variables is modelled in the following way:

- All the stochastic variables in the table above are assumed to be independent.
- Stochastic variables numbered 1–10 are assumed to be common for all cross-sections in a given bridge.
- Stochastic variables numbered 11–21 are assumed to be independent from cross-section to cross-section.
- The inspection measurement uncertainties for different inspections are assumed to be independent.

No.	Mean value of...	Distribution type	Coefficient of variation
1	compression strength for concrete (deck)	Lognormal	0.15
2	yield stress for steel (deck)	Lognormal	0.10
3	compression strength for concrete (column)	Lognormal	0.15
4	yield stress for steel (column)	Lognormal	0.10
5	uniformly distributed dead load	Normal	0.05
6	uniformly distributed traffic load	Gumbel	0.13
7	point traffic load	Gumbel	0.20
8	chloride diffusion coefficient in concrete	Lognormal	0.20
9	coefficient rate of carbonation	Normal	0.20
10	rate of corrosion of reinforcement	Normal	0.10
11	distance from underside of beam to center of reinforcement in layer 1	Normal	0.10
12	height of beam	Normal	0.03
13	height of deck	Normal	0.03
14	diameter of bars in layer 1	Normal	0.02
15	diameter of bars in layer 2	Normal	0.02
16	diameter of bars in layer 3	Normal	0.02
17	diameter of bars in layer 4	Normal	0.02
18	width of column	Normal	0.05
19	depth of column	Normal	0.05
20	diameter of bars in column	Normal	0.05
21	chloride concentration on the concrete surface	Normal	0.20
22	inspection measurement uncertainty	Normal	①

① The coefficient of variation for inspection measurement uncertainty is dependent of the type of inspection.

A time-invariant and time-variant reliability analysis of a deteriorating reinforced concrete bridge shows that the time-invariant analysis gives reasonable estimates of the reliability. The calculation time of a time-variant reliability index calculation is approximately 80 times the calculation time of a time-invariant calculation. Therefore, time-invariant reliability methods are used.

FORTRAN programs are developed to estimate the reliability of single failure elements, systems of failure elements and structural systems. The reliability of single failure elements can be estimated by first- and second-order reliability methods and simulation. In the developed main reliability program RELIAB integrated with the expert system, the estimated reliability index is updated based on inspection results.

2.7. Main Results and Conclusions

The results from the experiments provided evidence to indicate that corrosion of reinforcement in concrete leads to severe deterioration of bond strength between concrete and the reinforcing steel with subsequent inadequate transmission of stresses between the two materials, leading to deterioration of the load-carrying capacity of the structural member. An empirical model relating the residual flexural strength of corrosion-damaged beams with corrosion rate, corrosion duration and rebar diameter was developed. A classification of defects and their causes in concrete bridges has been made. Different repair treatments have been defined and the degree of strengthening required for the repair of the defects have been established. As a result of extensive experimental research on the influence of different corrosion induced defects in beams and columns, it was concluded that to differentiate between the repair materials only on the basis of the immediate gain in the load carrying capacity of the repaired element is misleading. Therefore, long-term deformation properties of three generic repair materials, which are significance to the structural behaviour of the repaired member, are presented. Correlation matrices between corrosion defects/repair techniques and corrosion defects/causes have been developed to use in BRIDGE 1 and BRIDGE 2. Also the characteristics of repair techniques results in terms of calculations, costs, and failure mode and resistance efficiency were compared to choose the best repair technique. Two FORTRAN programmes DISTUP and BETAEQ were develop to update stochastic models. It is also concluded that the time-invariant reliability analysis of a deteriorating reinforced concrete bridge gives reasonable estimates. Also FORTRAN programmes are developed to estimate the reliability of single failure elements, systems of failure elements and structural systems.

3. OPTIMAL STRATEGIES FOR INSPECTION AND MAINTENANCE OF CONCRETE STRUCTURES

The main objectives of this subproject 4 are the definition of the optimal strategies for inspection, maintenance and repair of concrete bridges.

The subproject presented at the beginning two tasks, one dealing with the characterization of the inspection techniques and the other with a review of the existing bridge management systems in the world.

The inspection techniques were characterized with a proposed rating and correlated with the defects. This correlation was implemented in the expert system (BRIDGE1) as a module to help the inspectors, suggesting the best technique to perform the inspection of the defect. An error analysis of the methods was also performed in relation to the reliability analysis.

About the bridge management systems, it was tried to collect as many references as possible on this subject and an effort was made to put forward all the relevant information in an organized and succinct way. A review of the management systems of several countries in Europe and America was presented, in order to provide some guidelines on the implementation of an expert system.

In the third task methods and computer programs for determining rational inspection and maintenance strategies for concrete bridges have been developed. The optimal decision is based on the expected benefits and total cost of repair and failure of the bridge.

The objective of the last task was to apply the expert system to real bridges. For this testing, it was necessary to select bridges where the following aspects should be considered:

- To use current bridges like short and medium span overpasses;
- To understand well the bridge behaviour, maintain a good construction control and to test the bridge and compare results with numerical methods;
- To inspect the bridge periodically to control the deterioration aspects.

Several bridges were selected with some of the above aspects, and case studies were performed to do their analysis using BRIDGE1 and BRIDGE2. This allowed the evaluation of the functionality of the expert system, with real bridges.

3.1. Review of Inspection Techniques

The main objectives of this task are to characterize the inspection techniques in bridges and to define the best procedures in each situation.

Based on the classification developed for the defects, the inspection techniques were characterized with a proposed rating and correlated with the defects. This correlation was implemented in the expert system BRIDGE1 as a module to help the inspectors, suggesting the best technique to perform the inspection of the defect. An error analysis of the methods was also performed in relation to the reliability analysis.

Classification of Diagnosis Methods: Diagnosis of concrete bridges showing signs of functional or structural deterioration are the first step that has to be taken before making any decisions. Before starting any diagnosis work, it is necessary to define clearly what the damage problems are. The reasons for concern usually point out a direction for investigation. It is very timely and money consuming to start diagnosis without knowing which information one wants to gather.

When the method (or methods) is selected, it is necessary to gather the know-how, equipment, manpower and facilities needed. The method procedure needs to be known accurately and the information needed has to be written down in order to avoid many visits to the site. Diagnosis work is usually disruptive for the normal functioning of the bridge and must be limited as much as possible in time and space. The interpretation of results can be a very frustrating task as experimental results very often are confusing, contradictory and do not follow highly mathematical and theoretical patterns. Faced with such results, the person in charge has to root out the dubious ones (and try to find an explanation for them), confirm the logical ones and, if necessary, point out the information needed from a further visit. Laboratory work, even though useful to clarify some points, must be avoided, as it is time and money consuming.

The limitations of each method have to be known: sometimes the accuracy of some methods does not permit anything but qualitative diagnosis. At the present level of know-how in the field of diagnosis, very few (and usually expensive) methods can give away quantitative information that is reliable to an acceptable degree.

Diagnosis methods for concrete bridges include a great number of tests based on principles as different as material behaviour to vibrations, its behaviour under a magnetic field or its surface hardness. The information gathered from them includes characteristics as distinct as tension or compression resistance, extension and rate of activity of steel corrosion within the concrete, place and depth of cracking, etc.

A practical classification is thus difficult and quite often these methods are divided into groups according to the amount of damage done during its performance (non-destructive, semi-destructive and destructive). Some authors choose to classify the tests according to the main principle used (electrical, acoustical, magnetic, mechanical, etc.) or to the results obtained (geometric measurements, resistances, deformations, etc.).

An attempt of a different classification was presented in this project. Even though no strict criteria are used, the listing does not include most laboratory performed tests. As a matter of fact, diagnosis heavily depends on in situ testing and, whenever possible, in situ interpretation of the results. On the other hand, this list was made as thorough as possible in order to include the latest advancements on this field i.e. tests which are potentially interesting but have not yet proved worthwhile in situ.

Rating of Diagnosis Methods : There is a large amount of experimental methods used for damage classification of concrete bridges. They vary a lot in cost, equipment used, information given and necessary know-how and workmanship. The knowledge of what has to be found out usually reduces the choice to a handful of methods. In this task the methods are rated according to the following characteristics:

A- low cost
B- easy and fast in situ performance
C - high amount of useful information
D - easy interpretation of results
E - non-destructiveness
F - transportability of equipments
G - no source of energy necessary (or energy easily accessible in situ)
H - not over specialized workmanship or know-how
I - reliability of the results
J - (whenever possible) no laboratory work needed
K - no (or small) disruption to the normal use of structure

The main methods referred in the list mentioned above are analyzed according to these criteria. A table was defined with the results of the rating, based on the following classification:

2 the method complies with the criteria
1 the method does not completely fill the requirements
0 the method does not comply with the criteria

The results of this rating are arguable (as some of the criteria are more important than some other) but it is felt that they contribute to an easier choice of the day-to-day diagnosis methods that show the most promising features. Methods with a low rating are usually limited to very specialized studies (quite often with laboratory backing), at high costs and very time consuming. They are to be used only when the ‘best’ methods do not yield the needed results.

´The Correlation Matrix: A correlation matrix between the diagnosis methods and the defects was organized so that each line represents a defect and each column a diagnosis method. In the intersection of each line and column a number representing the correlation between defect and diagnosis method is to be introduced. The criteria adopted for that number is:

0	*No correlation*:	no correlation whatever (direct or indirect) between the defect and the diagnosis method.
1	*Low correlation:*	the diagnosis method may be useful as a second choice to a high correlation method when this last one can not be performed or gives inconclusive results; it may also be useful to give some secondary information on the extent or cause of the defect.
2	*High correlation:*	the diagnosis method is, in principle, indispensable to the inspection of the defect; it provides essential 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 necessary.

This matrix is implemented in BRIDGE1 to help the inspector in choosing the best inspection method, as a function of the detected defect.

