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DECISION MAKING FOR BRIDGE STOCK MANAGEMENT

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SUMMARY-SOMMARIO

Bridges in service in most Western Countries were built according to codes with design loads that are now inconsistent with today's traffic demands. Currently, transportation agencies do not know how to respond to transit applications on their bridges. This thesis focuses on the legal issues entailed by overweight/oversize load permits issued by transportation agencies. Indeed, correct decision-making should consider the legal liabilities involved in possible catastrophic events. In this thesis I illustrate how this problem is guided by the Department of Transportation of the Italian Autonomous Province of Trento (APT's DoT), a medium-sized agency managing approximately one thousand bridges across its territory. In the basic approach, it does not authorize movement of overweight loads unless it is demonstrated that their effect is less than that of the nominal design load. When this condition is not satisfied, a formal evaluation is carried out in an attempt to assess a higher load rating for the bridge. If, after the reassessment, the rating is still insufficient, the bridge is classified as sub-standard and a formal evaluation of the operational risk is performed to define a priority ranking for future reinforcement or replacement.

Italian version

I ponti in servizio nella maggior parte dei Paesi occidentali, sono stati costruiti in accordo a normative che prevedevano carichi di progetto che al giorno d'oggi sono inconsistenti con l'attuale domanda di traffico. Oggigiorno, le agenzie di trasporto non sanno come comportarsi nel caso di richieste di transito di carichi eccezionali sui ponti di loro proprietà. Questa tesi si concentra sulle questioni giuridiche derivanti dai permessi dei carichi eccezionali emessi dalle agenzie di trasporto. Infatti, un corretto processo decisionale dovrebbe considerare le responsabilità giuridiche a seguito di eventuali eventi catastrofici. In questa tesi si illustra come questo problema è stato indirizzato nel caso del Dipartimento dei Trasporti della Provincia Autonoma di Trento (PAT), un'agenzia di medie dimensioni che gestisce circa un migliaio di ponti sul suo territorio. Nell'approccio di base, non si autorizza il transito del carico eccezionale sul ponte a meno che la sollecitazione non sia inferiore a quella prodotta dal carico nominale di progetto. Quando questa condizione non è soddisfatta, viene effettuata una valutazione formale nel tentativo di valutare un carico più elevato per il ponte. Se, dopo

questa nuova valutazione il risultato è ancora negativo, il ponte viene classificato come non conforme e viene eseguita una valutazione formale del rischio, allo scopo di definire una classifica di priorità per il suo futuro rinforzo o sostituzione.

DEDICATION

A Elena

ACKNOWLEDGEMENTS

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CONTENTS

1 INTRODUCTION

1.1 Motivation/Problem statement

Managing large infrastructure systems is a multi-disciplinary activity requiring expertise from many areas, including fields of research beyond the typical scope of the structural engineer. The process of risk analysis and decision-making itself involves knowledge of the cognitive mechanisms and the psychology of economic decisions. Mere structural reliability is just one of many aspects affecting decisions, while economic, social, ethical and legal issues must also be considered in a risk model. In recent years, transportation agencies have focused on the problem of overweight traffic management, due to the increase in the nominal load of heavy vehicles and to the increasing age and deterioration of the infrastructure.

In fact, most bridges operating today in Europe were built to old codes that foresaw design loads inconsistent with today's traffic demand, and currently transportation agencies do not know how to behave in the case of applications for transit on their bridges.

A formal re-assessment of old bridges with respect to new design codes would require analysis of design documents that may not be available and also structural recalculation; and very often expensive load tests. Because of the number of old bridges, an agency cannot normally carry the overall cost of a full analysis and will seek for simplified approaches.

In this thesis I illustrate how this problem can be managed, applying the methodology to the stock of the Department of Transportation of the Italian Autonomous Province of Trento (APT's DoT). Currently, the APT manages approximately 2410 kilometers of roadways and 953 bridges (Fig. 1).

