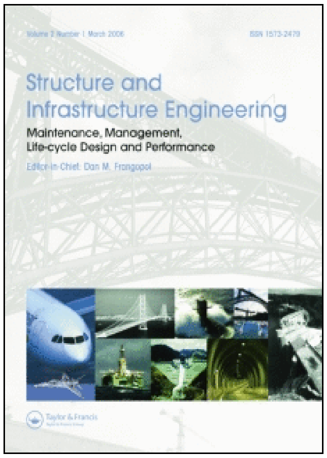

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Since 2004, the Autonomous Province of Trento, Italy, has adopted a Bridge Management System entirely based on reliability concepts. The system operates on the web, and includes sections for (1) condition state evaluation, (2) safety assessment, and (3) prioritization. Condition appraisal is based on visual inspections, and acknowledges the general rules of the AASHTO Commonly Recognized Standard Element system. Normally, the system conservatively estimates the prior reliability of each bridge, based on the sole inspection data. Where the condition of the bridge gives cause for concern, its reliability is evaluated in a more formal manner using multi-step procedures of increasing refinement. Decision-making is driven by a principle whereby priority is given to those actions that, within a certain budget, will minimize the risk of occurrence of an unacceptable event in the whole network. In this paper, the operation of the system is illustrated, with the support of a number of practical cases.

Keywords: Bridge management; Condition state; Reliability; Prioritization; Cost

1. Background

Administratively, Italy is divided into twenty Regions, each of which is in turn divided into provinces. Historically, the inter-regional network of roads was managed at a national level, while provinces were specifically charged with the management of local roadways. Since the seventies, Italy has undergone a political devolution process that is still ongoing and foresees the transfer of many of the administrative competences from national level to the local institutions.

The consequence of this process is that provinces have recently acquired the responsibility of a much broader stock of roadways both in number and type, and are now facing the problem of how to manage it in the most appropriate and effective way. At the same time, the need for new infrastructures has sharply decreased just as in other developed countries, while existing roads and bridges have been deteriorating. The combination of these factors explains the emerging interest in bridge management in Italy, and in the same way the particular nature of the specific demand.

This process has developed at a different pace nationwide, under the restraint offered by the different local political conditions, and has been streamlined in those regions and provinces already benefiting from special autonomy. In the case of the Autonomous Province of Trento (APT), the transfer of responsibility was completed in 1998. Since then, and without an adequate transition period, the Province has seen the number of bridges under its responsibility double from 412 to 936. The apparent need for a Bridge Management System (BMS), together with the associated belief that such a system should have been tailored to the specific stock and requirements, led to collaboration between the Department of Mechanical and Structural Engineering (DMSE) of the University of Trento (UniTN) and the Department of Transportation (DoT) of the Autonomous Province of Trento. The general objective for the BMS was to develop a management tool which could enable a systematic determination of the present and future need for maintenance, rehabilitation and replacement of bridges in the APT using various scenarios, along with a prioritization system which would provide guidance in the effective utilization of available funds. This effort

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resulted in the development of a management system which is reliability-based and fully operative on the web. This paper illustrates and discusses the main aspects of the system and is arranged as follows: in the next section an overview is given of the system and of its components; section 3 describes in detail the inspection system; in section 4 the reliability assessment methodology employed is discussed; section 5 introduces the prioritization principles and algorithms which control the decision-making process; in section 6 the operation of the system is illustrated with some practical examples; and a brief summary is provided at the end.

2. Outlines of the development of the BMS

2.1 *Features of the APT bridge stock*

The province of Trento is part of the mountainous region of the Alps. Currently, the APT has the ownership and the management of approximately 2340 km of roadways and 936 bridges, 461 of which were previously under State management. The APT stock, in terms of range of bridge types and ages, may be considered quite similar to most European stocks. Most APT bridges were constructed or reconstructed in the post World War 2 period, with the age distribution diagram in figure 1(a) showing a peak in the seventies. Reinforced concrete, regular and pre-stressed, is by far the most widely utilized construction material, covering more than 74% of the entire stock (see figure 1(b)). As for the typology, 65.1% of APT bridges are RC or PRC simply-supported or continuous beams, 10% are RC arches, 19.9% are unreinforced concrete or masonry arches, while only the remaining 6% include steel and steel-concrete composite bridges.

2.2 *Owner requirements*

The general approach to the project was based on the idea of the full involvement of the owner in each development phase. The initial outline of the system was hence made by a joint task group comprising representatives of the DoT (including the coordinator of maintenance works and the technical assistants who are the direct users of the system), the system planning unit of the University of Trento and a database and web-design specialist. The group defined and described the main features first and detailed them later. Based on this preliminary analysis, the BMS was planned with the objective of primarily fulfilling the following requirements:

- The system should give the owner a clear indication not only of the condition of each bridge, but also of its safety level.
- This information should be provided in real-time.
- The system procedures should be compatible with the most commonly recognized BMS standards, as well as with the developing Italian and EU bridge inventory standards.
- All of the subjects involved in the management operations should be allowed to directly interact with the system: DoT managers, DoT inspectors, professional engineers involved in the assessment procedure and external consultants.
- The system should incorporate the possibility to be connected to real-time permanent monitoring systems.
- Maintenance and upgrade of the system should be continuous and transparent to the users.

The step-by-step development of the system, which also involved a continuous *on site* interaction among the project

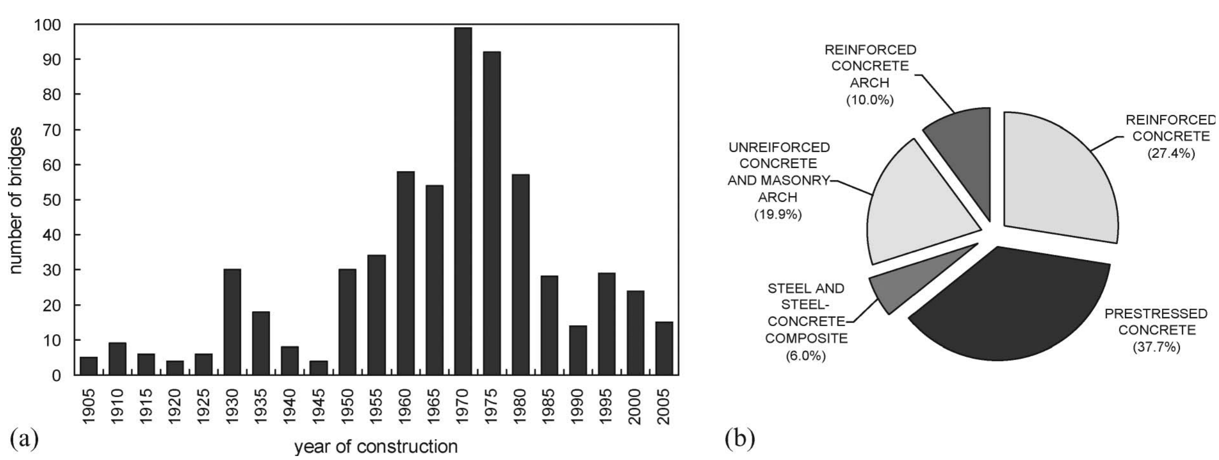


Figure 1. Bridge age distribution (a) and typological distribution (b) in APT.

partners, was used for a first check and calibration of the planning. This first two year phase of planning, implementation and trial was completed in early 2004. The subsequent inventory and calibration phase has started. This is when procedures and decision-making algorithms are reviewed and adjusted if necessary in order to acknowledge the observations of the users. While the first phase followed a top-down approach, the calibration is a bottom-up process, in which all of the actors are involved, including the low-level inspectors and the external consulting engineers. At this time, most of the bridge stock has been inventoried.

2.3 *State of the art of commercially available packages*

Specification of the components of the developing APT-BMS started with an analysis of the state-of-the-art in bridge management in Europe and North America. Also, the possibility of adopting commercially available packages was carefully considered.

In the USA, the Federal Highway Administration has developed the well-known software package named PONTIS (Thompson *et al.* 1998, Cambridge Systematics 2001), which has currently been adopted by about forty states in the USA and another two countries. This possibly represents one of the most advanced BMSs available on the market. At the same time, the BRIDGIT BMS (Hawk and Small 1998) has been developed mainly to address the needs of smaller DoTs.

An exhaustive review on bridge management in Europe is found in the documents resulting from the completion of the EU-funded BRIME research project (Astudillo 2002), which are fully available on-line (Woodward 2002). It appears that most of the European highway agencies have developed their management systems independently from the others (Binet 1996, Yanez and Alonso 1996, Das 1996, Hajdin 2002, Klatter *et al.* 2002) and no effort has been undertaken to divulge the developed products in the form of commercial packages. The main exception to this is DAMBRO (Lassen 2002, Lauridsen *et al.* 1998), originally developed for the Danish Road Directorate and specifically focusing on the organization of maintenance works.

2.4 *System concept*

Based on this preliminary analysis, the system developed towards a reliability-based and fully web-based concept. The basic parts of the BMS for the APT have been defined as shown in figure 2.

The system can be conveniently seen as a set of components, each specifically designed for a definite operative task, including inventory, condition inspection, safety assessment, cost appraisal, computation of priority indexes, decision-making. Each component consists of a procedure package and of operational tools that are computer- or paper-based. It is customary to distinguish project level modules and network-level modules, the former focusing on the single bridge, the latter concerning the bridge stock as a whole. The project-level modules aim at acquiring information which includes three different categories of data:

- Inventory data,
- Condition State (CS) data,
- Reliability data.

Inventory data includes all the information related to bridge identification, geographical location and features, administrative issues, construction and previous retrofits. This also includes a simplified model of the bridge, representing in a sufficiently detailed manner the logical distribution of its elementary units. This model serves as the basis for the definition of the Condition State appraised during routine inspections, as explained in section 3.