Errors in Concrete-Related Measurements: In practice, certain parameters are considered to be of fundamental importance in assessing the performance of concrete, and, therefore, dictate the investigation strategy and its implementation. A brief description of some of these parameters, and the errors commonly associated with their measurements was analysed.

Concrete Cover: A cover meter is the device, which is most commonly used in practice to identify the depth of concrete cover to reinforcement and to assess its contribution to reinforcement protection. Provided the diameter of the steel bar is known, the concrete cover to reinforcing bars can be indicated by measuring the change of inductance of an iron cored inductor as it is moved in the proximity of steel reinforcing. By polarizing a search head containing the inductor, the direction of a specific bar may also be determined. According to concrete cover measurements carried out on dry and wet beams, the absence of water appears to facilitate for more precise readings. Concrete cover meter readings on dry beams indicate a relative standard deviation of 4.79%, compared to 10.42% for wet beams.

Concrete Chloride Content: In order to measure the depth of penetration of chloride from external sources, such as de-icing salt, chloride content is determined on samples drilled from cores at intervals (usually 10 mm) from the surface, to a depth well beyond the level of reinforcement. Chloride content is then determined by titration, such as potentiometer titration against silver nitrate solution. Titrimetry is a relatively simple and still widely adopted procedure. Important among the contributions to the errors are the tolerances of the weights used in the gravimetric steps, and of the volumetric glassware used in the process. Weighing by difference usually minimizes the systematic errors. If the titration solutions used are of sufficient volume (not less than 10 ml), random errors in the volumetric procedures will not exceed 0.02%. In practice, a good volumetric analysis should have a relative standard deviation of not more than 0.1%.

Concrete Electrical Potential: The “half-cell potential” test is usually employed in practice to identify the probable extent of the ongoing corrosion of the steel reinforcement, especially in areas where no surface evidence of corrosion exists. The test apparatus basically comprises a rigid Perspex tube containing a copper rod immersed in saturated copper sulfate solution. The rod passes through the end of the tube, where it is provided with an electrical terminal. The face of the tube usually incorporates a 20 mm diameter porous plate above which is a sponge prewetted with copper sulfate solution. Connecting the terminal to a section of exposed reinforcement and pressing the sponge in contact with the concrete surface provide electrical continuity. The potential difference between the steel reinforcement and the copper rod in the half-cells is then measured by means of high impedance Voltmeter. There are a large number of possible factors, which may affect electrical potential measurements, and hence the values determined. Many of the effects may be regarded as insignificant compared with the electrical potential changes induced by the differences up to 0.4 Volts. The following table shows some of the influences on measured electrochemical potentials carried out by a number of workers.

Measurement Variation	**Magnitude of effect (mV)**
1 - Stability of Voltmeter	+ 6 mV
2 - Voltmeter accuracy	up to 20 mV
3 - Electrical connections	+ 7 mV
4 - Reproducibility of single reference electrode	+ 10 mV
5 - Electrolyte concentration in reference electrode	+ 40 mV
6 - Thermal coefficient of reference electrode	Approx. 1 mV per °C
7 - Moisture content of concrete	+ 70 mV
8 - Changes in original W/C	+ 100 mV
9 - Salt concentration	up to 250 mV
10 - Carbonation of surface	250-450 mV
11 - Concrete degradation	up to 400 mV

3.2. Review of Available Inspection and Maintenance Strategies

The main objectives of this task are the analysis of existing management systems for bridge maintenance, in the world, and to define their main parameters towards an optimal strategy.

The main result of this task is the definition of the basic lay-out of the bridge maintenance strategy, based on which the expert system was developed.

The implementation of bridge management systems and in particular those based on computerized expert systems is very recent. Even in the most advanced countries, a complete expert system is either still a plan for the future or is initiating a trial period. The management of bridges is different from country to country and, even within a country, only very rarely is the responsibility of a single entity. This results in a multitude of different approaches to the problem with many investigation teams working in parallel, very often multiplying the total effort to achieve the same thing.

In this project it was tried to collect as many references as possible on this subject but only in a few cases was able to have access to complete manuals being used in practice. Very often, the references referred to is the work of consultant firms or particular departments of a larger organization. Also, most of the references concerned only a small part of the expert system: the data base, the inspection procedure, the decision criteria, the structural evaluation, etc. An effort was made to put forward all the relevant information in an organized and succinct way.

A review of the management systems of several countries in Europe and America is presented, in order to provide some guidelines on the implementation of an expert system.

France - The Roads Department of the French Ministry of Transportation prepared a document named “Technical Instruction for the Surveillance and Maintenance of Engineering Structures”. It is divided in many fascicles: general rules, bridge files, current surveillance, high surveillance, foundations, bearings, approaches, auxiliary equipment, etc. There is also one fascicle for each particular 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 taking into account the features of the structure to be inspected. It is a very thorough document, which covers the inspection module and the maintenance module of the expert system as it was proposed for the project.

At the time that it was first published, 1979, there wasn't a very clear guidance to the use of computer either to store data or to develop degradation mechanisms. Neither was a decision module developed.

Canada - The Highway Engineering Division of the Ontario Ministry of Transportation prepared four documents named “Ontario Structure Inspection Manual - OSIM”, “Ontario Structure Inspection Management System (OSIMS) User's Manual”, “Structure Rehabilitation Manual” and “Structural Financial Analysis Manual”. Together they form what has been found to be nearest to an expert system on bridges actually being used in practice. The inspection module is detailed with a maintenance (and rehabilitation) module and is a first step towards the decision module is presented.

The system very clearly points to the widespread use of computers both in handling the vast amount of information stored and in making decisions at the inspection site and at the headquarters. In the edition analysed, the decision system hadn't yet been completely developed as it was reduced to the project (single bridge) level. Some other parts of the documents are also still incomplete even though some updating has been done since the first edition.

U.S.A. - The Pennsylvania Department of Transportation organized a Bridge Management Task Group with the objective of developing a Bridge Management System (BMS).

In the system presented, little focus is given to the inspection module which does not seem to present any novelties. On the other hand, the decision system, both at the maintenance and rehabilitation / replacement levels, is very clearly defined and allows the easy use of computer programs for decision-making.

The system has but a few years of practical use and some of the coefficients proposed at this stage may have to be calibrated taking into account the experience acquired.

U.K. - There was not a direct access to any document describing a complete bridge expert system being used in the United Kingdom. The references gathered concern particular parts of the country or refer only in general terms to one of the modules that make a complete expert system: inspection, maintenance and decision. However, there are some promising results particularly in the area of computer software to automatize bridge management.

Switzerland - At the École Polytechnique Fédérale de Lausanne a PhD Thesis named “Bridge Maintenance: Surveillance Methodology” was prepared in 1987. Even though its conclusions and recommendations have not, been officially adopted throughout Switzerland, it is still the most complete document on bridge expert system in Switzerland that has been collected.

Even though the document is very thorough on the description of the inspection and maintenance modules, its approach to the decision system is not very satisfactory. Only the maintenance subsystem is approached (and the criteria mentioned is 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 hasn't been used in the field without some kind of adaptation. No mention is made to the use of computer databases or management systems.

Denmark - The Danish bridge management system DANBRO is presently used by the Danish State Railways (2500 bridges) and the Department of Highways in Thailand (10000 bridges). The objective of DANBRO is to give the bridge authority a decision tool that helps to:

- Ensure the safety and capacity of a network of bridges;
- Ensure objective information on all bridges;
- Optimize utilization of allocated funds;
- Ensure technical-economical feedback.

Rational handling of all information related to inspection, repair maintenance, etc. is done through DANBRO taking into account experience from more primitive management systems. DANBRO allows for determination of priorities of maintenance and rehabilitation, as well as producing short and long term forecasting of technical (service life) and economical (optimal budgeting) consequences of alternative decisions regarding repair and rehabilitation. In short DANBRO cover:

- Recording of bridge data (inventory, damage conditions and rating);
- Priorities and budgeting of maintenance and rehabilitation works;
- Management of maintenance and rehabilitation works.

All data necessary for running the system are stored in the database. Inventories assure that data in the database are updated when a bridge has been completed or remedial works have been carried out. The systems find out which sort of inspection is to be performed and the optimum time between the inspections. For each bridge ranking points are calculated and a priority list for remedial works is formed. The overall budget requirements are calculated and adjusted to budget limitations. Finally the maintenance strategies are optimized for the individual bridge. Manuals for all the activities standardize the inspections and the damage indication, so the results are coherent from bridge to bridge.

Among the reviewed bridge management systems in the Nordic countries the Danish system DANBRO seems to be the only advanced system really in a working state today. It is apparently the only systems really in a working state today. It is apparently the only system, which is in a commercialized form. DANBRO is useful in decision-making, but the ranking of the bridges is based on deterministic analysis methods, i.e. the system is not reliability based.

Sweden - The Swedish National Road Administration is responsible for some 11000 bridges. Within the Administration a new aid to systematic administration of the actions required to manage the bridges in an optimum manner is currently being developed. The system is being build up around a database containing a large quantity of data necessary for the bridge management system. The database currently consists of five components, and two new are being planned:

- Drawing section
- Administrative section
- Technical section
- Load-carrying capacity section
- Damage section
- Planning section (under planning)

- Projecting section (under planning)

The bridge management system is planned to include routines for:

- Inspections, condition assessment, load-carrying capacity classification
- Selection of planned action, optimization per bridge
- Prioritization, optimization per road network
- Specification of commonly performed maintenance and minor repair tasks
- Economic and technical follow-up
- Reporting
- Route finding for heavy transports.

Finland - The Finnish Roads and Waterways Administration is responsible for almost 12000 bridges. A database of bridge information has been used for several years. The data mainly consists of administrative and simple description information. The system is currently being revised. A two-level bridge infrastructure improvement management is being developed, the two levels are:

- Network level. The management system defines the optimum condition level of the bridge level by using a cost-benefit analysis. This analysis also gives the optimum condition level for each bridge.
- Project level. The management system assists the bridge engineers in planning works and making action schedules for a given bridge. The system can immediately show the engineer what effects adding, removing or changing an action will have on costs and condition.