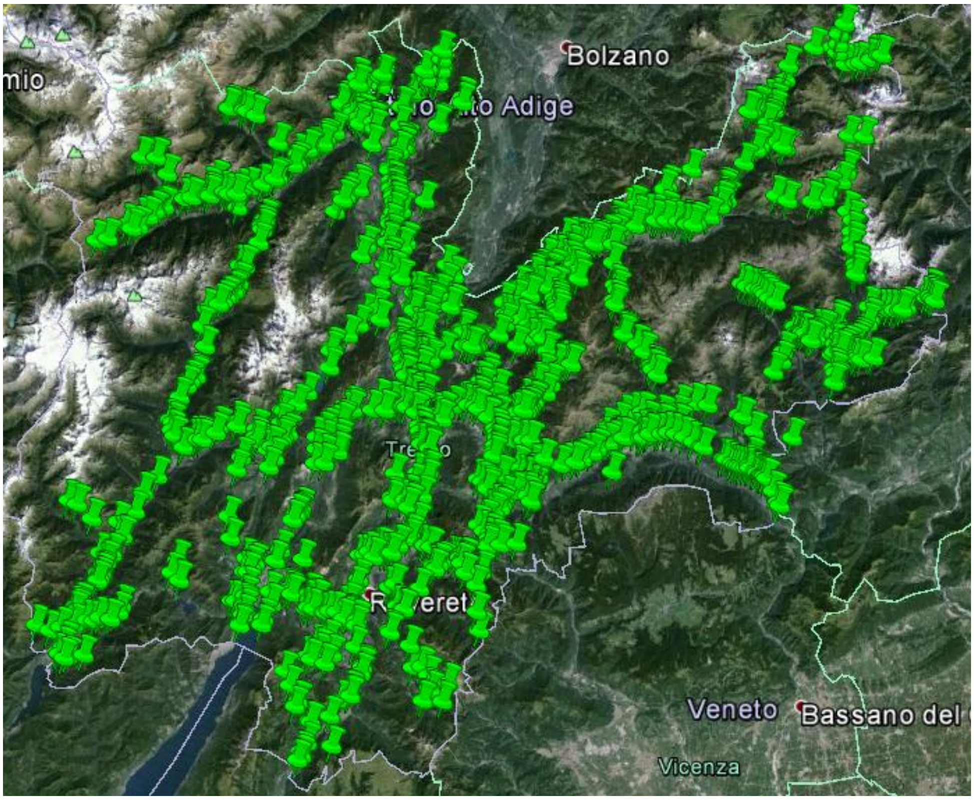


Fig. 1: APT bridge stock

With reference to type and year of construction, the APT's bridge stock has characteristics similar to those found elsewhere in Europe: most of the bridges were built after the Second World War, peaking in the 70's (Yue et al., 2010) (Fig. 2). Approximately 66% of the bridges were built of reinforced or prestressed concrete, 25% have an arch structure and 9% were built in steel or are composite steel-concrete structures (Bortot et al., 2009) (Fig. 2).

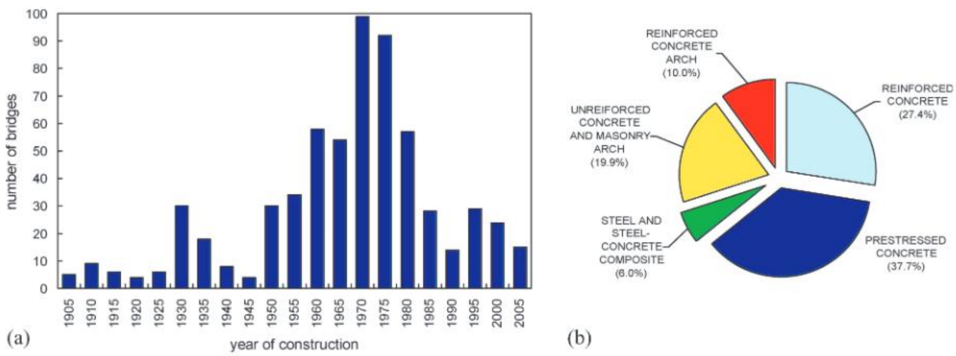


Fig. 2: a) age distribution and b) typological distribution in the APT (Zonta et al., 2007)

In order to allow effective planning of maintenance policies and to meet the new requirements, the Department of Transportation of the APT has begun to collaborate with the Department of Mechanical and Structural Engineering, University of Trento, to develop a comprehensive Bridge Management System (APT-BMS) (Bortot et al., 2009) (Fig. 3).