Most of the commercially available BMSs are based on the evaluation of the Condition State of bridge elements, implicitly assuming that the reliability of the bridge is somehow related to the CS. A weak point of this approach, highlighted by Das (1996), is that the influence of defects on the reliability of the bridge is ignored and that the assessment of the load-carrying capacity is not involved. Although bridges exhibiting poor CS can reasonably be expected to exhibit lower reliability indexes, it should be clear that there is no direct relation between safety level and CS. An old bridge can be unsafe, even when perfectly preserved, if it has been designed with a load-carrying capacity that is no longer adequate due to traffic changes. In the same way, a bridge exhibiting deterioration may still exhibit sufficient capacity. The APT's BMS departs from the usual approach, and foresees a specific section for formal reliability assessment of bridge structures, beside the CS assessment section. Reliability data directly concerns the capacity of the bridge, and consists of a set of reliability indexes β, each associated with an ultimate limit state and a specific Structural Unit or substructure. The method for appraising the reliability information of bridges will be described in detail in section 4.

Network-level data include all of the information that is not related to a specific bridge, but is relevant to the whole stock, or to a group of bridges having the same structural system and/or material. Network-level data are, for instance, the price list of intervention that defines the cost model and the deterioration rates of the Markovian matrices that define the deterioration model of each Standard Element. The network-level operation of the system will be discussed in more detail in section 5.

The database is fed by the information that comes from the documentation stored in the *archive* of the DoT and

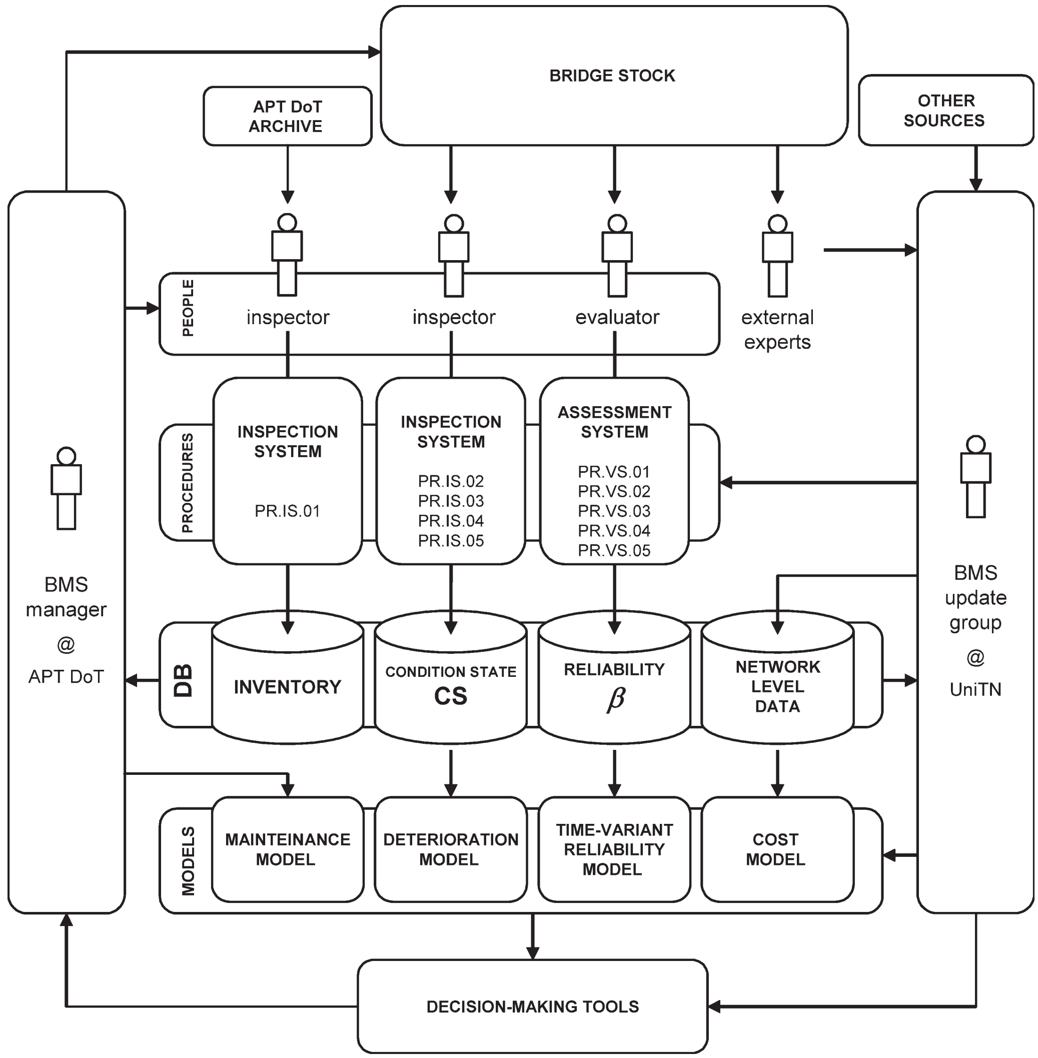


Figure 2. Layout of the APT-BMS showing its main components and information flows.

from the direct analysis of each bridge of the stock. The operation of feeding the database can be manual or automatic. Manned feeding includes *inspections* and *safety evaluations*, the first aimed at appraising the Condition State (CS) of the bridge, the latter at appraising its reliability index β. The subjects performing these operations are known as *inspectors* and *evaluators*. The system is also capable of automatically receiving data streaming from a permanent monitoring system, although this option has not been fully exploited by APT so far. While automatic feeding is by definition objective, the homogeneity of the results of an inspection or an evaluation are guaranteed by a set of procedures to which the inspector/evaluator must conform. The choice of substituting all or part of the manned work with automatic processes in the near future is not rigidly defined at the outset, but depends on issues that are technological (the actual availability of a technology, in the broad sense, capable of reproducing manual labour) and economical (the cost of instrumentation and its operation with respect to the cost of inspection).

2.5 *Software implementation*

One of the most appealing aspects of the APT's BMS is its full web-based operation. The system is based on a SQL database, which is accessible through the Internet. The actors involved in the operation of the system enquire and modify the database through a web-based user-friendly interface (an example is shown in figure 3). From the same web application, inspectors and evaluators download the proper procedures, and upload the data resulting from a condition assessment or from a safety evaluation, including possibly attachments, such as pictures, documents, FEM

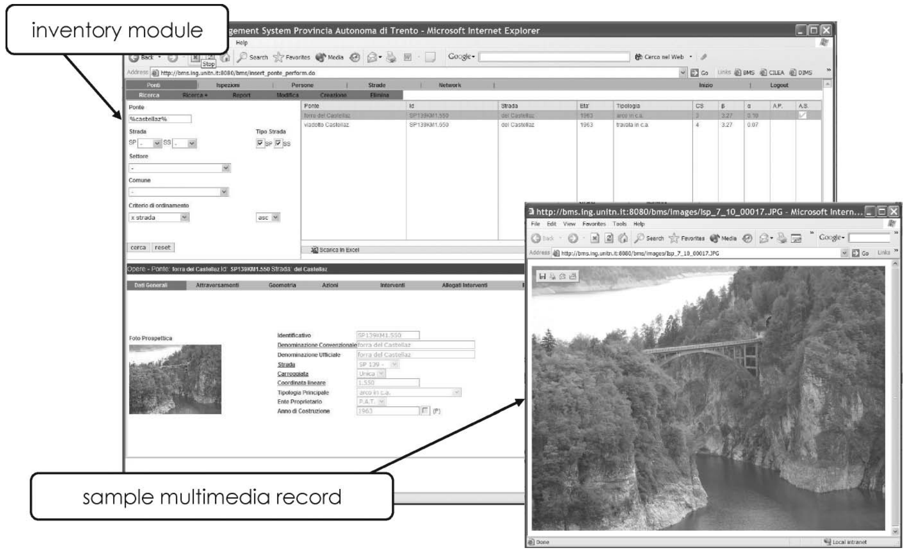


Figure 3. Display of the APT-BMS inspectors' and manager's web-interface.

models, AutoCad files. Appropriate restrictions to the system are given in such a way that each user can access only the information necessary for carrying out his task. Network-level analysis is performed real-time by a self-standing application, currently hosted and maintained at the University of Trento. As this tool is also Internet-based, the manager can access the result of the analysis on a real-time basis through the same web-interface used for browsing the database. The system is continuously updated by an operative group of the University of Trento. Updates include: detailing of the procedures, refinement of the network-level models, monitoring and evaluation of the work of inspectors and evaluators, optimization of the usability of the software and debugging. All these operations are remotely performed and are totally transparent to the users of the system.

3. Inventory and Condition State appraisal

3.1 *Aims of the evaluation*

The aim of the condition assessment of bridge structures is to detect whether a deterioration process is going on and, if so, to evaluate the degree of deterioration, with respect to the bridge in its original condition. This kind of information should be properly represented in a qualitative manner, for example through a plain description of type and extension of damage, possibly supported with pictures and tests results. However, the philosophy of the BMS is to condense this information in the form of one or a few quantitative parameters, suitable for being utilized in network-level algorithms, such as prioritization of interventions or calculation of deterioration speed. The issue of translating the CS information into numerical data in an *objective* way is basically an unsolved problem, mainly due to the fact that there is no commonly recognized approach that allows for 'measuring' damage in general terms. Therefore, at present the deterioration's assessment and the related quantification of a CS is purely conventional, and depends on the specific procedure adopted by the BMS.

3.2 *Bridge representation*

In order to combine simplicity and efficiency, the bridge is broken down into Structural Units (SU), such as deck, piles, abutments (as shown in figure 4), which are defined as conceptual entities characterized by common attributes (such as length, material, typology, spatial location, etc.). The spatial arrangement of SUs is defined through logical entities labelled connections (C). Each SU and C includes a set of Standard Elements (SE), which are specified in terms of quantity and Condition State (CS). CS is evaluated on the basis of a procedure that acknowledges, as long as possible, the general rules of the AASHTO (1997) Commonly Recognized (CoRe) Standard Element System. In detail, the APT system currently recognizes 22 types of Structural Units, 7 types of connections and 89 basic elements, 51 of which coincide with AASHTO CoRe elements. Elements are characterized by up to five discrete reference states, which describe the type and severity of deterioration mostly in visual terms. The choice of basing the CS evaluation on Standard Elements is an attempt to conserve the compatibility of the APT system with the PONTIS evaluation and deterioration models, as PONTIS is by far the most widely employed BMS worldwide.