The new management system necessitates the development of a new database of bridge information. In order to support an effective deterioration modeling and optimization capability the bridge management system requires a more thorough bridge inspection system, including the collection of specific damage conditions data. The bridge management system works in conjunction with two other modules: a bridge inspection system and a bridge directory.

3.3. Development of Optimal Strategies

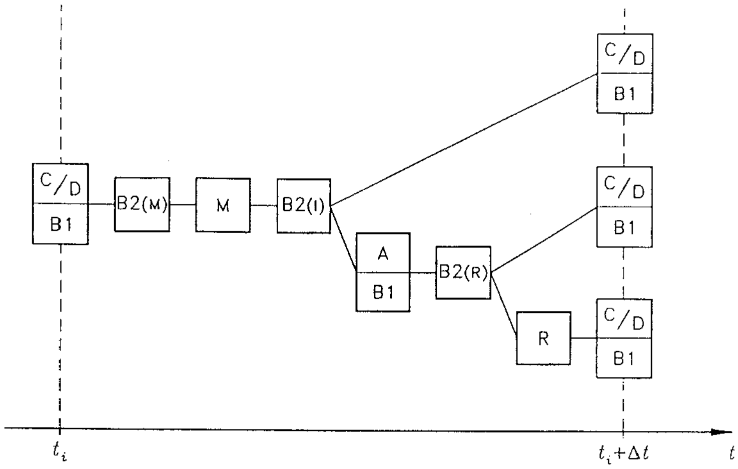
Methods and computer programs for determining rational inspection and maintenance strategies for concrete bridges are developed. The optimal decision is based on the expected benefits and total cost of inspection, repair, maintenance and complete or partial failure of the bridge. Further, the reliability has to be acceptable during the expected lifetime.

The final modelling of inspection strategy and decision system for concrete bridges is formulated. Inspections of bridges are divided in three types:

- *Current inspections*, which are performed with a fixed time interval, e.g. 15 months. The inspection is mainly a visual inspection.
- *Detailed inspections* are also periodical with a fixed time interval, which is a multiplum of the current inspection time interval, e.g. 5 years (replacing the current inspection when it occurs). The detailed inspections are also visual inspections. The inspections can also include non-destructive in situ tests.
- *Structural assessments* are only performed when a current or detailed inspection shows some serious defects, which require a more detailed investigation. Structural assessments are thus not periodical inspections. The structural assessment can include laboratorial tests, in situ tests with non-portable equipment, static and dynamic load tests. The tests are usually very costly compared with the other two inspection types. A structural assessment can also be performed when changes in the use of the bridge are being planned.

The part of the decision system, which is used to assist in maintenance and repair planning, is divided in two subsystems:

- The *maintenance subsystem* deals with maintenance repair techniques and small repair, i.e. repair of unimportant structural defects (either because its repair does not involve great sums of money or because no expert advice is needed to repair them). Generally this subsystem is always used after a current or detailed inspection.



- The *repair subsystem* helps in choosing the best option of structural repair when an important deficiency that impairs the functionality of the bridge is detected. It is basically an economic decision (based obviously on structural and traffic engineering data) in which the costs are quantified. Generally this subsystem is used after a structural assessment.

The application of the expert system modules BRIDGE1 and BRIDGE2 in the general inspection, maintenance and repair model from inspection no i at time t_i to inspection no $i+1$ at time t_{i+1} is shown in the figure, where the symbols used are: C/D is current or detailed inspection, A is structural assessment, M is maintenance work and repair of minor defects, R is repair, B1 is use of the expert system module BRIDGE1, B2 is use of the expert system module BRIDGE2. The following sub modules can be used in BRIDGE2: B2(M) is the maintenance/small repair submodule BRIDGE2(M), B2(I) is the inspection strategy submodule BRIDGE2(I) and B2(R) is the repair submodule BRIDGE2(R). Δt is the time between the periodic inspections.

After a current or detailed inspection there are two possibilities: the next inspection after Δt years is a current or a detailed inspection according to the inspection plan or the next inspection is a structural assessment to be performed immediately after the periodic inspection. The quality control inspection after a repair is not included in the modelling. After the structural assessment the repair decision is taken.

The main functions of the expert system module BRIDGE2 are:

- After a *current or detailed inspection* maintenance work is planned by the submodule *BRIDGE2(M)*. The decision is based on a classification of the defects based on three factors: rehabilitation urgency, structural importance and affected traffic. According to the inspector's experience and some pre-fixed rules each defect is given a classification, which corresponds to a global number of deficiency points.
- After a *current or detailed inspection* it is decided if a structural assessment has to be performed before the next periodic inspection by the submodule *BRIDGE2(I)*. The decision is based partly on estimates of the reliability of the bridge and partly on expert knowledge. The decision does generally not include economic considerations. The (updated) reliability of the bridge is estimated using the FORTRAN module RELIAB developed in task 2.5 and 2.6.
- After a *structural assessment* the submodule *BRIDGE2(R)* is used to decide if repair work has to be performed and the time for the repair. The decision is partly based on expert knowledge and partly on a cost-based optimization where different repair possibilities and no repair are compared. The cost-based optimization is performed using the FORTRAN program INSPEC.

The FORTRAN program INSPEC (optimal repair planning for concrete bridges) can be used estimate the optimal repair time and number of repairs for a given repair method. After a structural assessment at time T_0 the problem is to decide if the bridge should be repaired and the time of repair. Solution of this optimization problem requires that all future inspections and repairs are taken into account. However, the numerical calculations can become very time consuming. Therefore, some approximations are introduced:

- After each structural assessment the total expected benefits minus expected repair and failure costs in the remaining lifetime of the bridge are maximized considering only the repair events in the remaining lifetime.
- It is assumed that N_R repairs of the same type I_R are performed in the remaining lifetime. The first repair is performed at time T_{R_1} and the remaining is performed at equidistant times with 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.

The above decision model can be used in an adaptive way if the stochastic model is updated after each structural assessment or repair and a new optimal repair decision is taken. Therefore, it is mainly the time and type of the first repair after a structural assessment, which is of importance.

In order to decide which repair type (including no repair) and repair time to choose after a structural assessment the following optimization problem is considered with the optimization variables:

- The type of repair I_R (including no repair) to be selected
- The time T_{R_1} of the first repair
- The total number of repairs N_R in the remaining lifetime of the bridge.

$$\max_{I_R, T_{R_1}, N_R} \quad C_T(I_R, T_{R_1}, N_R) = B(I_R, T_{R_1}, N_R) - C_R(I_R, T_{R_1}, N_R) - C_F(I_R, T_{R_1}, N_R)$$

$$\text{s.t.} \quad \beta^U(T_L, I_R, T_{R_1}, N_R) \geq \beta^{\min}$$

where C_T is the total expected benefits minus costs in the remaining lifetime of the bridge. B is the expected benefits in the remaining lifetime of the bridge. C_R is the expected repair cost in the remaining lifetime of the bridge. C_F is the expected failure cost in the remaining lifetime of the bridge. T_L is the year at the end of the expected lifetime of the bridge. β^U is the updated reliability index (estimated by using RELIAB). $\beta^{\min}$ is the minimum acceptable reliability index for the bridge (related to critical elements or to the total system). The mixed integer and real variable optimization problem formulated above is solved sequentially by fixing the integer variables.

The stochastic variables used in INSPEC (different from stochastic variables 1–22 described for RELIAB) are listed in the following table, where the description, distribution type and coefficient of variation are shown for each variable.

The correlation structure for the stochastic variables (the stochastic variables numbered 1–22 are described for RELIAB in task 2.6) are modelled in the following way (the correlation structure for the stochastic variables in RELIAB is also used):

- All the stochastic variables 1–34 are assumed to be independent.
- When a repair is performed some of the stochastic variables 23–33 are used to model the repair. It is assumed that these stochastic variables are independent with respect to the other stochastic variables and independent from repair to repair.
- To model the half-cell potential inspection method an auxiliary stochastic variable (numbered 34) is needed.

No.	Mean value of...	Distribution type	Coefficient of variation
23	repair strength uncertainty	Normal	①
24	chloride diffusion coefficient in concrete for the repaired element	Lognormal	0.20
25	coefficient rate of carbonation for the repaired element	Normal	0.20
26	rate of corrosion of reinforcement for the repaired element	Normal	0.10
27	distance from underside of beam to center of reinforcement in layer 1 for repaired element	Normal	0.10
28	height of beam for repaired element	Normal	0.03
29	diameter of bars in layer 1 for repaired element	Normal	0.02
30	width of column for repaired element	Normal	0.05
31	depth of column for repaired element	Normal	0.05
32	diameter of bars in column for repaired element	Normal	0.05
33	chloride concentration on the concrete surface for repaired element	Normal	0.20
34	auxiliary stochastic variable	—	—

① The repair strength uncertainty is dependent of the type of repair.

Discussion: Inspections of bridges are divided in three types: current inspections, detailed inspections and structural assessments. The main submodules of BRIDGE2 are:

- BRIDGE2(M): Rating of defects related to maintenance. This module is used after a current or detailed inspection.
- BRIDGE2(I): Decision regarding necessity of structural assessment. This module is used after a current or detailed inspection.
- BRIDGE2(R): Optimization of type of repair and time of repair. This module is used after a structural assessment.

A FORTRAN program INSPEC is integrated with BRIDGE2 in the submodule BRIDGE2(R).

3.4. Application to Bridges

The objective of task 3.4 is to apply the expert system to real bridges. For this testing it was necessary to select bridges where the following aspects should be considered:

- To use current bridges like short and medium span overpasses;
- To understand well the bridge behaviour, what means, a good construction control and to test the bridge and compare results with numerical methods;
- To inspect it periodically to control the deterioration aspects.

Several bridges were selected with some of the above aspects, and case studies were performed to do their analysis using BRIDGE1 and BRIDGE2.