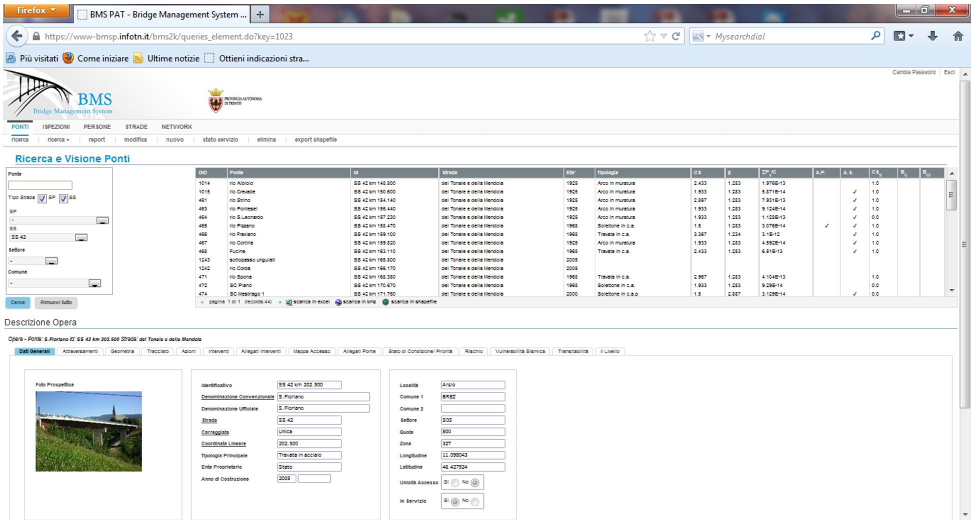


Fig. 3: Web application of APT-BMS (www.bms.provincia.tn.it)

1.2 Objectives

In this thesis I illustrate how the problem of applications for transit of overweight loads can be guided. In particular, this thesis focuses on the legal issues entailed by overweight/oversize load permits issued by transportation agencies. Indeed, correct decision-making should consider the legal liabilities involved in possible catastrophic events. The main objective is therefore to develop a decision support system for the management of the transit of the overweight traffic loads on bridges, in particular for the APT stock. This thesis provides simple, practical rules for deciding whether an overweight load permit can be issued or not, and if so, under what restrictions.

Moreover, I apply utility theory to optimize the expected cost in order to propose a process to guide the APT in making the most cost effective decision when selecting the appropriate level of verification.

1.3 Method

In general, permits for overweight vehicles are issued by transportation agencies based on the total gross weight, the maximum axle load, the distribution of axle weights and on the axle spacing (see for example Federal Highway Administration, (2012)).

One of the main problems is the high variability of axle loads and spacing. A set of predefined overweight load configurations is therefore defined that are more conservative than other axle combinations having the same gross weight. In the basic approach, it does not authorize the transit of the overweight load unless it is demonstrated that the effect is less than that of the nominal design load. It is worth emphasizing that this approach protects transportation agencies against legal liabilities in the case of a catastrophic event, because it has authorized the transit of a vehicle whose effects are less than those produced under the design assumptions. When the bridge is not verified, a formal evaluation is carried out in an attempt to assess a higher load carrying capacity for the bridge. If, after the reassessment, the capacity is insufficient, the bridge is classified as sub-standard and a formal evaluation of the operational risk is performed in order to define a priority ranking for future reinforcement or replacement.

The method is proposed as an alternative to the recent methods developed in order to solve the problem of transit permits. In fact, the permit issuing process would require a detailed structural analysis, which in turn would require very laborious analyses. Furthermore, in many cases, the geometrical data necessary to apply the analyses are not fully available. Recently, to overcome this difficulty three methods were discussed (Vigh and Kollar, 2006). The three methods are the following (Vigh and Kollar, 2006):

1) application of a simplified structural analysis with a comparison of the effects (bending moment, shear force) caused by the design loads and overweight loads (e.g. Correia and Branco, 2006);
2) comparison of the axle loads using the *bridge gross weight formula* (Bridge formula weights, 1994), or their improvements (James et al. 1986; Chou et al. 1999; Kurt 2000);
3) application of approximate transversal load distributions (Vigh and Kollar, 2007).

The three methods are discussed in detail in Section 2.2.

1.4 Outline

In this thesis I illustrate how to guide the legal issues arising from the issue of transit permits by the transportation agencies. In the next Chapter I first introduce the overweight vehicle permit problem and describe how it is currently formally managed in the APT. I then defined the travel conditions and a set of predefined overweight load configurations that are more conservative than other axle combinations having the same gross weight.

In Chapter 3, I show the multi-level assessment protocol conceived to estimate the response of the bridge stock to overweight loads. In particular, three levels of refinement of bridge assessment are foreseen to verify the bridge under overweight loads. At the end an interpretation of the risk is proposed.