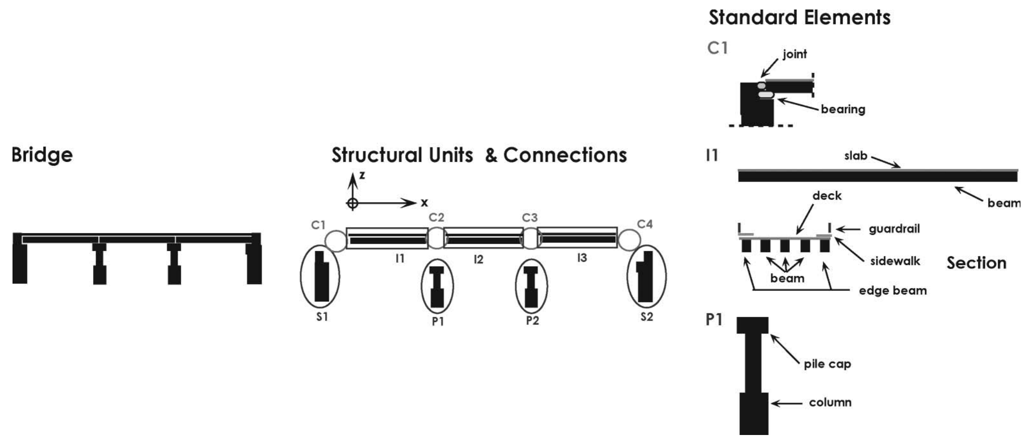


Figure 4. Example of bridge representation and break down into Structural Units and CoRe Standard Elements (AASHTO 1997), as implemented in the APT-BMS.

However, Structural Units in the APT system have a specific role in the estimation of the *a priori* reliability index of the bridge, while PONTIS Structure Units are conceived as mere logical groupings of Standard Elements.

3.3 *Inspection system*

The inspection system aims at collecting information concerning the inventory and the Condition State of each single bridge, and includes specific procedures for:

- Inventory inspections,
- Routine inspections,
- Special inspections.

The main objective of inventory inspections is to upload the aforementioned simplified bridge model into the system. The inspector is also required to carry out a formal verification of the consistency of the design documents with the as-built situation.

CS is normally appraised by annual and three-yearly routine inspections. Annual inspections are based on a simplified procedure, which requires the inspector to assess whether the bridge shows evidence of defects that are a source of concern either for its safety or for its future deterioration. Principal inspections are scheduled every three years, and consist of a detailed analysis of the condition of the bridge. Inspectors are required to appraise the condition, element-by-element, and to rank it based on a CS index, according to the APT evaluation manual, which in turn partially refers to AASHTO (1997).

Broadly speaking, all non-routine inspections aimed at assessing the condition of one or more elements of the bridge are referred to as special. Special inspection procedures are activated on the occurrence of particular events, such as:

- The inspector was unable to evaluate one or more elements during a routine inspection (for example because the element was not accessible using ordinary equipment), and the evaluation of these elements is estimated to be critical for the safety of the bridge.
- A routine inspection highlights the presence of structural anomalies, which are of concern for the safety of the bridge or the users, or the evaluated CS index can be associated to a condition of potential hazard requiring further investigation. In these cases the scope of the inspection is to verify and possibly better assess the extent of the damage.

Based on the outcomes of a special inspection, the System Manager can stop the evaluation procedure (*do nothing*), activate a safety assessment procedure or directly proceed with an intervention. This decision is supported by the principle of prioritization of actions, which will be more extensively discussed in section 5.

4. Assessing reliability

Quantitative bridge assessment has been identified as the essential part of the APT's BMS. Although the main purpose of assessment is to determine the safe load-carrying capacity, the accurate knowledge of the reliability index β of each bridge also plays a fundamental role in the decision-making process that controls retrofit, repair and reconstruction interventions. From a probabilistic standpoint, it is widely acknowledged that the problem of assessing an existing structure is conceptually different from design, because it makes reference to a different set of *a priori* information (e.g. ISO 2001, JCCS 2001, Ang and Tang 1984, Micic *et al.* 1995, Vrouwenvelder 1997, 2002, Das 1998, Nowak and Collins 2000). However, in most European countries, Italy included, the basis of assessment calculation is the same as for the design of new bridges, and the standards and other regulations used in the design apply equally well to the assessment, possibly by modifying the partial safety factor used (e.g. DIN 1985). An overall review on this topic can be found in Kaschner *et al.* (1999). On the other hand, countries like the UK, the USA and Canada have formulated an extensive system for assessment codes. More specifically, in the USA, a different target reliability β may be used in assessment (AASHTO 1994). Clause 12 of former Canadian CAN/CSA-S6-88, now replaced by CSA (2000), departs from usual practice by explicitly varying the reliability index β, and therefore the load factors, depending on the behaviour of the structure, the amount of warning of failure and the consequence of failure (Allen 1992, Buckland and Bartlett 1992, Kennedy *et al.* 1992). In the UK, the Design Manual for Road and Bridges (DMRB), published by the Highway Agency (1998) contains rules for assessment. Standard BD21 and Advice Note BA16 specifically deal with assessment loading, and the action to be taken when a sufficient load capacity of a bridge cannot be demonstrated. In addition, Advice Note BA79 aims at rationalizing the assessment procedure, by defining five levels of assessment of increasing complexity. The bridge is first evaluated with simple conservative methods and then with more refined methods, when a higher assessed load capacity is required, with the general expectation that higher levels will produce higher assessed capacities.

The general procedure of BA79 has been basically acknowledged in the recommendations resulting from the completion of the BRIME research project (Woodward 2002), and reported in detail in Cullington *et al.* (2001). In the knowledge of the authors, this report, along with DMRB-BA79, possibly represents the most advanced paradigm for including safety assessment in bridge management practice, and has been selected as the benchmark model in the implementation of APT-BMS. The developed assessment methodology is stated in detail in a set of procedures (APT 2004), and is briefly introduced further on. Consistently with BRIME recommendations, levels of assessment differ in respect of: (i) the information on material properties and loads, (ii) the calculation models, and (iii) the assessment methodology, as depicted in diagram form in figure 5.

Assessment Level (Procedure ID)	Strength and Load Models	Calculation Models	Assessment Methodology
1 (PR.VS.01)	Strength and load models as in design code. Material properties based on design documentation and standards.	Simple, linear-elastic calculation.	LRFD-based analysis. Load combinations and partial factors as in design code.
2 (PR.VS.02)		Refined, load redistribution is allowed, provided that the ductility requirements are fulfilled.	
3 (PR.VS.03)	Material properties can be updated on the basis of in-situ testing and observations using Bayesian approach.		
4 (PR.VS.04)			LRFD-based analysis. Modified partial factors are allowed.
5 (PR.VS.05)	Strength model including probability distribution for all variables.		Probabilistic analysis.

Figure 5. General scheme of the five stage procedure recommended by BRIME (Cullington *et al.* 2001), as implemented in the APT-BMS.

4.1 *Information on material properties*

Information on material properties and load models are provided by design specifications and standards at levels 1 and 2. At levels 3 and higher the evaluator is allowed to change the characteristic values of the variables used in the assessment on the basis of the outcomes of material sampling and observations. The main concern of the procedures is to provide unequivocal principles for assessment purposes, rather than practical application rules. The procedure leaves the evaluator free to carry out characterization tests on the materials, without specifying the minimum number of samples or the type of test. However, if the evaluator wants to utilize the outcomes of the tests in a quantitative manner, a probabilistic update technique should be employed. The value updating process is based on Bayes' theorem:

$$P(R_i \mid X) = \frac{P(R_i) \cdot P(X \mid R_i)}{\sum_i P(R_i) \cdot P(X \mid R_i)} \tag{1}$$

where R_i is a sequence of mutually independent events that completely cover the sample space and X is any event.

With Bayesian updating techniques, subjective judgments based on intuition, experience or indirect information are incorporated systematically with the observed data to obtain a balanced estimation. If the observed data agrees with the *a priori* assumptions, the updated information has a lesser degree of uncertainty than the initial information (Ang and Tang 1975, Miller and Freund 1985). If not, the updated information takes them both into account, giving greater weight to the observations as their number grows.

In practice, the APT-BMS procedures recommend three alternative methods that fulfill this general requirement. These procedures distinguish between direct and indirect tests. Direct tests are those that directly give the mechanical property values used in the structural evaluation; compression tests on cores or tensile tests on steel samples are, for instance, direct tests. When the number of samples x_i is sufficiently high for experimental definition of an average x_m and standard deviation σ_x, the following updated mean value R_m and standard deviation σ_R of a material property R can be recalculated (method A, Miller and Freund 1985):

$$R_m = \frac{(x_m \sigma_{R0}^2 + R_{mo} \sigma_x^2)}{(\sigma_{R0}^2 + \sigma_x^2)}, \tag{2a}$$

$$\sigma_R = \sqrt{\frac{\sigma_x^2 \sigma_{R0}^2}{\sigma_x^2 + \sigma_{R0}^2}}, \tag{2b}$$

where R_{mo}, and σ_{R0} represent the mean values and the standard deviations of the *a priori* distribution respectively. Normal distributions are assumed. This method does not take into account that the total uncertainness of resistance R depends on one side on the variability of R within the same lot or casting, and, on the other side, on the variability between different lots or castings. This information becomes crucial when only a small number of samples is available. To understand this concept better, let us suppose that we are interested in knowing the resistance of a material from a perfectly homogeneous lot. Given the perfect homogeneity of the lot, one single sample should theoretically be sufficient to assess the value of R in a deterministic manner. Conversely, equations (2) give no weight to the experimental outcome, as the standard deviation σ_x for a single sampling tends to infinity.

In order to overcome this contradiction, Ciampoli *et al.* (1990) proposed an alternate formulation of equations (2), acknowledged as method B in the APT-BMS procedures. According to this method, the non-homogeneity of the material within the same lot is described using a normal statistical distribution with standard deviation σ_1, which in fact is the prior standard deviation of R conditioned upon the knowledge of the actual structure strength mean value R_{m1}. In order to take into account the variability between different lots or casting, R_{m1} is in turn assumed normally distributed with standard deviation σ_2 and mean value equal to the prior mean value R_{mo}. Evidently, the two prior standard deviations σ_1 and σ_2 of method B are related to the total prior standard deviation σ_{R0} of method A through $\sigma_{R0}^2 = \sigma_1^2 + \sigma_2^2$. Using these assumptions, it is demonstrated that the posterior distribution of R is defined by the following parameters:

$$R_m = \frac{(n x_m \sigma_2^2 + R_{mo} \sigma_1^2)}{(n\sigma_2^2 + \sigma_1^2)}, \tag{3a}$$

$$\sigma_R = \sqrt{\sigma_1^2 + \sigma_4^2}\ , \tag{3b}$$

where n is the number of samples and σ_4 is provided by:

$$\sigma_4 = \sqrt{\frac{\sigma_1^2 \sigma_2^2}{\sigma_1^2 + n\sigma_2^2}}. \tag{4}$$

The reader can find additional information on the theory underlying equations (3) and (4) in many Bayesian statistics textbooks, such as Gelman *et al.* (2003).