The main result of this task was the evaluation of the functionality of the expert system, with real bridges. Two Portuguese and two Danish reinforced concrete bridges were chosen to check the expert system.

At the beginning of the Project a small Portuguese bridge, the ALOMBADA Bridge, built with precast girders was selected. This type of bridges has been largely employed, especially for short and medium span viaducts or overpasses. Basically, these solutions consist of, after placing the precast girders, in situ building a deck continuous slab and achieving longitudinal continuity of the girders over supports. The slab is usually in situ concreted over precast slab elements, which lay on the precast girders.

The advantages of this bridge for the BRITE Project arise from the fact that its construction was controlled, the bridge was fully instrumented, and load tests were performed to analyse its structural behaviour. The bridge was built in 1990, and since then is periodically inspected for deterioration purpose. The bridge is not expected to have important bridge deterioration problems during the three years of the BRITE project, but as it is well studied, it will allow the development of several simulations of the Expert System and it will use the BRITE Project, along the years, as a prototype.

The other selected Portuguese bridge is an old reinforced concrete arch structure, Viaduto do Arco do Carvalão, built in 1940, presently with significant corrosion problems. Here several tests, included in a structural assessment were performed, and the results were used to check BRIDGE1 and BRIDGE2. At this stage the inspection recommendations obtained within BRIDGE1 were quite satisfactory.

One of the Danish selected bridges is a beam-slab bridge build in 1921 and enlarged in 1936 to double width of 9.5 m. The bridge is a three span structure with a total length of 33 m. The superstructure is supported at the ends and by two intermediate columns. Information about the bridge is based on an inspection report from a structural assessment made in 1988. During the inspection severe reinforcement corrosion is observed. The main cause of corrosion is carbonization. The chloride content in the bridge is not serious. This bridge is mainly used to check the FORTRAN programs developed in task 2.5, 2.6 and 3.3.

The other selected Danish bridge is a beam-slab bridge built in 1945. In 1962 a complete overhaul of the bridge was performed. The total length of the bridge is 26 m; the width of the bridge is 15 m. The superstructure is supported at the ends and by one intermediate column. The column cannot be analyzed by the expert system due to the materials used. Information about the bridge is based on an inspection report from a structural assessment made in 1988. During the inspection severe reinforcement corrosion is observed. This bridge is used to check the expert system modules BRIDGE1 and BRIDGE2 and the FORTRAN programs called within BRIDGE2.

Further, during checking the FORTRAN programs the variables important for the reliability index are identified. Sensitivity analysis with regard to statistical parameters for the identified important variables is performed.

3.5. Main Results and Conclusions

The main achievements of this subproject are the definition of the optimal strategies for inspection, maintenance and repair of concrete bridges.

The inspection techniques are characterized with a proposed rating and correlated with the defects. This correlation was implemented in the expert system BRIDGE1 as a module to help the inspectors, suggesting the best technique to perform the inspection of the defect. An error analysis of the methods was also performed in relation to the reliability analysis.

The evaluation of existing bridge management systems allowed obtaining all the relevant information in an organized and succinct way. This review of the management systems of several countries in Europe and America provided some guidelines to the implementation of the expert system.

The global functionality of the system is defined in this subproject, namely the inspection strategy, and the maintenance and repair sub modules. The organization and interconnection between BRIDGE1 and BRIDGE2 was also defined here.

Within this subproject the expert system is also applied to real bridges. Several bridges were selected with some deterioration problems and case studies were performed to do their analysis using BRIDGE1 and BRIDGE2. This allowed the evaluation of the functionality of the expert system, with real bridges.

4. EXPERT SYSTEM FOR IDENTIFICATION OF DETERIORATION OF CONCRETE STRUCTURES

Expert Systems technology is nowadays being considered as a powerful mechanism for helping human experts in their everyday decision tasks. Being able to represent into the computer system the knowledge structures and reasoning strategies that the human expert follows when approaching a problem, enables other users to share this knowledge and the expert system thus constructed establishes a common decision criteria for the perspective users of the system.

The objective of this subproject was to develop, using expert system technology, a software tool to aid bridge inspectors as well as engineer experts in their tasks of assessing and improving the reliability of concrete bridges. In this subproject the research results obtained in the other subprojects were structured and introduced in a proper way into the computer, as well as the inspection and repair strategies developed along the duration of the project.

4.1. Definition of General Architecture

The objectives of this task are to identify the various software subsystems and the relations among them, that is, the software architecture, that will set the basis for the development of the expert systems. But previous to the task of the design itself it was considered the need to study the state of the art of these kinds of systems.

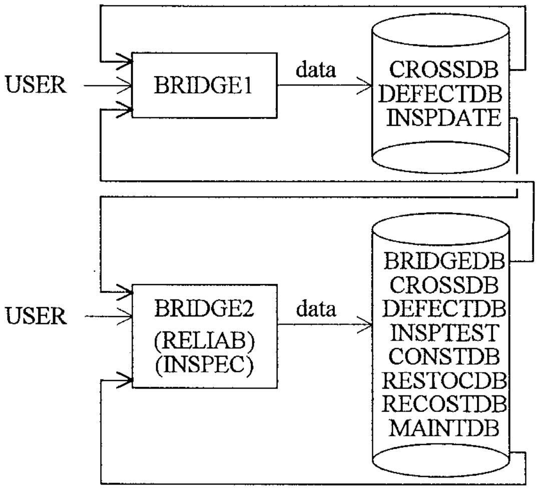
A study of the knowledge-based systems in is carried out and presented, were the state of the art of this kind of systems is mainly analyzed and described.

It was early detected, in the activity of the design, the need to develop two different modules, so called BRIDGE1 and BRIDGE2, aimed at different goals. Whilst BRIDGE1 was conceived to provide technical support to the inspector during the inspection process at the bridge site, BRIDGE2 was designed to assist the engineer both in the analysis of the safety of the concrete bridges, as well as in the selection of maintenance and repair methods.

For this purpose an initial external architecture design based on experience with a previous BRITE project was presented, in which it was outlined the relations of the two modules with the FORTRAN functions as well as with some initial databases. Further, good functional ideas were provided. Taking these suggestions into account as well as other ideas provided by all partners in the initial technical meetings of the project, what would be the basis of the subsequent designs that followed the development of the project was designed and presented. Thus, the architecture has improved on the basis of this design, which was approved by all the partners.

In this task a set of FORTRAN programs was defined. They interact with the Expert Systems through specific designed data files, and they are as follows:

- Updating analysis: On bases of inspection information and other new information the reliability estimates and the data in the databases will be updated.
- Reliability analysis: The reliability of the bridge will be evaluated as function of time.
- Structural analysis: There will not be developed a structural analysis program in this project. The user is supposed to supply the finite element program.
- Inspection program: On bases of the data in the databases and the reliability estimations the optimal time for the next inspection is calculated using the updating module.



The identification of the different components that constitute the expert systems helped towards the establishment of the final functionality to incorporate into the expert systems. The final architectural design is here presented:

Only the databases that can be changed by input from the user to the expert systems are shown in the figure. The FORTRAN programs RELIAB and INSPEC are called by BRIDGE2.

4.2. Selection of Knowledge Mechanisms and Software Tools

The objectives in this task are to identify the representation schemes and inference mechanisms best suited for the implementation of the expert systems, as well as the evaluation and selection of the most promising available expert system shells that would guarantee that the representation and inference requirements identified are fulfilled.

In order to satisfy these goals, an exhaustive market analysis through the available expert system shells was carried out, and the results obtained are collected and analyzed in a report. Another expert system tool, distributed by Exsys, were suggested, but no final contact was possible to perform on this company.

As a result of the market analysis, the use of a particular tool, ADS/PC, was suggested. The features of this tool that made it appropriate for the problem at hand were also described. The choice was mainly due to the great variety of representational schemes as well as inference mechanisms that, among other important features, the software tool has. The rest of the partners approved the decision.

The functional interrelations between BRIDGE2 and the FORTRAN programs were presented, that can be described as follows: After an inspection the engineer can use BRIDGE2 to estimate the updated reliability index computed by the FORTRAN program RELIAB. This function was developed in task 2.5 and 2.6. The plan for other FORTRAN programs was also included, that based on the data of the databases and the reliability estimations the optimal time for the next inspection was to be calculated.

A very extensive market analysis was carried out in this task which helped us to choose the most appropriate expert system tool for the development of the project. Apart from the functionality criteria of the tool other aspects were taken into account, as they are the availability of affordable runtime versions, technical support and a well know standard of the tool.

4.3. Implementation of Prototype Expert System

4.3.1 Introduction

This task has certainly been the most important and difficult of subproject 4. The main difficulty was due to the fact that the research in the other subprojects was performed simultaneously, what did not allow a deep understanding of the problem at hand from the very beginning. This resulted in a set of prototypes, which represented each of the states that the whole project went through.

Although a decision was taken in task 4.1 regarding the implementation of two different expert systems BRIDGE1 and BRIDGE2 and the analysis of specifications was sketched from the very beginning, the implementation task was initiated with BRIDGE1, the module that provides support to the inspector at the bridge site.

In this task the objective was to represent the expert knowledge and strategies using the expert system tool selected in task 4.2.

The first prototype for BRIDGE1 was presented in June 1991, at the end of the first year of work. The main purpose was to implement the correlation matrices. The correlation matrices were defined for: defects/diagnosis methods, defects/causes and defects/repair methods. A pseudo-quantitative classification of the type no correlation, low and high correlation was proposed. The correlation between defects and both diagnosis and repair methods was presented. Each matrix is organized so that each line represents a defect and each column a possible diagnosis/method, cause or repair method. In the intersection of each line and column a number representing the correlation between defect and possible element of reference is to be introduced. The criteria adopted for that number was:

0 No correlation: No correlation between the defect and the element of reference.

1 Low correlation: The diagnosis method, repair method can be used for that defect, although they are not the best ones. For the case of the matrix of causes, it represents an indirect cause of the defect connected only with the very early stages of the deteriorated process and not necessary for its development.