Chapter 4 applies the methodology to two case studies in order to test the method. The sub-standard girder bridge *Fiume Adige* and arch bridge *Rio Cavallo* are chosen to perform the levels of refinement.

In Chapter 5 I propose a decision making process to guide transportation agencies in making the most cost effective decision. The utility theory is used to optimize the expected cost. In addition, a methodology based on Bayesian statistics is used to update parameters on the basis of a number of case studies.

Then, in Chapter 6 I apply the decision making process to the APT bridge stock in accordance with the strategic objectives to assess/retrofit the APT bridge stock. Based on the decision tree proposed, a prediction of the future demand for re-assessment and costs is presented. Finally I illustrate the optimization of the thresholds of the lack of capacity based on the results of 20 APT bridge case studies.

Some concluding remarks are made in Chapter 7.

1.5 Limitations

Although the methodology proposed in this thesis can be used by any transportation agency, the set of APT vehicle models and the decision tree proposed (see Section 2.6) have been created following the needs of the APT and the limitations of the Italian codes. Therefore, the reference loads must be changed when the DoT needs or the codes are different. However, the needs of the APT could be similar to many other transportation agencies.

2 PROBLEM STATEMENT

2.1 Introduction

In recent years, the APT's DoT has focused on the problems arising from the increase in the nominal load of heavy vehicles and the increasing age and deterioration of the infrastructure. A formal re-assessment of old bridges with respect to new design codes would require analysis of the original design documents, often unavailable, and structural recalculation; and very often expensive load tests. Because of the number of old bridges, an agency cannot normally carry these costs and will seek simplified approaches.

To re-assess the bridges taking into account the current overweight vehicles crossing them, it is firstly necessary to define representative loads (called henceforth APT loads) that are more conservative than other overweight loads having the same gross weight. This is done in order to ensure that if a bridge is sufficient for a given APT load, it is automatically adequate for any load less than or equal to the APT load.

Secondly, it is important to define the possible travel conditions of the vehicle on the bridge and the potential restrictions. This is necessary because different travel conditions can place different demands on the bridge.

Therefore, in this Chapter I define the representative loads and the travel conditions starting from the current conservative procedure adopted by APT DoT to release the transit permits.

2.2 State of the art

The increase in the number of overweight vehicles travelling on highways has caused growing interest in the policies for issuing permits (Fiorillo and Ghosn, 2014). Various works on this topic have been developed in recent years and included optimization process to obtain optimum paths (Adams et al., 2002; Osegueda et al., 1999) and procedures for the statistical categorization of overweight loads (Fiorillo and Ghosn, 2014, Correia et al., 2005; Fu and Hag-Elsafi, 2000). As mentioned in Section 1.3, three main methods were recently developed to manage transit permits. Other studies on permit checking of overweight vehicles were discussed by Fu and Moses (1991), Ghosn (2000), Bakht and Jaeger (1984), Ghosn and Moses (1987). Phares et al. (2004) presented a procedure for permit checking of overweight vehicles based on the bridge load rating using experimental testing.

The first method (Correia et al., 2005) compares the internal forces (bending moment, shear stress, normal force) caused by design code loads and overweight vehicles. The methodology requires the knowledge of the transversal section and longitudinal model of the bridge. Four transversal sections and three longitudinal models can be chosen for the analyses (Fig. 4). The software BIST (Bridge Investigation for Special Trucks) developed by Branco and Correia (2004) calculates, on the basis of the transversal section and longitudinal model chosen, the transversal distribution load factors and the influence lines necessary for the comparison of the effects of the design load with those caused by the overweight vehicle.