An alternate implementation of Bayes' principle (method C), provided by the CSA (2000), is based on the calculation of the probability P_r that the investigated materials belong to a specific grade r:

$$P_r = \frac{P_{r0} L_r}{\sum_j P_{j0} L_j}, \tag{5a}$$

$$L_r = \prod_i f_r(x_i), \tag{5b}$$

where P_{r0} is the engineer's prior estimate of probability for grade r and L_r is the likelihood of test results x_i. f_r is the frequency of occurrence of grade r. With this in mind, the *a posteriori* characteristic value R_k is obtained by directly calculating the 5% fractile of the posterior distribution, solving the following implicit equation:

$$\sum_r P_r F_r(R_k) = 0.05, \tag{6}$$

where F_r is the cumulative function of distribution f_r. The mean value and standard deviation of the distribution are obtained in the same way. This technique may give unreliable results if different grades of material are sampled as if they were all one grade. If the updated probability does not represent a satisfactory confidence in the result, more tests may be conducted.

According to procedures, a diagnosis method is classified as indirect when its y_i outcomes are only indirectly correlated to the $x_{i,ind}$ value utilized in the calculation. Hardness and residual stress tests on steel, as well as pull-out tests on concrete, are typical examples of indirect methods. When using the results of an indirect test, the evaluator must account for the fact that the relation between a strength value $x_{i,ind}$, evaluated by means of an indirect test, and the corresponding value x_i, obtained through a direct test, features a certain degree of uncertainty, and must therefore be described in statistical terms. In most cases, the Probability Density Function (PDF) of $x_{i,ind}$ conditional to x_i is assumed to be normally distributed, and the standard deviation σ_5 of this relation can be obtained experimentally, evaluating the scatter between direct and indirect results for a number m of samples:

$$\sigma_5 = \sqrt{\frac{\sum (x_{i,ind} - x_i)^2}{m-1}}. \tag{7}$$

Eventually, the updated mean value and standard deviation of the material strength may be calculated as follows:

$$R_m = \frac{(nx_m\,\sigma_2^2 + R_{mo}\,\sigma_1^2)}{(n\sigma_2^2 + \sigma_1^2)}, \tag{8a}$$

$$\sigma_R = \sqrt{\sigma_1^2 + \sigma_4^2 + \sigma_5^2}. \tag{8b}$$

4.2 *Calculation models*

The difference between the first and the other four assessment levels is the method used to determine the effects of loads. Level 1 uses simple methods of structural analysis; simple load distribution and elastic analysis are allowed. More refined models should be adopted (starting at level 2) that can take into account the inelastic re-distribution of stresses and spatial effects in a more realistic manner.

4.3 *Assessment methodology*

Levels 1 to 4 are Load-Resistance-Factor-Design (LRFD) based, and the scope of the assessment is to calculate, for each relevant limit state, the live loads' amplification factor θ, which is defined as the coefficient that assesses a specific limit state equation, of the type:

$$R_d = \gamma_G\,G_k + \theta\gamma_{Q1}Q_{1k} + \sum_{i=2}^{n}\theta\gamma_{Qi}\,\psi_{0i}\,Q_{ik}, \tag{9}$$

where G_k and Q_k represent characteristic dead and live loads respectively, while γ and ψ are appropriate partial factors and combination coefficients. When equation (9) can be written as a linear combination of design loads and strengths, the limit amplifying factor θ can be calculated with the explicit expression:

$$\theta = \frac{R_d - \gamma_G\,G_k}{\gamma_{Q1}Q_{1k} + \sum_{i=2}^{n}\gamma_{Qi}\,\psi_{0i}\,Q_{ik}}. \tag{10}$$

At level 4 the evaluator is allowed to reduce the partial safety factors consistently with the actual knowledge of the information. In any case, the evaluator must formally demonstrate that the safety level of the structure obtained with the modified factors γ^* and the *a posteriori* information is not lower than the safety level obtained with the design code γ factors and the *a priori* information. In practice, a reliability analysis has to be carried out to validate any change in the partial factors.

Level 5 allows the evaluator to employ II level probabilistic methods, thus the safety assessment consists of calculating the reliability index β. Mean Value First Order Second Moment (MVFOSM) reliability methods are generally allowed, thus the reliability index is generally evaluated using the simple algebraic expression:

$$\beta = \frac{z(\mu_1, \mu_2, \ldots, \mu_n)}{\sqrt{\sum_i \left(\left.\frac{\partial z}{\partial x_i}\right|_{\mu_i}\sigma_i\right)^2}}, \tag{11}$$

where $z(x_1, \ldots, x_n)$ is the limit state function of n non-correlated random variable $x_1, \ldots, x_n$, μ_i and σ_i, are the mean value and the standard deviation of random variables x_i. It is worthwhile remembering that the basic concept of second moment methods is to set out all random quantities in terms of their first two moments, while no assumption is made as to the type of distribution of the variable. When this approximation is unacceptable, full probabilistic methods should be adopted. In this case, safety is directly evaluated in terms of the probability of failure P_F, using the Montecarlo simulation or direct integration of the safe domain. In any case, the evaluator is asked to report the result in terms of the reliability index, assuming $\beta = -\Phi^{-1}(P_F)$, where Φ is the cumulative normal distribution function. The reader, who is unfamiliar with the underlying theory of reliability, may refer to textbooks such as Benjamin and Cornell (1970) or Madsen *et al.* (1986).

4.4 *Correlation between θ and β*

Factors θ and reliability indices β obtained from the evaluation provide information for the BMS database, and offer the manager quantitative tools for decision-making. A statistical correlation between the factors θ assessed at each level and the corresponding index β is required, in order to use this information at the network-level. This is obtained through a calibration process which includes: (i) the definition of an *a priori* relation between θ and β; and (ii) the update of this relation, based on the outcomes of the full application of the five-step evaluation procedure to a number of specific case studies. The prior estimate of this relation is obtained by assuming that the limit state can be formulated as the difference between capacity (e.g. a resistance) R and demand (e.g. a load-induced solicitation)

S. The reliability index is related to the central safety factor $\gamma_0 = \mu_R/\mu_S$ through the expression:

$$\beta = \frac{\gamma_0 - 1}{\sqrt{V_R^2 \cdot \gamma_0^2 + V_S^2}}, \tag{12}$$

where V_R and V_S are appropriate estimates of the coefficients of variation of R and S, respectively. In turn, the central safety factor γ_0 is related to the characteristic safety factor γ_k through:

$$\gamma_0 = \frac{1 + kV_S}{1 - kV_R}\gamma_k, \tag{13}$$

where $k = 1.645$ for normal distributions. The characteristic safety factor γ_k depends on θ, as well as on the partial factors employed in the safety assessment. Unfortunately, the relation between these quantities is not straightforward and the formulation changes with the specific type of limit state. Assuming that the limit state can be formulated as $R_k/\gamma_R = \gamma_G G_k + \theta\gamma_Q Q_k$, a rough estimate of γ_k is obtained as follows:

$$\gamma_k = \frac{\gamma_R \gamma_Q \theta\left(\frac{\gamma_G G_k}{\theta\gamma_Q Q_k} + 1\right)}{\left(\frac{G_k}{Q_k} + 1\right)}. \tag{14}$$

As an example, the outcomes of the five-step evaluation procedure are reported in section 6 for a case study.

5. Decision-making and network-level tools

5.1 *Objectives and principles*

The primary objective of network-level bridge management is to provide transportation agencies with tools for the best allocation of economic resources, while maintaining an appropriate level of stock safety and serviceability. Theoretical research on network-level bridge management has developed strongly in the last decade, covering aspects such as condition rating prioritization (Stewart *et al.* 2001, Akgul and Frangopol 2003), optimum inspection strategy (Sommer *et al.* 1993, Onofriou and Frangopol 2002), and maintenance and repair optimization (Dekker 1996, Frangopol *et al.* 1997, Estes and Frangopol 1999, 2003, Frangopol and Estes 1999, Kong and Frangopol 2003). In most cases the optimum inspection and repair program is based on minimizing the expected life-cycle cost, while maintaining an acceptable level of reliability, which has to be defined somehow. Branco and Britto (1995, 1996) introduced a decision-making system based on the definition of a cost effectiveness index (CEI) for each option, underlying the possibility of quantifying the benefit of an intervention in terms of its economical value. More recently, Frangopol and Neves (2004) have proposed an approach incorporating condition, safety and cost. Prioritization is based on the minimization of a target function including three quantities, and provides decision-makers with a set of optimum solutions. It is worth mentioning that decision-making based on multi-objective optimization applies to a broad range of civil engineering problems, not always limited to bridge asset management (see for instance Chunlu and Hammad 1997, Lounis and Vanier 2000). However, multiple objective optimization often yields results which are sensitive to the weighting given to the parameters of the target function.

5.2 *Decision-making principle*

The prioritization approach adopted in the APT-BMS is based on the following principle: priority is given to those actions that, given a certain budget, will minimize the probability of occurrence of an *unacceptable event X* in the whole network, in the next $t_L = 50$ years. In practice, the decision-making process is based on the association of a priority index α to each of the potential actions (further assessment, repair, retrofit, replacement) as well as to any specific maintenance scenario. A priority index α is expressed by:

$$\alpha = \frac{\Delta P_X(t_L)}{\Delta C} = \frac{P_X(t_L) - P_{X|a}(t_L)}{\Delta C}, \tag{15}$$

where $P_X(t_L)$ is the cumulative-time probability of occurrence of an unacceptable event X in the stock over the duration $(0,t_L)$, a is the action, ΔC is the actualized life-long cost associated with implementation of the action.