2 High correlation: Diagnosis and repair methods most appropriated for the defect. For the correlation matrix of causes, it represents a direct cause of the defect associated with the final stages of the deterioration process. When the cause occurs, it is one of the main causes of the deterioration process and is indispensable to its development.

In this prototype it was also shown the possibility of integrating the FORTRAN programs with the Expert Systems.

It was proposed that BRIDGE1 would be divided in five main blocks: general information about the bridge, related diagnosis methods, probable causes, associated defects and provisional defect report. This initial internal structure was enriched by many opinions from all of the partners. These ideas were collected and implemented in a prototype that certainly set the basis for the actual version of BRIDGE1. What the graphic data would be for concrete bridges was also described.

A crucial task in the development of the expert systems was the definition of the databases. In this regard, a view of how a concrete bridge database should be organized and which should be the main characteristics was presented. Further an exhaustive study of the data to be collected for concrete bridges, both at the design stage and after it has been built was provided. At relevant moments of the bridge's service life (usually after it is built and after important rehabilitation work is performed), its real situation must be thoroughly described so that future inspections have something to relate to. Such information is stored in this file from which only those parameters that would be used by the internals of the expert systems were selected. This database definition was presented, and the set of parameters required for the reliability estimation in RELIAB, the cost optimization in INSPEC, additional bridge parameters dealing with the bridge repair cost and corrosion descriptive parameters were added. This bridge database BRIDGEDB.DBF also improved with the different prototypes.

An innovative concept from the point of view of the description of the data of the bridges was introduced, and it is the idea of cross-section. A cross-section permits to describe in greater detail the concrete and bridge characteristics at different points of the bridge structure. We identified two types of cross-section related to the deck and columns of the bridge, and for each of them the data needed to describe each of the cross-sections is different. Cross-sections are used to:

- Better describe the concrete changes/replacements/deteriorations at different parts of the bridge.
- Locate in the bridge the detected defects.
- Estimate the reliability index at each of the cross sections, and by means of a combination of the indices, estimate the reliability index of the whole bridge.

The user can define as many cross-sections as he/she requires that shall be recorded in the database CROSSDB.DBF.

The communication between BRIDGE1 and BRIDGE2 is done through the data, and in particular, through the DEFECTDB.DBF database that stores the provisional defect report entered via BRIDGE1. The main element in this database is the defect and for each of the detected defects in the bridge inspection, a record is generated and stored. The definition of the database was performed taking into account all the data requirements.

Regarding BRIDGE2, the initial architecture that this expert system would implement was described where the following modules were identified:

- Database
- Inspection Module
- Decision Module

The decision module is divided in three sub-modules:

- Maintenance/Small repair sub-module
- Inspection strategy sub-module
- Repair/Upgrading/Replacement sub-module

A first prototype for BRIDGE2 was developed based on the contents of this report and ideas raised in the technical meetings. This, as well as the set of evolving prototypes that have been implemented along the duration of the project, were presented in the technical meetings and were very valuable to improve and refine them. All partners collaborated in this process. In this way, the specifications were completed.

It was agreed by all partners in a first version of the expert systems development to concentrate in the following six typical corrosion related defects: rust stain, delamination/spalling, crack over/under a bar, exposed bar, corroded bar and bar with reduced cross-section. At a latter stage in the development process of the expert systems all the defects were introduced into the system.

The expert systems implement the following strategies:

- Should technical knowledge regarding the need to perform a structural assessment be incorporated into the system and would it also be used to double-check when the reliability index estimates that the bridge is in good condition? Elicitation of expert knowledge was performed and a set of decision rules that were integrated into the expert system was formulated.
- When defects are detected in an inspection, what should be the strategy to consider them either maintenance or repair? When is the most appropriate time to repair the defect? We based the strategy to consider a defect either maintenance or repair on a well-established defects' classification scheme, based on the experience and knowledge collected from technical experts. The procedure for maintenance defects is under the BRIDGE2(M) module, and BRIDGE2(R) analyses defects that require repair. Nevertheless, some defects, depending on their degree of deterioration or on the location in the bridge can be considered of the both types. In this first version the user will decide to consider the defect as maintenance or repair. It is an open issue for future development of the project to incorporate the decision criteria that an expert in this matter will follow to decide in the classification.
- How should costs be estimated? The approach followed to implement this feature into BRIDGE2 was the result of many different proposals from all the partners. Parameters that for each defect could be used to describe the defect in greater detail as well as to compute the repair costs were identified and a cost function was designed.

We now summarize the functionalities that have been implemented into the expert systems developed in this task. With regard to BRIDGE1, the inspector can perform the following activities:

- Review all the information contained in the database of the bridges. Different type of data is recorded for each bridge: identification and bridge site information, design information, budget information, traffic information, strength information, load information, deterioration information, factors that model the costs and data for the cross-sections defined for the bridge.
- Define new cross-sections.
- Receive technical support regarding the most appropriate diagnosis methods to be used in order to conclude about the existence of a defect.
- Receive technical support regarding the possible causes responsible for a defect.
- Record the results of the inspection (defects and their associated diagnosis methods and causes, as well as other important information to be used in the analysis of the bridge by BRIDGE2).

A user manual was initially written, but a second version incorporating a complete description of the data is under development.

The engineer at the office can use BRIDGE2 to:

- View the inspection results recorded at any previous inspection performed in any of the bridges of the database.
- Enter the data of a bridge in the bridge's database.
- View the data of a bridge and edit it.
- Define new cross-sections for any of the bridges in the database.
- Get a relation of the set of bridges contained in the database with the next inspection dates for each of the bridges.
- Complete the data of the defects detected at the inspection, by describing the defect in greater detail and by entering the results of the tests performed on the concrete.

Two sub modules of BRIDGE2, BRIDGE2(M) and BRIDGE2(R) provide the functionalities for the maintenance and repair activities.

With BRIDGE2(M) the engineer is offered with:

- A rating of the defects related to the maintenance subsystem.
- The relevant maintenance techniques appropriate for the defects related to the maintenance subsystem. To identify the relevant maintenance techniques, the correlation matrix between defects and maintenance techniques is used.
- An estimation of the maintenance costs.

With BRIDGE2(R) the engineer can:

- Find the relevant structural repair techniques. Identification of relevant structural repair techniques for a defect is based on flowcharts.
- Obtain an optimization of repair plan. The FORTRAN program INSPEC is used to estimate the optimal method and time for repair, the optimal number of repairs in the remaining lifetime of the bridge and the expected benefits minus costs. INSPEC is based on the cost-benefit analysis.

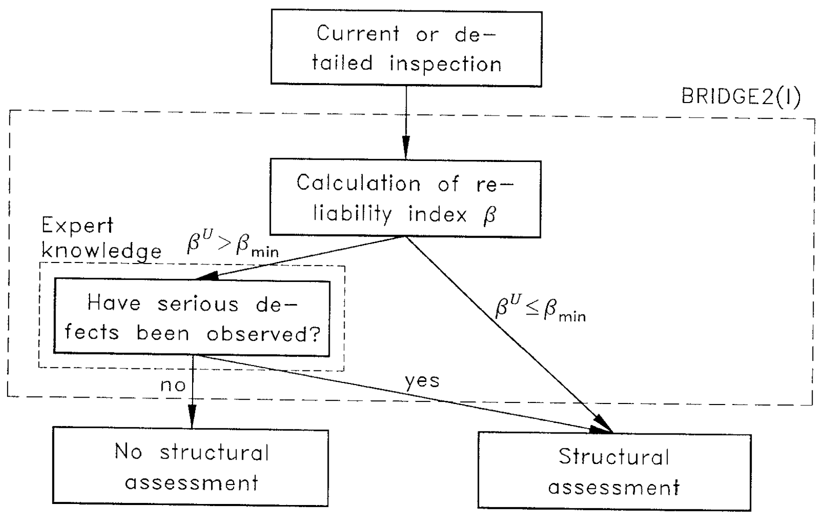
The module BRIDGE2(I) interacts with the FORTRAN function RELIAB and presents to the user the updated reliability index estimated by the function. If the updated reliability index is smaller or equal to a minimum fixed value, a structural assessment should be performed in the bridge before the next current or detailed inspection. But if the estimated index is greater a structural assessment should be performed if from a structural point of view one or more serious defects have been selected.

A matrix of local repair tradition in the repair module BRIDGE2(R) was proposed to be included. This matrix is a correlation matrix between defects and the correlated structural repair techniques and it was used to rate the relevant structural repair techniques.

BRIDGE2 is, as well as BRIDGE1, very easy to use, as both were developed with the aim of being friendly systems.

The computer design and implementation of the expert systems have resulted in a number of prototypes of the expert systems. Each of the prototypes produced has been tested and all partners have provided comments and suggestions for improvement of the system. The contributions were merged and the final changes were described in a number of reports with overview of the unclear details related to the expert systems and knowledge elicitation that were discussed at the meetings. They were useful for the detection of difficulties and helped to focus the efforts of the partners in a fruitful way.

4.3.2 Decision model regarding structural assessment



The decision model in BRIDGE 2(1).

A structural assessment is recommended if the updated reliability index for the bridge β^U is smaller than or equal to a minimum reliability index β_{min}. If the updated reliability index for the bridge is greater than the minimum reliability index then the decision is taken based on expert knowledge. The decision model in BRIDGE 2(1) is:

- Execution of the reliability program RELIAB
- If $\beta^U \leq \beta_{\text{min}}$ then a structural assessment is recommended before the next periodic inspection. In this case the decision is taken solely on basis of the reliability index - no expert knowledge is used.
- If β^U then a structural assessment is recommended if from a structural point of view one or more serious defects have been detected at the inspection.

4.3.3 *Elicitation results*

In this section the decisions resulting in *Structural assessment* or *No structural assessment* are described for the most important defects. The presentation includes questions to be asked to the user and possible answers to these questions ("resulting in" is symbolized by $\Rightarrow$).

4.3.3.1 Rust stain

If rust stain is observed at the inspection then the following question is asked:

Question 1: *What is the extent of rust stain?*

Possible answers to question 1:

1. Single rust stains $\Rightarrow$ No structural assessment (it is assumed that single rust stains do not question the structural safety or the global functionality of the bridge)
2. Locally many rust stains $\Rightarrow$ question 2

3. Widespread rust stains ⇒ Structural assessment (it is assumed that there is a global corrosion of the reinforcement in the bridge)

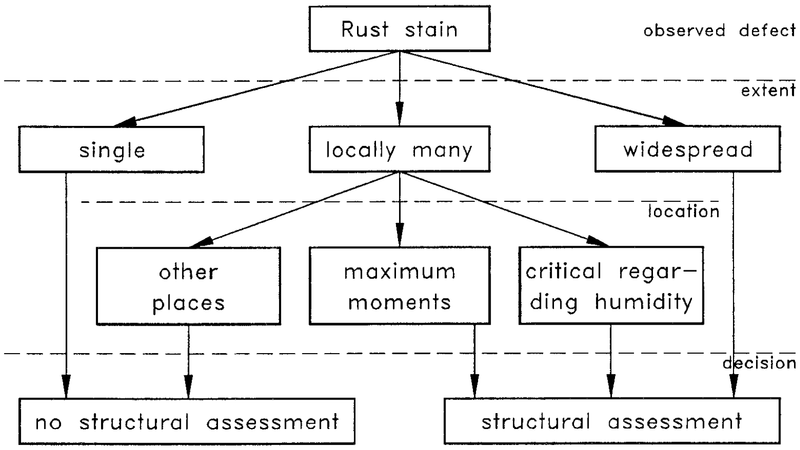
If item 2 is the result of question 1 then question 2 is asked.

Question 2: *What is the location of rust stains?*

Possible answers to question 2:

1. A critical place regarding humidity ⇒ Structural assessment (a place is critical if it e.g. is exposed to splash of water from cars passing under the bridge)
2. Near places where maximum moments occur ⇒ Structural assessment
3. Other places ⇒ No structural assessment

The result of the elicitation is illustrated in the figure.



4.3.3.2 Delamination/spalling

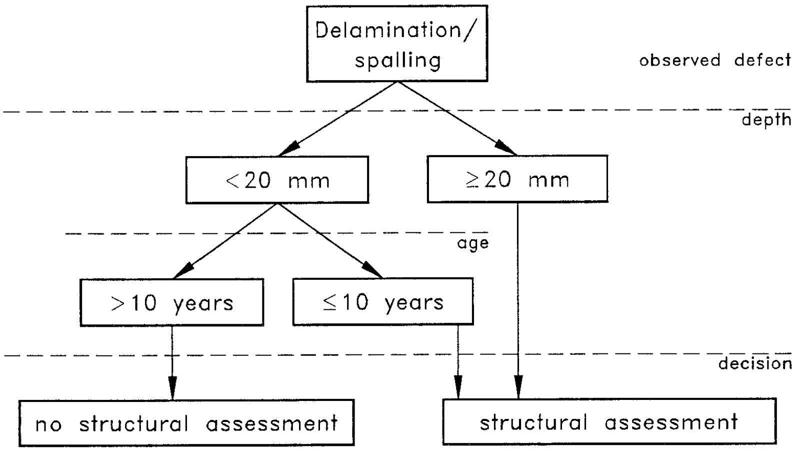
If delamination/ spalling is observed at the inspection then the following question is asked:

Question 3: *What is the depth of delamination/spalling?*

Possible answers to question 3:

1. Delamination/ spalling is on the surface of the concrete (depth of delamination/spalling is less than the concrete cover say 20 mm). The age of the structure is calculated by data in the database. There are then two possibilities:
   (a) The structure is relative new (10 years or less) ⇒ Structural assessment (if there is delamination/spalling of the cover of a relative new structure then something serious is wrong)
   (b) The structure is relative old (more than 10 years) ⇒ No structural assessment
2. Delamination/ spalling is deep (depth of delamination/ spalling is deeper than the concrete cover 20 mm or more) ⇒ Structural assessment

The result of the elicitation is



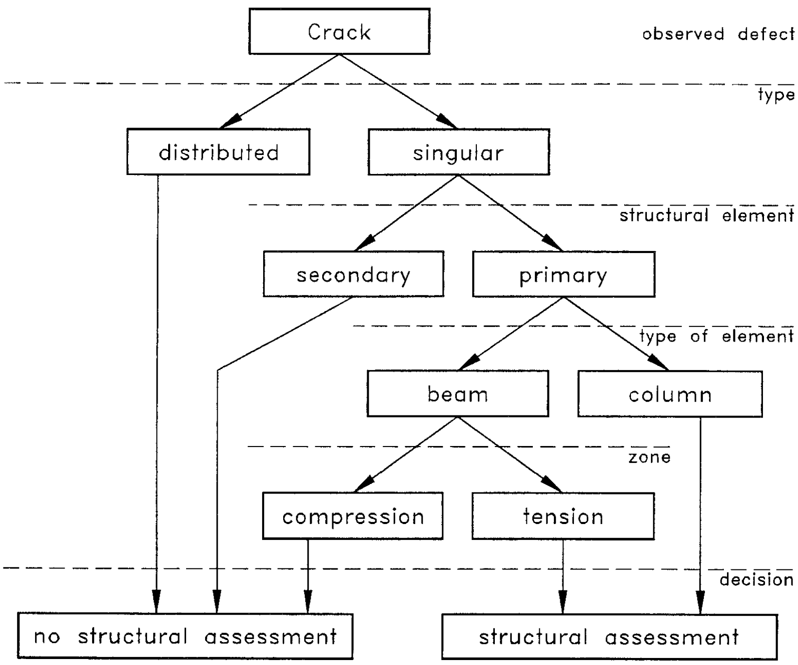
illustrated in the figure.

4.3.3.3 Crack over/under bar

If cracks over/under reinforcement bars are observed at the inspection then the following question is asked:

Question 4: *What is the type of cracks?*

Possible answers to question 4:
1. Singular cracks (due to corrosion and/or loading) ⇒ question 5
2. Distributed cracks (due to shrinkage and/or creep) ⇒ No structural assessment



If item 1 is the result of question 4 then question 5 is asked.

Question 5: *What is the importance of the structural element?*

Possible answers to question 5:
1. Primary structural element (girder or column) ⇒ question 6
2. Secondary structural element (other like e.g. an edge beam) ⇒ No structural assessment

If item 1 is the result of question 5 then question 6 is asked.

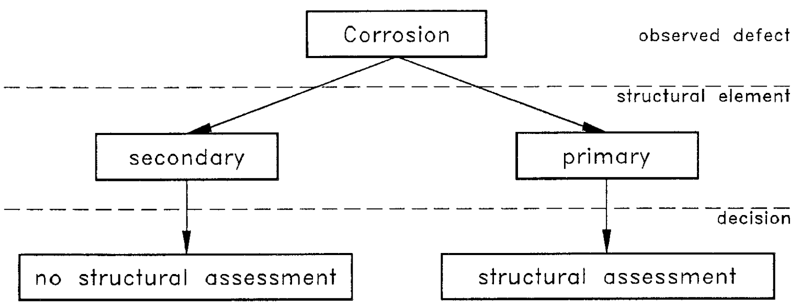
Question 6: *What is the type of the structural element?*

Possible answers to question 6:
1. Beam ⇒ question 7
2. Column ⇒ Structural assessment

If item 1 is the result of question 6 then question 7 is asked.

Question 7: *In what type of zone are cracks observed?*

Possible answers to question 7:
1. Tension zone (near primary reinforcement) ⇒ Structural assessment
2. Compression zone (near secondary reinforcement) ⇒ No structural assessment. The result of the elicitation is illustrated in the figure.



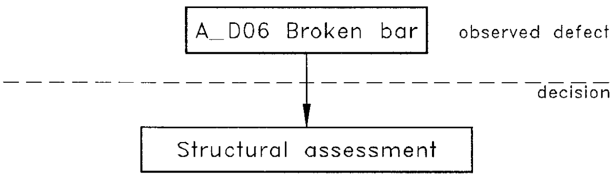
4.3.3.4 Direct visual observation of corrosion

If corrosion is observed visually at the inspection then the following question is asked:

Question 8: *Mat is the importance of the structural element?*

Possible answers to question 8:
1. Primary structural element (girder or column) ⇒ Structural assessment
   Secondary structural element (other like e.g. an edge beam) ⇒ No structural assessment. The result of the elicitation is illustrated in the figure.

4.3.3.5 Broken bar

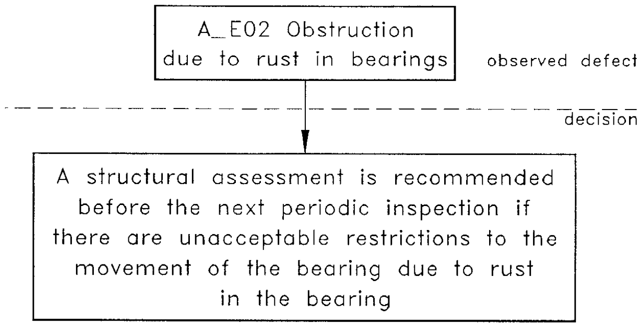


If broken bar is observed at the inspection then a structural assessment should always be used to investigate the extend of the defect and to evaluate the load capacity of the bridge. The result of the elicitation is illustrated in the figure.

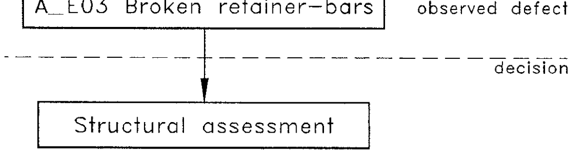
4.3.3.6 Obstruction due to rust in bearings

If obstruction due to rust in bearings is observed at the inspection then the reliability index due to the movement should be evaluated and taken into account in the reliability model of the bridge. If the updated reliability index for the bridge β^U is larger than the minimum acceptable value $\beta_{\min}$ then No structural assessment should be recommended else structural assessment should be recommended.