To define an *unacceptable event* is an issue that concerns the owner, and is related to the management policy. From the structural engineer's point of view, a structural failure is typically seen as an unacceptable event, in which case $P_X(t_L)$ coincides with the cumulative-time probability of failure $P_F(t_L)$. However, the owner's attention is mainly focused on the consequences of a failure, should it occur. It should be noted that failure is not always associated with casualties, as this depends on the type of failure and the importance of the bridge. Failure of a pile or of a main member of the deck would probably determine the total collapse of the bridge, with the likelihood of a high number of casualties. Conversely, local failure of a slab would typically cause only the temporary closure of the bridge, resulting in an additional agency repair cost, as well as user costs associated with loss of use. Most of the reliability-based codes take account of this aspect by varying the target reliability with, (i) the expected consequences of a potential failure, and (ii) the importance of the structure. In LRFD-based codes roughly the same result is obtained by introducing appropriately modified partial factors or importance factors. A more rigorous approach would merit a formal definition of the statistical correlation between the occurrence of a failure and the occurrence of an unacceptable event, such as a casualty. Assuming that failure modes F_i are uncorrelated events, the cumulative-time probability of occurrence of an unacceptable event can be assessed through:

$$P_X(t_L) = \sum\nolimits_i P_{X|F_i} P_{F_i}(t_L), \tag{16}$$

where $P_{X|Fi}$ is the probability of an unacceptable event for the specific failure mode F_i. $P_{X|Fi}$ depends on the importance of the structure as well as the consequences of the specific failure mode, thus its meaning corresponds to that of the importance factor in LRFD. Currently the APT-BMS estimates the importance factor on the basis of failure mode, bridge dimensions and average daily traffic. The correlation of these factors is based on a simplified risk analysis, accounting for the consequence of a single failure mode.

The manager makes decisions by comparing the effectiveness of pairs of intervention scenarios, using the general formulation of equation (15). In addition, the system automatically assigns a ranking index to each bridge, calculated with reference to the following three predefined intervention scenarios:

- 0 (ZERO): do-nothing (maintenance only),
- A: repair intervention at time 0,
- B: reconstruction intervention at time 0.

For each scenario, the system describes the deterioration history of the bridge, and evaluates the corresponding time-cumulative probabilities P_{X0} P_{XA} P_{XB}, and life-cycle costs C_0 C_A C_B. Consistently with equation (15), the system calculates a repair priority index α_A and a reconstruction priority index α_B, according to:

$$\alpha_A = \frac{P_{X0} - P_{XA}}{C_A - C_0}; \qquad \alpha_B = \frac{P_{X0} - P_{XB}}{C_B - C_0}, \tag{17a,b}$$

and estimates the overall ranking index α as the maximum of the two values.

In summary, the estimate of the priority index α (according to the previously mentioned principle) requires the definition of models for calculating parameters $P_F(t_L)$, $P_{X|Fi}$ and ΔC for each decision option. This, in turn, requires the definition of models for deterioration, maintenance and costs. The link between these datasets is schematically represented in the diagram in figure 6. It is worthwhile noting that even if the condition does not explicitly enter the formulation of the priority index, it indirectly influences, on one side: life-cycle cost, and on the other side: time-cumulative reliability and associated risk. In turn, future Condition States are statistically estimated based on the actual condition assessed during past routine inspections, the deterioration model and the maintenance plan, as described later on.

5.3 *Deterioration, repair and maintenance models*

In the APT-BMS, the deterioration model of the bridge applies to the single Standard Element, considered as being independent of the bridge, just as in PONTIS and BRIDGIT. For each Standard Element a transition state matrix **D** is defined and the associated Markov process is calculated. A Markov chain is a discrete-time stochastic process for which the future condition only depends on the current condition. Matrix **D** collects the transition

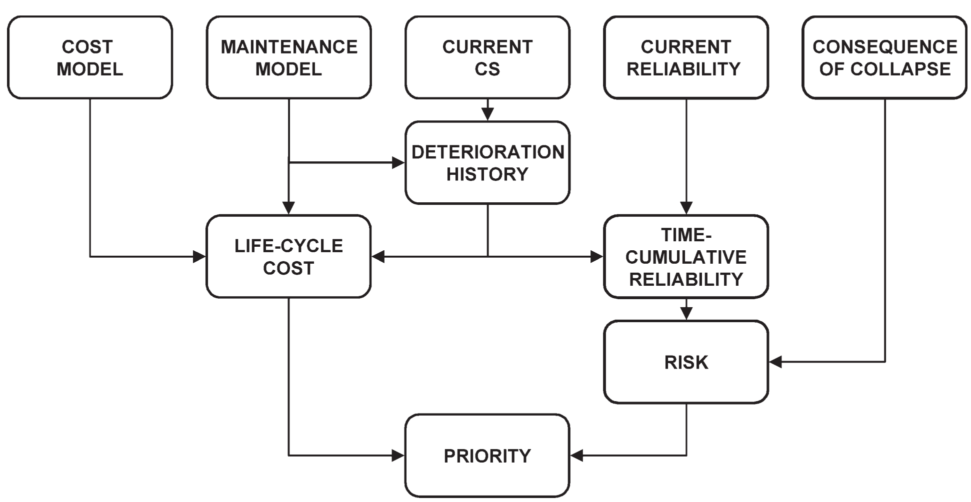


Figure 6. Linkage between datasets involved in the prioritization process.

probabilities between all possible CS pairs, thus its dimension is given by the maximum value of CS for the specific SE, varying from 3 to 5. In detail, the p_{ij} element of the transition matrix represents the conditional probability of moving into state i at year $n+1$ given that at the current year n the element is in state j:

$$p_{ij} = P(CS_{n+1} = i \,|\, CS_n = j). \tag{18}$$

Therefore, the time-variant **s**, the vector collecting the probabilities of the element being in one of the reference states, changes after m years according to:

$$\mathbf{s}_{n+m} = \mathrm{D}^m \mathbf{s}_n. \tag{19}$$

Similar to PONTIS and BRDGIT, the APT-BMS runs transition matrices that do not depend on t. This approach has been criticized, and may actually lead to inconsistencies, especially for those elements that have a small number of reference states. Das (1996) has also criticized the application of the deterioration model to the single elements, observing that the global data from a large number of bridges can lead to erroneous results in the determination of the deterioration rates of each component.

In the APT-BMS a *repair action* is defined as non routine intervention aimed at restoring the initial design capacity of the bridge. Conversely, *maintenance* includes all those scheduled activities that are part of the normal preservation program (i.e. the *maintenance plan*). Painting of steel elements or minor routine repairs of the deck overlay are examples of maintenance actions. The maintenance plan is stated by the BMS manager by defining types of preservation action and their time frequency. As with deterioration, the effect of an action, either repair intervention or maintenance, on the Condition State of the element is also statistically modelled using a Markovian transition matrix $\mathbf{T}(a)$. In this case the generic elements of the transition matrix p_{ij} represent the conditional probability of moving into state i given that the element is currently in state j, when an action a is taken.

5.4 *Cumulative-time failure probability*

Reliability-based analysis, as currently implemented in codes and recommendations, aims at evaluating the reliability index β of the structure, and therefore its probability of collapse P_F, assuming that the structure will maintain its mechanical characteristics over the years. The APT-BMS five-step assessment procedure is also based on this assumption. Nevertheless, the prioritization approach requires the calculation of a cumulative-time probability of collapse $P_F(t_L)$, which takes into account the deterioration of the construction materials. Mori and Ellingwood (1993) first proposed a time-variant method for directly evaluating the cumulative-time failure probability of the series system. Assuming that the system reliability is dominated by a single limit state Z, and that this limit state can be formulated as the difference between a capacity R and a demand S, the cumulative time probability of collapse can be formulated with:

$$P_F(t_L) = 1 - \int_0^{\infty} \exp\left(-\lambda\left[t_L - \int_0^{t_L} F_S\{rg(t)\}dt\right]\right) f_{R_0}(r)dr, \tag{20}$$

where λ and F_S are the mean occurrence rate and the cumulative distribution function of the demand S, $g(t)$ is the capacity degradation function and f_{R0} is the PDF of the baseline capacity R_0 assumed in design. Enright and Frangopol (1998, 1999a, 1999b, 2000) extended this formulation to the case of parallel systems and applied it to the assessment of deteriorating concrete bridges.

In Mori and Ellingwood's formulation, the degradation function $g(t)$ is not statistically defined. A consequence of this assumption is that the standard deviation $\sigma_R(t)$ of bridge capacity $R(t)$ decreases with time and degradation, and this is apparently against common experience. To overcome this limit, the authors propose to define a probabilistic degradation model. In detail, given a Structural Unit and a failure mode, the model indicates that the specific limit state is controlled by a time-variant capacity function of the type:

$$R(t) = R_0(1 - \delta(\mathbf{s})), \tag{21}$$

where δ is a probabilistic capacity degradation function, depending on time variant vector **s**, which collects the Condition States of the Standard Elements that control the capacity of the Structural Unit at the limit state. The degradation function δ is probabilistically defined, with a Probability Density Function:

$$f_\delta = \sum_{i=1}^{CS_{\max}} s_i \delta_i, \tag{22}$$

where s_i is the probability of being in the i-th CS, while δ_i is the PDF of the loss in capacity when the element is in the i-th CS. In order to better understand the practical meaning of the functions δ_i, it is worthwhile remembering that the elements are rated on the basis of visual inspections, therefore δ_i represents the likelihood of a certain loss in capacity when the element has been rated into the i-th reference state. Typically, low values of CS are not associated with any loss of capacity, in this case δ_i coincides with a Dirac delta function. Higher CSs are associated with distributions that reflect the uncertainty of the system in correlating the actual loss in capacity, with the verbal description of the reference state proposed by the inspection manual.

For example, Standard Element #012, *unprotected concrete deck*, is associated with five reference states. The Condition State 4 description reads: *patched area and/or spalling/delamination exists in the deck surface (etc.); reinforcement corrosion may be present but any section loss is incidental and does not affect the strength or serviceability of the element*. The system associates this description with a uniform distribution δ_4 of loss in capacity, for values of δ included in [0, 5%]. In the same way, the system associates the reference state 5 with a triangular distribution, for values of δ included in [5%, 70%]. Assuming that no maintenance action is performed over time, the PDF of the loss in capacity δ changes as shown in figure 7. Adopting the probabilistic degradation model of equation (21), it is shown that the cumulative-time failure probability of equation (20) becomes:

$$P_F(t_L) = 1 - \int_0^{+\infty} \exp\left(- \lambda \left[t_L - \int_0^{t_L} \left\{ \int_0^1 F_S(r(1-\delta)) \times f_\delta(\delta)\mathrm{d}\delta \right\} \mathrm{d}t \right] \right) f_{R_0}(r)dr. \tag{23}$$

5.5 *Normalized LS equation*

In order to calculate the priority ranking, the system must evaluate $P_F(t_L)$ according to equation (23), for each limit state, SU and bridge. Unfortunately most of the information required to define the distribution of capacity R and actions S are not explicitly contained in the system database, therefore a simplified approach must be adopted.

For this purpose, the first assumption is that S is a random stationary process, with mean value μ_S and standard deviation σ_S. Furthermore, it is convenient to define a normalized capacity $R^* = R/\mu_S$, a normalized demand $S^* = S/\mu_S$, and, therefore, a normalized limit state equation Z^* of the type $Z^* = R^* - S^*$.

It may be immediately observed that the normalized variables feature the following properties:

- R^* has mean value $\mu_{R^*} = \mu_R/\mu_S$ equal to the central safety factor γ_0 associated with limit state Z.
- S^* has mean value $\mu_{S^*} = 1$.
- The coefficients of variation of the normalized variables R^* and S^* are equal to those of R and S.
- The probability of failure P_F associated with the limit state Z coincides with that of Z^*, i.e.

$$P_F = P(Z < 0) = P(Z^* < 0). \tag{24}$$

Therefore, the system evaluates the cumulative-time failure probability of the limit state Z using the distribution associated to a normalized demand S^* and a normalized time variant capacity model:

$$R^*(t) = \gamma_0(1 - \delta(\mathbf{s})). \tag{25}$$

As an example, figure 8 shows how the PDF of R^* varies over the time for CoRe Standard Element #12.

5.6 *Estimation of the parameters*

Upper bound estimations of coefficients of variation V_R and V_S are easily defined on the basis of the failure mode. Indeed, the most delicate task is the evaluation of the central safety factor γ_0, that characterizes the design limit state. As previously mentioned, when the bridge is formally evaluated, the result of the evaluation can be expressed in terms of live load rating factor θ or in terms of reliability index β, depending on the level of refinement of the evaluation. In the first case, the characteristic safety factor

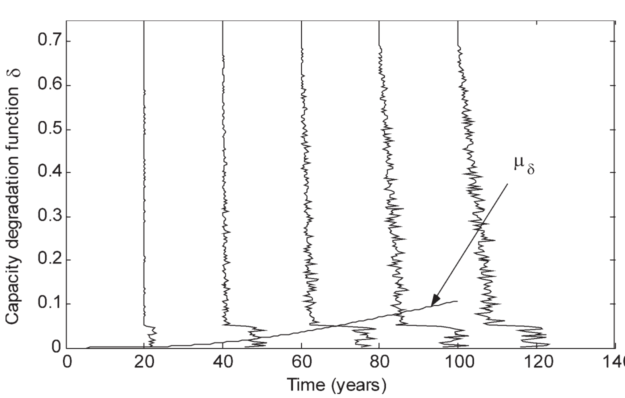


Figure 7. Capacity degradation function for a limit state associated with Element #12 (concrete deck).

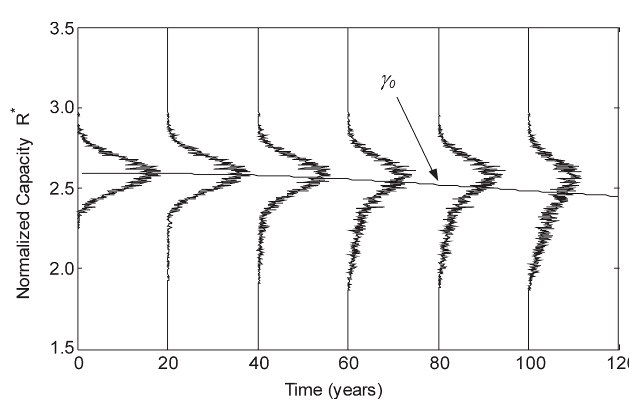


Figure 8. Time variant normalized capacity PDF for a limit state associated with Element #12 (concrete deck).

γ_k is evaluated using equation (14), while γ_0 is related to γ_k through:

$$\gamma_0 = \frac{1 + k\bar{V}_S}{1 - k\bar{V}_R}\gamma_k. \tag{26}$$

This expression is basically the same as equation (13), except that $\bar{V}_R$ and $\bar{V}_S$ are now lower bound estimations of V_R and V_S, respectively. If the bridge has been evaluated in terms of β, the value of γ_0 is calculated by numerically solving the following equation:

$$\Phi(-\beta) = P_F(\gamma_0, \mathbf{s}), \tag{27}$$

where:

$$P_F(\gamma_0, \mathbf{s}) = \int_0^{+\infty} f_{R_0}(r)\left[1 - \int_0^1 F_S(r) f_\delta(\delta)\mathrm{d}\delta\right]\mathrm{d}r \tag{28}$$

with f_{R0} being normally distributed, with mean value γ_0 and standard deviation V_R. Where the bridge has never been formally evaluated, the BMS estimates a conservative rating factor θ on the basis of the design code in force at the time of bridge construction and of the potential failure modes associated with the SU of which the bridge is composed. The value of γ_0 is then calculated using equations (14) and (26).

5.7 *Cost model*

The life-cycle cost C_{TOT} associated with a MR&R scenario is estimated as:

$$C_{TOT} = C_0 + C_I + C_M + C_R + C_F, \tag{29}$$

where C_0 is the reconstruction cost, C_I the inspection cost, C_M is the maintenance cost, C_R is the repair cost and C_F is the failure cost. This is a quite classical formulation also found, with minor differences, in Branco and Britto (1996) or in Frangopol *et al.* (1997). New construction or replacement costs, as well as rehabilitation costs, are estimated on the basis of euros per square metre of deck area, and depend on the typology stated by the SUs. Unit costs per square metre *uc* are linearly related to the span length L according to:

$$uc = uc_0(1 + mL). \tag{30}$$

As with the PONTIS procedure, bridge maintenance and repair costs are estimated on the basis of maintenance/repair unit costs of Standard Elements. Unit costs are calculated for each SE and each allowable action on the basis of official bulletins (tenders) and are calibrated on the basis of agency records. Typically SE unit costs are assumed to be independent of the SE quantity, except in a few special cases. Inspection cost C_I is estimated on the basis of bridge geometry and accessibility. The failure cost C_F takes account of all of the structural and functional costs associated with a potential failure. An estimate of C_F is obtained by integrating year by year the product of the annual probability of failure by the total cost C_{F0} associated with the bridge deficiency caused by the failure. In the current version of the BMS, C_{F0} is simply estimated as a percent of the reconstruction cost. All future costs are calculated using the official financial discount rate published yearly in the APT Construction Pricelist Bulletin.

6. Examples

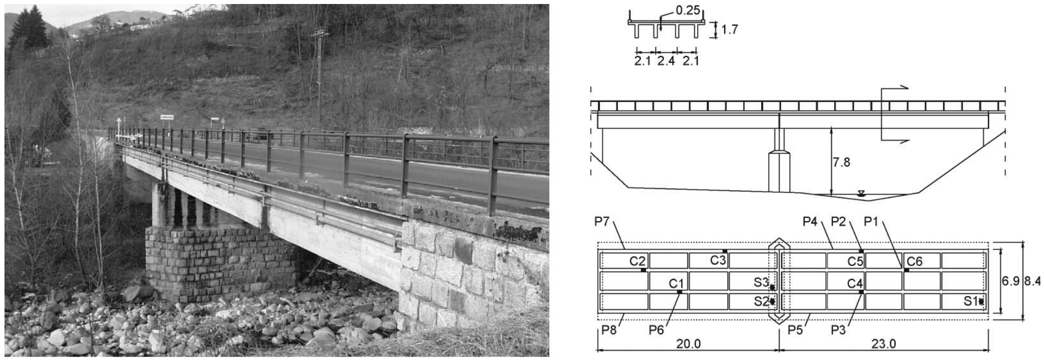
In order to improve understanding of the way in which the BMS deals with the reliability information in assessment and prioritization, the operation of the system is illustrated in the following sections with reference to three case studies.

6.1 *The SP65 bridge on the Maso river*

The SP65 bridge on the Maso river (figure 9(a)) is a common type of bridge in the APT stock, and for this reason was included among the principal case studies during the start-up phase. The structure has two simple spans of 19.0 m and 22.0 m, and a total length of 43.0 m. Each span has four girders spaced at 2.1 m, 2.4 m and 2.1 m respectively. The cross-section of the girders is shown in figure 9(b). The deck consists of 22 – 27 cm of reinforced concrete and a 15 cm surface layer of asphalt. The roadway width is 7 m with 0.70 m pedestrian pavements and hand railing on each side. The deck dates from 1962, and replaces an older timber superstructure.

The structure features minor deterioration of the beams, including localized concrete cover spalls, mostly due to an inefficient drainage system. The bridge was formally evaluated during the start-up phase, through the full application of the five-step assessment procedure.

At the lower assessment levels, an *a priori* characteristic strength of concrete $f_{k0} = 25$ MPa and yield value of steel $f_{yk} = 360$ MPa, have been assumed based on design documents, which prescribe a C20/25 grade for concrete and an AQ60 hard steel for reinforcement. The result of the evaluation in terms of live load rating factor θ and equivalent reliability index β are reported in the corresponding column of table 5. It may be observed that, in general, the bridge appears adequately dimensioned for carrying the loads envisaged by the design code currently in force in Italy (MLLPP 1990). The only exception is the slab which resulted as unverified ($\theta_i = 0.44 < 1$), with respect to a moment limit state, although this structure does not exhibit any sign of degradation, and has been evaluated in



(a) (b)

Figure 9. SP65 bridge on Maso river, near Carzano: (a) overview; (b) plan view, elevation and cross section of the deck.