No failure modes related to the bearings have been formulated during the project so the following message is shown at the screen: *A structural assessment is recommended before the next periodic inspection if there are unacceptable restrictions to the movement of the bearing due to rust in the bearing.* The result of the elicitation is illustrated in the figure.



4.3.3.7 Broken retainer-bars

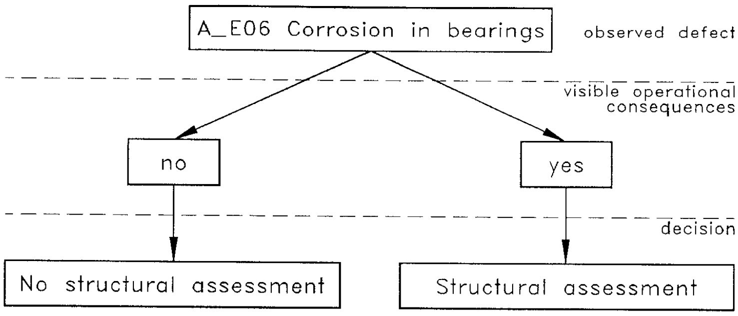


If broken retainer-bars are observed at the inspection then a structural assessment should always be performed. The result of the elicitation is illustrated in the figure.

4.3.3.8 Corrosion in bearings

If corrosion in bearings is observed at the inspection then the following question is asked:

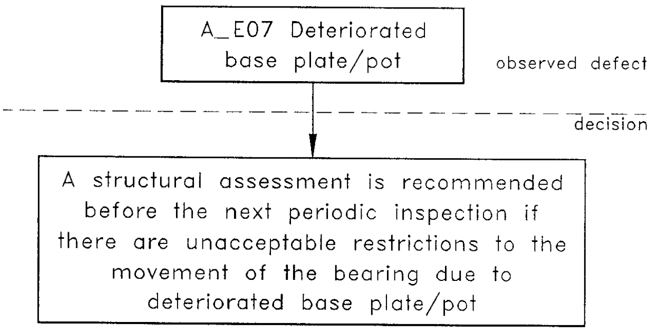
Question 9: *Are there visible operational consequences for the bearing itself or for nearby bearings?*



Possible answers to Question 9:
1. Yes ⇒ Structural assessment.
2.No ⇒ No structural assessment.
The result of the elicitation is illustrated in the figure.

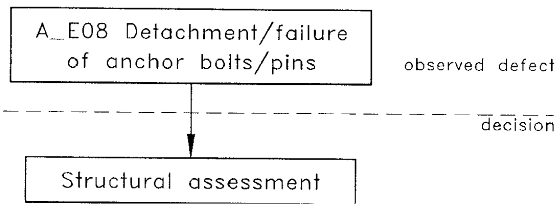
4.3.3.9 Deteriorated base plate/pot

If deteriorated base plate/pot is observed at the inspection then the reliability index due to the movement should be evaluated and taken into account j reliability model of the bridge. If the updated reliability index for the bridge β^U is larger than the minimum acceptable value $\beta_{\min}$ then no structural assessment should be recommended else structural assessment should be recommended.



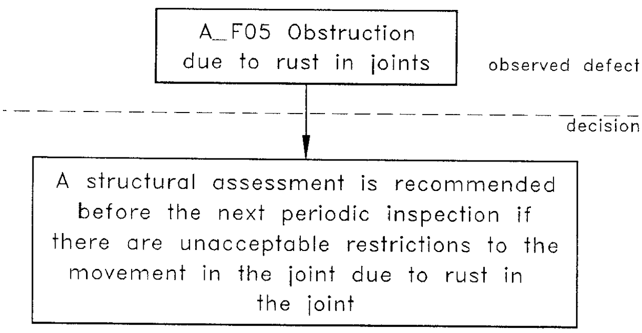
No failure modes related to the bearings have been formulated during the project so the following message is shown at the screen: *A structural assessment is recommended before the next periodic inspection if there are unacceptable restrictions to the movement of the bearing due to deteriorated base plate/pot.* The result of the elicitation is illustrated in the figure.

4.3.3.10 Detachment/failure of anchor bolts/pins



If detachment/failure of anchor bolts/pins is observed at the inspection then a structural assessment should always be performed. These kinds of defects are serious since they can affect on the static of the bridge.

4.3.3.11 Obstruction due to rust in joints



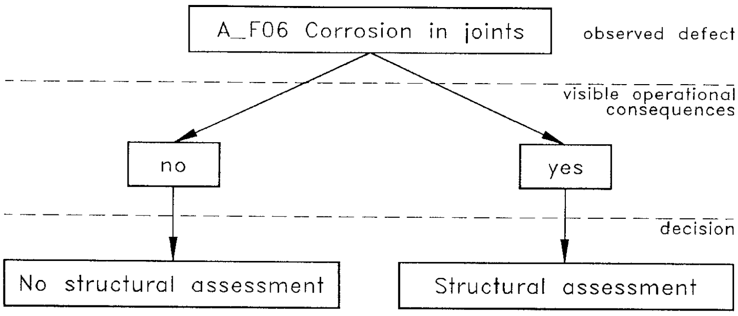
If obstruction due to rust in joints is observed at the inspection then the reliability index due to the movement should be evaluated and taken into account i reliability model of the bridge. If the updated reliability index for the bridge β^U is larger than the minimum acceptable value $\beta_{\min}$ then no structural assessment should be recommended else structural assessment should be recommended.

No failure modes related to the joints have been formulated during the project so the following message is shown at the screen: *A structural assessment is recommended before the next periodic inspection if there are unacceptable restrictions to the movement in the joint due to rust in the joint.* The result of the elicitation is illustrated in the figure.

4.3.3.12 Corrosion in joints

If corrosion in joints is observed at the inspection then the following question is asked:

Question 10: *Are there visible operational consequences for the joint itself or the nearby joints?*



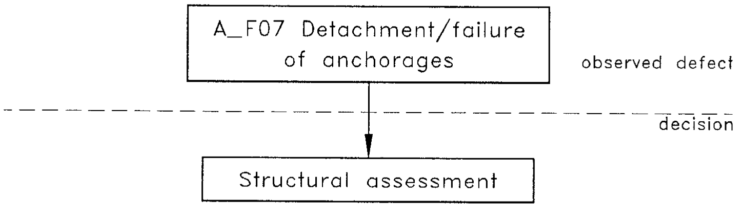
Possible answers to Question 10:

1. Yes ⇒ Structural assessment.
2. No ⇒ No structural assessment.

The result of the elicitation is illustrated in the figure.

4.3.3.13 Detachment/failure of anchorages

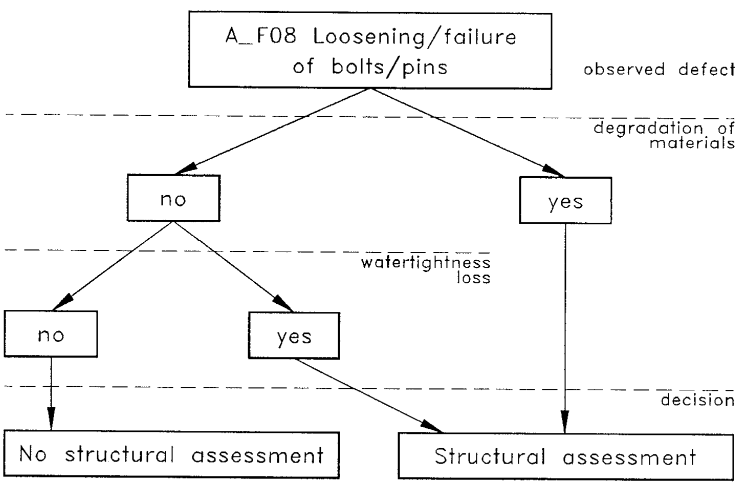
If detachment/failure of anchorages is observed at the inspection then a structural assessment should always be performed since such defects are of importance for the safety of the traffic. The result of the elicitation is illustrated in the figure.



4.3.3.14 Loosening/failure of bolts/pins

If loosening/failure of bolts/pins is observed at the inspection then question is asked:

Question 11: *Is there visible degradation of materials (organic or in the joint itself) or the bearings under the joint?*



Possible answers to Question 11:

1. Yes ⇒ Structural assessment.
2. No ⇒ Question 12 is asked.

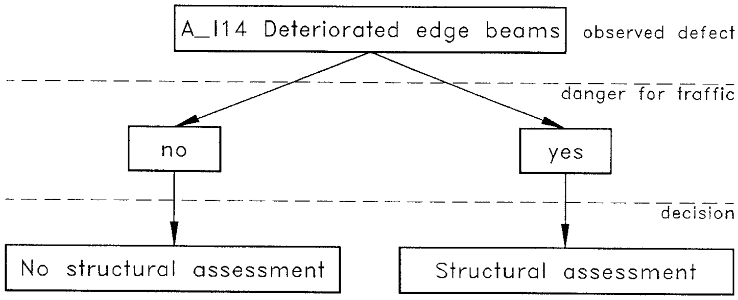
Question 12: *Is there loss of water tightness in the joint?*

Possible answers to Question 12:
1. Yes ⇒ Structural assessment.
2. No ⇒ No structural assessment.
The result of the elicitation is illustrated in the figure.

4.3.3.15 Deteriorated edge beams

If deteriorated edge beams are observed at the inspection then the following question is asked:

Question 13: *Is there danger for the traffic?*



Possible answers to question 13:
1. Yes ⇒ Structural assessment.
2. No ⇒ No structural assessment
The result of the elicitation is illustrated in the figure.

4.4. Main Results and Conclusions of Subproject 4

As a result of this subproject, two expert systems have been designed, implemented and tested by all the partners. Due to the set of prototypes that were developed along the duration of the project, many improvements were introduced into the systems, guided mainly by the objective of obtaining a useful tool for the bridge inspector and engineer. In particular a prospective user was selected that provided useful comments, mainly concerned with the adequacy of the system to the inspection, maintenance and repair strategies that they follow in their day by day basis.