CS1 (*no damage*) during the principal inspection. This is due to the fact that the nominal concentrated print load $Q_k = 100$ kN required by the current design code, issued in 1990, is double the corresponding load $Q_k = 50$ kN required at the time of the bridge's construction. Using a finite element model of the slab, accounting for the spatial redistribution of stresses, the live load factor increases to $\theta_2 = 0.53$, which is still insufficient for considering the bridge formally assessed.

Starting at level 3, the mechanical properties of materials have been directly investigated through tests on 6 concrete cores (C) and 6 reinforcement coupons (S), sampled from the girders and the slab, at the positions shown in figure 9(b). In order to clarify the updating procedures, the outcomes of the direct tests are reported in table 1. Normal prior distributions have been assumed, both for concrete and steel. The corresponding mean values and standard deviations have been assigned, consistently with the provisions found in the design documents. The application of the three different update methods, A B and C, provides posterior characteristic values of $f_{ckA} = 38.5$ MPa, $f_{ckB} = 35.8$ MPa and $f_{ckC} = 37.7$ MPa for the concrete, and values of $f_{yA} = 358$ MPa, $f_{yB} = 380$ MPa and $f_{yC} = 350$ MPa for the steel, with distribution values that are detailed in table 2. The assessment performed with the updated value yields a live load rating factor $\theta_3 = 0.56$.

Concrete has also been investigated through pull-out tests. A pull-out test consists of measuring the extraction force F of a standard expansion bolt from concrete. As the pull-out force depends on the concrete grade f_c, the test can be used for indirectly investigating the properties of the concrete. Pull-out tests have been carried out at eight positions (P) on the slab and the girders, four of those coinciding with core-drilling points. For each measurement position, three or four extraction tests have been repeated, obtaining the results shown in table 3. A parametric linear model, correlating pull-out forces F with corresponding concrete strengths f_c, has been identified utilizing the values measured at positions P1, P2, P3 and P6. The scatter of these values with respect to the correlation trend line has been calculated as $\theta_5 = 4.5$ MPa using equation (7). Assuming $f_{cmB} = 41.7$ MPa and $\theta_{fcB} = 3.6$ MPa as a prior mean value and standard deviation, equation (8) yields a posterior characteristic value of $f_{ck} = 34.5$ MPa. It should be observed that in this case the updated characteristic value is lower than that assumed *a priori*, and is equal to $f_{ckB} = 35.8$ MPa. The justification of this disappointing result is found in the high standard deviation θ_5, which underlies, in this specific case, a definitively poor correlation between the compressive strength of the concrete and the pull-out force. This experiment clearly highlights how, as a rule, the use of indirect-test results may be misleading when the statistical treatment of the samples is not appropriately performed.

Table 1. SP65 bridge on Maso river: outcomes of compression tests on concrete (C) and tensile tests on steel (S).

Test ID	f_c (MPa)	R_c^* (MPa)	Test ID	ε_u^{**} (%)	f_y (MPa)	f_t^{***} (MPa)
C1	18.2	21.9	S1	23	408.0	638.2
C2	43.0	51.8	S2	24	452.3	709.8
C3	40.1	48.3	S3	22	440.8	759.8
C4	33.3	40.2	S4	18	422.0	720.9
C5	42.2	50.8	S5	29	401.7	616.8
C6	37.5	45.2	S6	30	388.6	626.9

*Cubic compressive strength. **Ultimate elongation. ***Tensile strength.

At level 4 the partial factor for steel $\gamma_m = 1.15$, prescribed in the design code, have been reduced, taking into account the *a posteriori* distribution of f_y resulting from the updating procedure. Consistently with the assessment procedure, the modified partial factor γ_m^*, must satisfy the previously mentioned *equal-safety-level principle*, i.e. the safety level obtained with γ_m^* and the *a posteriori* information should

Table 2. SP65 Bridge on Maso river: prior and posterior resistance distributions obtained by different update methods.

Compressive strength of concrete f_c				Yield strength of steel fy			
Update Method	Characteristic Value (MPa)	Mean Value (MPa)	Standard Deviation (MPa)	Update Method	Characteristic Value (MPa)	Mean Value (MPa)	Standard Deviation (MPa)
Prior	25.0	34.0	5.5	Prior	360.0	442.0	50.0
A	38.2	45.1	4.2	A	358.0	420.0	37.8
B	35.8	41.7	3.6	B	350.0	415.5	39.8
C	37.7	44.0	3.8	C	380.0	420.2	24.5

Table 3. SP65 Bridge on Maso river: outcome of pull-out tests.

Pull-out Test ID	Corresponding Core ID*	*F1* (kN)	*F2* (kN)	*F3* (kN)	*F4* (kN)
P1	C6	52.7	67.5	67.5	64.8
P2	C5	48.6	51.3	45.9	–
P3	C4	56.7	41.9	59.4	–
P4	–	48.6	47.3	52.7	–
P5	–	56.7	40.5	52.7	–
P6	C1	50.0	44.0	63.5	–
P7	–	52.7	59.4	62.1	–
P8	–	59.4	63.5	50.0	48.6

*ID of the concrete core sampled at the same position as pull-out test.

not be lower than the safety level obtained with γ_m and the *a priori* information. This principle is fulfilled when the following procedure is adopted:

- A reliability index β_0 associated to moment failure of the slab is estimated, using the posterior mean value $f_{ym} = 420.2\,\text{MPa}$, and the prior standard deviation $\sigma_{fy0} = 50.0\,\text{MPa}$.
- The mean yield value $\tilde{f}_y = 392.5\,\text{MPa}$ is calculated, defined as the mean value of yield which, along with the posterior standard deviation $\sigma_{fy} = 24.2\,\text{MPa}$, gives a reliability index at least equal to β_0 for the specific limit state.
- The reduction factor α_{fy} is calculated, according to:

$$\alpha_{fy} = \frac{\tilde{f}_y}{f_{y0}} = 0.934. \tag{31}$$

- The new partial factor is calculated with $\gamma_m^* = \alpha_{fy}\,\gamma_m = 1.071$.

Using the reduced partial factor, the assessed rating factor changes to $\theta_4 = 0.68$, corresponding to an equivalent reliability index $\beta = 2.71$. At level 5 the safety indices of the bridge, with respect to the occurrence of eight different failure modes, have been directly evaluated using a probabilistic approach. For instance, the limit-state equation for the moment failure of the slab is:

$$Z = M_R(f_y) - M_S(G_s, G_p, Q), \tag{32}$$

where M_R and M_S indicate the bending capacity and the bending demand of the slab, respectively. The limit state function depends on the random variables reported in table 4. Using a MVFOSM method, a value of $\beta = 2.87$ has been calculated for the specific limit state, corresponding to a failure probability $P_F = 2.05 \times 10^{-3}$. It is worth noting that this value almost coincides with the β-equivalent, assessed at level 4.

Table 5 summarizes the results of the five-step assessment procedure carried out on the SP65 bridge, in terms of β or β-equivalent, as well as of θ for the first four levels of assessment. It may be observed how it is generally verified that higher levels of assessment provide higher safety indices. In any case, it is clear that the reliability associated with the moment limit state of the slab is always formally unacceptable, regardless of the level of refinement of the assessment procedure.

This fact, however, does not excessively penalize the ranking of the bridge. In fact, print load affects limit states associated to a secondary structure, and these are associated with a relatively low importance index ($P_{X|F} = 2.06 \times 10^{-4}$). Table 6 reports a summary of the outcome of the prioritization analysis, automatically carried out by the BMS for the three reference scenarios mentioned in section 5.1. The system recognizes that reconstruction (scenario B) would significantly improve the cumulative-time bridge reliability. However the high associated cost $C_0 = 365.28$ k€, is out of proportion to the expected benefit, and this is quantitatively highlighted by the definitively low priority index $\alpha_B = 1.14 \times 10^{-10}$ €$^{-1}$. Repair is not as effective as reconstruction from the reliability point of view. However the cost of repair is much lower ($C_R = 75.71$ k€), and the associated priority index $\alpha_A = 5.37 \times 10^{-10}$ €$^{-1}$ shows that repair is more appropriate in this case.

6.2 *Canova viaduct*

The Canova Viaduct (figure 10), located in Northern Trento, carries a four-lane carriageway which represents one of the most critical road connections in the region, with average daily traffic of over 15000 vehicles. The main structure is 686 m long and 17.70 m wide, and has 34 simply

Table 4. Variables used in reliability analysis of the slab of the SP65 bridge on Maso river.

	Variable definition	Unit	Distribution	Mean Value	Standard Deviation	CoV*
G_s	Structural dead load	kN m^{-1}	Normal	6.0	0.3	0.05
G_p	Non-structural dead load	kN m^{-1}	Normal	3.3	0.4	0.15
Q	Live load	kN m^{-1}	Normal	162.8	12.8	0.09
f_y	Yield stress of steel reinforcing	MPa	Normal	360.0	91.9	0.18
L	Slab span	M	Deterministic	2.38	–	–
l	Load print dimension	M	Deterministic	0.86	–	–
A_{s1}	Reinforcement area at middle-span	mm^2	Deterministic	628	–	–
A_{s2}	Reinforcement area at girder	mm^2	Deterministic	628	–	–
d	Effective depth of slab	m	Deterministic	0.24	–	–

*Coefficient of variation.

Table 5. Outcomes of the assessment of the SP65 bridge on Maso river.

Limit State	Substructure	Level 1 $\beta(\theta)$	Level 2 $\beta(\theta)$	Level 3 $\beta(\theta)$	Level 4 $\beta(\theta)$	Level 5 β
Moment	Slab	2.00 (0.44)	2.30 (0.53)	2.36 (0.56)	2.71 (0.68)	2.69
Shear/Punching	Slab	3.43 (1.12)	3.43 (1.12)	3.71 (1.43)	–	3.68
Moment	Deck beam	3.58 (1.27)	3.59 (1.28)	3.62 (1.32)	–	3.80
Shear/Punching	Deck beam	3.76 (1.49)	3.80 (1.56)	3.81 (1.57)	–	4.69
Moment	Deck beam	3.29 (1.01)	–	3.32 (1.03)	–	3.52
Shear/Punching	Deck beam	3.72 (1.44)	–	3.77 (1.51)	–	4.86
Axial Load	Pile	4.73 (7.90)	–	4.74 (8.20)	–	–
Axial Load	Pile/Foundation	3.44 (1.13)	–	3.44 (1.13)	–	–
Overturning	Abutment	4.12 (2.21)	–	4.12 (2.21)	–	10.72
Sliding	Abutment	3.85 (1.63)	–	3.85 (1.63)	–	6.18
Moment	Abutment	3.50 (1.19)	–	3.56 (1.25)	–	–
Shear/Punching	Abutment	4.85 (13.9)	–	4.85 (14.4)	–	–

Table 6. Summary of the prioritization parameters of the SP65 bridge on Maso river.