Although the initial purpose of this project was to develop a prototype we obtained a piece of software that is closer to a commercial product, in particular BRIDGE1, which will only require some additions in order to consider other problems apart from the corrosion here implemented.

BRIDGE2 integrates a reasonable amount of expertise in different tasks performed by the engineer. Thus, the system evaluates a set of possible repair methods by means of a benefits minus cost analysis and suggests the best optimal repair method as well as the better time to perform it and an approximation of the real cost. This was certainly a challenge that although completely implemented needs of improvement and refinement.

The work carried out in this subproject provided all the partners with a clear understanding of the difficulties and benefits of approaching an engineer problem with expert system technology. We expect to continue the work in this area by improving the prototypes.

5. CLOSING REMARKS

The main achievements of this research project are:

- A new expert system BRIDGE 1, which can assist the inspector during the inspection on

- the site of the bridge has been developed and implemented. BRIDGE 1 will supply information on: the causes of observed defects, appropriate diagnosis methods and related defects.
- A new expert system BRIDGE2, which is used as a decision tool for the inspection engineer after the inspection is completed and after testing in the laboratories has taken place has been developed and implemented.
- A number of FORTRAN modules MISDAT, BETAEQ, RELIAB, and INSPEC has been developed and implemented on basis of existing software. Some of these modules are used in the expert systems, but can also be used independently, e.g. can RELIAB be used to estimate the reliability of any structure.
- New stochastic models for corrosion, inspection, maintenance and repair are used in the expert systems, but are also of great interest in other connections.
- New experimental results on corrosion, repair materials, strength of damaged structural elements and long time behavior of concrete structural elements have been obtained.

APPENDIX: LIST OF PUBLICATIONS

T1.1-01 *Definition of Data Needed for the FORTRAN Programmes,*
P. Thoft-Christensen & J.D. Sørensen, CSR, February 1991.

T1.1-02 *Parameters for Corrosion Models,*
M.F. Costa Montemor, M.G.S. Ferreira & A.M.P. Simões, IST, May 1991.

T1.1-03 *Definition of Data Needed for the Expert Systems,*
P.S. Mangat & M.S. Elgarf, UA, May 1991.

T1.1-04 *Definition of Data Needed for the Expert Systems,*
A.C. Spoon, JAHN, June 1991.

T1.1-05 *Definition of Data Needed for the Expert Systems,*
J.L. Ramírez & J.M. Bárcena, LABEIN, June 1991.

T1.2-01 *Durability of Maritime Structures,*
A.C. Spoon, JAHN, March 1991.

T1.2-02 *Inspection and Research Data for Concrete Bridges,*
A.C. Spoon, JAHN, March 1991.

T1.2-03 *Data from LABEIN's Pathology Activities for Statistical Analysis and Model Derivation,*
J.L. Ramírez, J.M. Bárcena, J. Urreta & J.A. Sánchez, LABEIN, May 1991.

T1.3-01 *Linear Regression with Missing Data,*
H.I. Hansen & P. Thoft-Christensen, CSR, November 1991.

T1.3-02 *Field and Laboratory Data on Reinforcement Corrosion Initiation and Propagation Parameters,*
P.S. Mangat & M.S. Elgarf, UA, November 1991.

T1.3-03 *FORTRAN Programme: MISDAT,*
H.I. Hansen & P. Thoft-Christensen, CSR, December 1991.

T1.4-01 *Modelling the Corrosion of Steel in Concrete,*
P.S. Mangat, UA, November 1990.

T1.4-02 *Corrosion of Steel in Concrete — Causes and Models,*
M.F. Costa Montemor, M.G.S. Ferreira & A.M.P. Simões, IST, November 1990.

T1.4-03 *The Effect of Reinforcement Corrosion on the Performance of Concrete Structures,*
P.S. Mangat & M.S. Elgarf, UA, May 1991.

T1.4-04 *Models for Corrosion of Steel in Concrete,*
M.F. Costa Montemor, M.G.S. Ferreira & A.M.P. Simões, IST, May 1991.

T1.4-05 *Life Prediction of Corroding Reinforced Concrete Elements (Numerical Example),*
P.S. Mangat & M.S. Elgarf, UA, September 1991.

T1.4-06 *Diffusion Coefficients of Chloride and Carbon Dioxide in Cement Pastes and Concrete,*
M.F. Costa Montemor, M.G.S. Ferreira & A.M.P. Simões, IST, January 1992.

T2.1-01 *Classification of Anomalies in Concrete Bridges,*
J. Brito, V. Lúcio & F.A. Branco, IST, November 1990.
T2.1-02 *Classification of Possible Causes of Anomalies in Concrete Bridges,*
J. Brito, V. Lúcio & F.A. Branco, IST, November 1990.
T2.1-03 *Correlation Between Corrosion Related Anomalies in Concrete Bridges and Their Causes,*
J. Brito, V. Lúcio & F.A. Branco, IST, November 1990.
T2.1-04 *Behaviour of Corroded Reinforced Concrete Members,*
J.L. Ramírez & J.M. Bárcena, LABEIN, November 1990.
T2.1-05 *Repair of Concrete Columns with Partial Localized Damages,*
J.L. Ramírez, J.M. Bárcena, J.I. Urreta, B. de Val & J.R. Aurrecoechea, LABEIN, June 1991.
T2.1-06 *Definition of Different Repair Treatments,*
A.C. Spoon, JAHN, June 1991.
T2.2-01 *Flexural Strength Analysis for a Defective Reinforced Concrete Beam,*
P.S. Mangat & M.S. Elgarf, UA, September 1991.
T2.2-02 *Performance of Corrosion Damaged Beams: Laboratory Investigation,*
P.S. Mangat & M.S. Elgarf, UA, November 1991.
T2.2-03 *Shrinkage, Swelling and Creep Deformation of Repair Materials,*
P.S. Mangat & M.K. Limbachiya, UA, August 1992.
T2.2-04 *The Performance of Corrosion Damaged Beams after Repair: Laboratory Investigation,*
P.S. Mangat & M.S. Elgarf, UA, October 1992.
T2.2-05 *Load-Strain Characteristics of Defective and Repaired Reinforced Concrete Columns,*
P.S. Mangat & M.K. Limbachiya, UA, November 1992.
T2.3-01 *Classification of Repair Techniques in Concrete Bridges,*
J. Brito, V. Lúcio & F.A. Branco, IST, November 1990.
T2.3-02 *Correlation Between Corrosion Related Anomalies in Concrete Bridges and Repair Techniques,*
J. Brito, V. Lúcio & F.A. Branco, IST, November 1990.
T2.3-03 *Proposal for the Repair File,*
V. Lúcio, J. Brito & F.A. Branco, IST, November 1990.
T2.3-04 *Repair Techniques for Corroded Concrete Bridges,*
A.C. Spoon, JAHN, November 1990.
T2.3-05 *Matrix of Correlation Between Corrosion Related Anomalies in Concrete Bridges and Their Causes,*
J. Brito, V. Lúcio & F.A. Branco, IST, January 1991.
T2.3-06 *Matrix of Correlation Between Corrosion Related Anomalies in Concrete Bridges and Repair Techniques,*
J. Brito, V. Lúcio & F.A. Branco, IST, January 1991.
T2.3-07 *Repair Costs,*
A.C. Spoon, JAHN, July 1991.
T2.3-08 *Repair Costs,*
A.C. Spoon, JAHN, November 1991.
T2.3-09 *Local Repair of Concrete Columns with Total Compressive Strength Loss,*
J.L. Ramírez, J.M. Bárcena, J. Urreta, J.A. Sánchez & B. Hernández, LABEIN, January 1992.
T2.3-10 *Materials for Concrete Repair. Typical Properties,*
P.S. Mangat & M.K. Limbachiya, UA, May 1992.
T2.3-11 *Performance of Defective and Repaired Reinforced Concrete Columns,*
P.S. Mangat & M.K. Limbachiya, UA, May 1992.
T2.3-12 *Local Repair of Concrete Columns with Total Compressive Strength Loss. 2nd Part,*
J.L. Ramírez, J.M. Bárcena, J. Urreta, J.A. Sánchez & B. Hernández, LABEIN, July 1992.
T2.3-13 *Defect and Repair Technique Parameters and Repair Cost Estimation,*
G.M. Luikens, JAHN, June 1992.
T2.3-14 *Comparative Behaviour of Different Methods of Individual or Local Concrete Column Repair,*
J.L. Ramírez, LABEIN, November 1992.
T2.3-15 *Defect Describing Parameters (Type 2), Repair Parameters (Type 3), Repair Technique Choice Parameters (Type 4), a Repair Cost Estimation and Repair Method Description,*
G.M. Luikens, JAHN, December 1992.
T2.4-01 *Preliminary Modelling of Corrosion of Reinforcement,*
H.I. Hansen & P. Thoft-Christensen, CSR, April 1992.

T2.4-02 *The Reliability of Concrete Durability Measurements,*
P.S. Mangat & M.S. Elgarf, UA, April 1992.
T2.4-03 *Stochastic Models for Failure, Inspection and Repair,*
J.D. Sørensen & P. Thoft-Christensen, CSR, June 1992.
T2.4-04 *Information for further development of BRIDGE2,*
P.S. Mangat, M.K. Limbachiya & M.S. Elgarf, UA, November 1992.
T2.4-05 *Estimation of Chloride Content at the Level of the Reinforcement,*
H.I. Hansen, CSR, March 1993.
T2.5-01 *Updating Techniques,*
J.D. Sørensen & P. Thoft-Christensen, CSR, October 1991.
T2.5-02 *BETAEQ: Probability of Equality and Inequality Events,*
J.D. Sørensen, CSR, April 1993.
T2.5-03 *DISTUP: Updating of Distribution Parameters,*
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