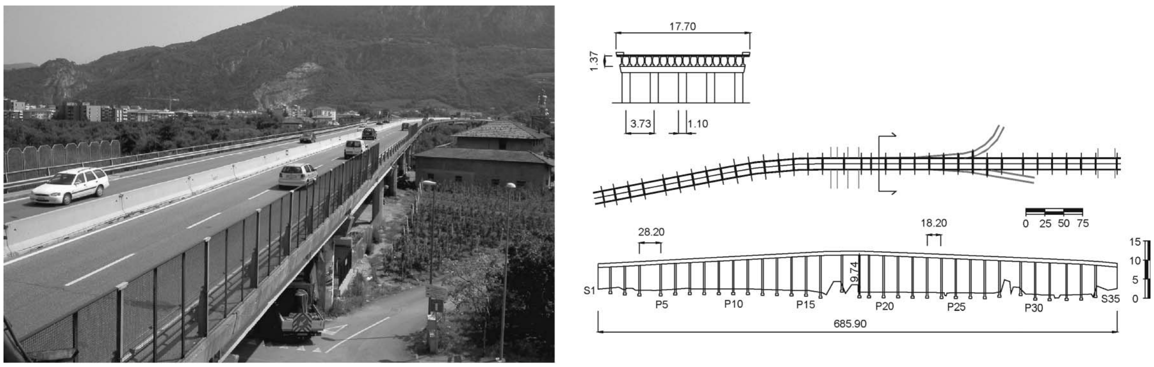
	Variable definition	Unit	ZERO do-nothing	A repair	B reconstruction
CS	Condition State	–	2.90	1.18	1.00
β_{min}	Minimum assessed value of β	–	2.550	2.611	3.304
β_{main}	Minimum β associated to the main structure	–	3.433	3.477	3.304
C_M	Life-cycle maintenance cost	k€	26.30	8.05	4.04
C_R	Repair cost	k€	–	75.71	–
C_0	Reconstruction cost	k€	–	–	365.28
P_X	Cumulative-time risk	–	4.17×10^{-5}	1.02×10^{-5}	5.96×10^{-7}
α	Priority index	€$^{-1}$	–	$\mathbf{5.37 \times 10^{-10}}$	1.14×10^{-10}

supported spans of variable length. Each span has 17 equally spaced (0.90 m) double-T pre-stressed concrete beams, connected by post-tensioned cross-beams. The deck consists of 20 cm of reinforced concrete plus a 10 cm surface layer of asphalt. The carriageway width is 15.70 m with 1.00 m pedestrian pavements, and the multi-column piers have a maximum height of 13.6 m. The bridge dates from 1978 and shows signs of advanced deterioration at the cross-beams, resulting in some cases in the failure of the post-tensioning system. These faults are due to poor design in detail and execution. The bridge is currently undergoing a formal assessment of its safety condition.

Based on information provided by visual inspection only, the system currently yields a conservative value of the reliability index $\beta_{main} = 1.632$ associated with the moment limit state of the deck beams. As shown in table 7, the risk associated with the *do-nothing* scenario is relatively high, but can be significantly reduced by repair. Repair is ranked as the most cost-effective action ($\alpha_A = 1.05 \times 10^{-9}$ €$^{-1}$). However, reconstruction is also associated with a high priority ($\alpha_B = 8.11 \times 10^{-10}$ €$^{-1}$), higher than that calculated for a possible repair of the SP65 bridge.

6.3 *SP12 bridge on Vignola river*

This is a minor bridge serving a local road with average traffic of a few vehicles per day and consisting of a single simply supported 5 m wide and 10 m long span (see figure 11). The superstructure consists of a 20 cm concrete

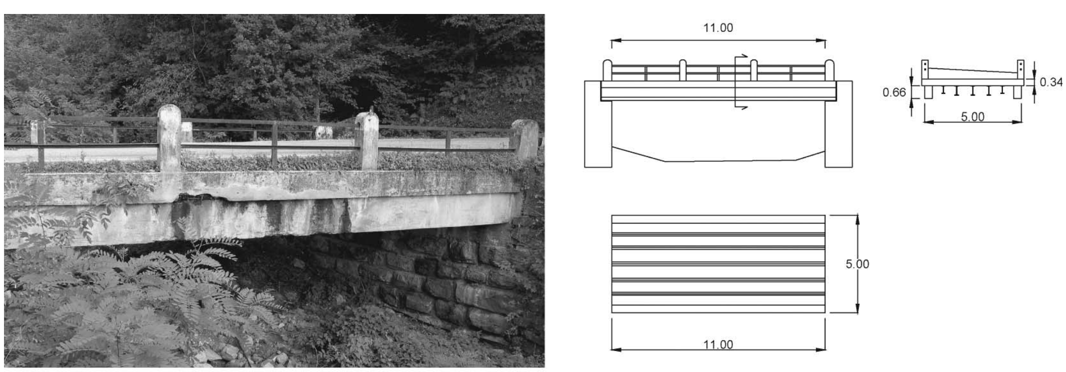


(a) (b)

Figure 10. Canova Viaduct: (a) overview; (b) plan view, elevation and cross-section of the deck.

Table 7. Summary of the prioritization parameters of the Canova Viaduct.

	Variable definition	Unit	ZERO do-nothing	A repair	B reconstruction
CS	Condition State	–	4.43	1.72	1.00
β_{min}	Minimum assessed value of β	–	0.432	1.175	3.304
β_{main}	Minimum β associated to the main structure	–	1.632	2.503	3.304
C_M	Life-cycle maintenance cost	k€	2068.80	648.49	333.59
C_R	Repair cost	k€	–	6084.50	
C_0	Reconstruction cost	k€	–	–	11462.00
P_X	Cumulative-time risk	–	8.00×10^{-3}	3.08×10^{-3}	2.50×10^{-5}
α	Priority index	€$^{-1}$	–	$\mathbf{1.05 \times 10^{-9}}$	8.11×10^{-10}



(a) (b)

Figure 11. SP12 bridge on Vignola river: (a) overview; (b) plan view, elevation and cross-section of the deck.

slab supported by the two edge beams along with five additional steel I-beams. Visual inspection assessed high CS for most of the elements forming the superstructure. Specifically, the steel beams show sign of advanced corrosion including flaking and swelling, causing considerable loss of section in some points, estimated at 20%.

Given the minor importance of the bridge, the cumulative-time risk evaluated by the system is relatively low ($P_X = 7.80 \times 10^{-5}$), as reported in table 8. However, the cost of potential action is also very low, and for this reason the resulting priority indices are comparable with those of the Canova Viaduct. In this case, replacement is ranked as the most cost-effective action, with a priority index close to 10^{-9} €$^{-1}$.

7. Final remarks

Today, bridge management is typically based on the evaluation of the Condition State, while bridge safety is often only indirectly involved, implicitly assuming that the reliability index is somehow related to the CS. However, some bridges require a more formal approach to their safety, while a quantitative evaluation of β is also needed for implementing prioritization techniques that explicitly take into account the probability of occurrence of a failure.

Assessing the bridge stock with a reliability-based method is expensive. With this in mind, the APT's BMS has evolved towards a multi-stage assessment scheme, which guarantees an adequate degree of confidence of the safety information, at reasonable operational costs. However, assessment procedures must envisage very strict provisions in order to ensure homogeneity in the evaluators' judgment. This issue is more critical at higher assessment levels, when the evaluation methodology departs from the usual practice stated by design codes.

Condition State is still quite a nebulous concept. Its scope is to represent (in a quantitative form) the visual extent of damage, or the deterioration of a bridge, which is actually a qualitative concept. Simple systems for assessing the CS, like those adopted by most commercial BMSs, are based on a few discrete ranking values that make the inspection easy to perform and ensure unequivocal outcomes. On the other hand, a continuous CS is preferred in order to operate with network-level algorithms. So far, it appears that no truly convincing principle for assessing condition state has been proposed.

The APT's prioritization system is also reliability-based, where the effectiveness of each allowable action is ranked on the basis of the rate of risk reduction to cost. In turn, risk is related to the cumulative-time probability of occurrence of a structural failure. The implementation of this framework has required the formulation of models for estimating deterioration, life-cycle cost and cumulative-time reliability. The effectiveness of the prioritization algorithm has been validated in a number of case studies, which are representative of the APT stock for bridge type and construction material. In this paper, the details of the models developed have been presented. The cases reported here highlight how the prioritization procedure operates correctly and yields a realistic measure of the cost effectiveness of a decision option, even when the level of refinement of the models is relatively low.

As optimization models are becoming more and more refined, bridge managers are provided with more powerful tools for project planning and budget allocation. However the great effort spent in theoretical research is not always accompanied by appropriate validation work, based on experimental observations and data-mining of existing records. Without adequate calibration, higher complexity in the management tools merely translates into a higher degree of freedom in administration. The risk is that the owner might conceive the BMS as a highly refined instrument for justifying what are, in fact, political decisions with technical arguments.

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Table 8. Summary of the prioritization parameters of the SP12 bridge on Vignola river.

	Variable definition	Unit	ZERO do-nothing	A repair	B reconstruction
CS	Condition State	–	4.30	1.61	1.00
β_{min}	Minimum assessed value of β	–	0.709	1.204	3.304
β_{main}	Minimum β associated to the main structure	–	1.865	2.584	3.304
C_M	Life-cycle maintenance cost	k€	20.41	7.36	2.35
C_R	Repair cost	k€	–	65.36	–
C_0	Reconstruction cost	k€	–	–	94.10
P_X	Cumulative-time risk	–	7.80×10^{-5}	3.31×10^{-5}	2.79×10^{-7}
α	Priority index	€$^{-1}$	–	8.41×10^{-10}	$\mathbf{9.98 \times 10^{-10}}$

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