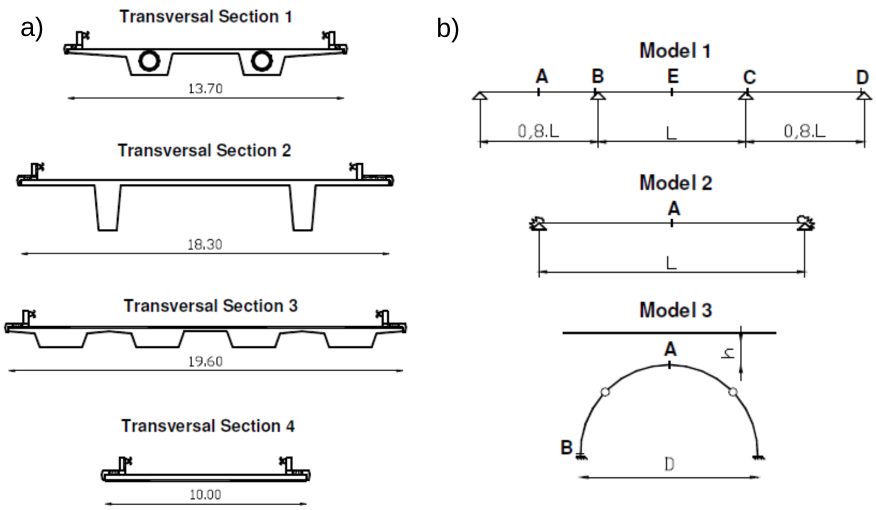


Fig. 4: a)Transversal sections and b)longitudinal model considered by software BIST (Correia and Branco, 2006)

The analyses are performed by placing the loads in each critical section. A simplification regarding overweight loads is assumed and consists in considering each group of wheels of the vehicle, which is composed of 7 axles. All the possible combinations of positions of the loads are tested to determine the maximum effects in each critical section.

The method calculates the structural safety of the bridge by defining two simplified safety factors (SSF) evaluated in the critical sections, one for bending moment and the other for supporting reactions (1).

$$SSF = \frac{\gamma_S \cdot S_{(code\ loads)}}{1 \cdot S_{(overweight\ loads)}} \tag{1}$$

where γ_s (usually equal to γ_s=1.5) is the safety factor of the code loads, and $S_{(code\ loads)}$ and $S_{(overweight\ loads)}$ are the load effects caused by design loads and overweight loads respectively. If SSF>1.5, the structural safety of the bridge is greater than that defined in the design code, and so the crossing of the bridge is safe. Correia and Branco (2006) states that a lower safety factor for the overweight load may be adopted, since the exact value of the action is known. As a consequence, the Portuguese Expressway Authority (*Brisa*) allows the crossing of the bridge with a factor of up to γ_s=1.1, while if γ_s is between 1.1 and 1.0, the engineer must decide whether to issue the permit by evaluating the bridge conditions.

The second method, developed by FHWA (1994), estimates the equivalent load of a generic overweight vehicle. The formula is adopted by the National System of Interstate and Defense Highways of the U.S.. The generic vehicle is divided into groups of axles and the equation provides the allowable weight for each group (Kurt, 2000). The Federal Bridge Formula is the following:

$$W = 500\left[\frac{L \cdot N}{N-1} + 12N + 36\right] \tag{2}$$

where W is the overall gross weight on any group of 2 or more consecutive axles, L is the length of the group, and N is the number of axles in the group.

The allowable gross weight for the overweight vehicle is equal to the sum of the allowable weights for each axle group (Kurt, 2000). This formula requires many calculations that depend on the number of axles of the vehicle. In addition, the equation does not take into account the total gross weight and the total length of the vehicle, and can therefore be inaccurate especially for long span or multispan bridges (Vigh and Kollar, 2007). Numerous researchers (Meyerburg et al. (1996), Nowak et al. (1994), Nowak et al. (1991), Harmon (1982)) have observed that the overall length of many trucks and the number of axles have significantly increased since the original bridge formula (2) was developed. This means that the allowable weight has significantly increased. For this reason numerous studies to improve the procedure for issuing transit permits have been done (e.g. Kurt, 2000; Chou, 2005; Ghosn, 2000).

The third method (Vigh and Kollar, 2007) proposes the application of approximate transversal load distribution to estimate the effects of the loads. The principle of the method is the following:

The bridge is safe if the internal forces caused by the overweight vehicle are smaller than those caused by the design load vehicle (Vigh and Kollar, 2007) (3).

$$n = \frac{E_{design\ load}}{E_{overweight\ load}} \tag{3}$$

where n is the safety of the bridge, and $E_{design\ load}$ and $E_{overweight\ load}$ are the effects of the design load and overweight load respectively. If $n \geq 1$ the overweight vehicle can cross the bridge.
The method also takes into account the different location of the load through the width of the bridge. In fact, Vigh and Kollar (2007) consider the following travel conditions:

- the overweight load is a part of the traffic and crosses close to the exterior of the bridge;
- other traffic loads are not permitted and the overweight load crosses close to the exterior of the bridge;
- the road is closed to free traffic and the overweight load crosses in the center of the roadway.

The methodology calculates the load that acts in an arbitrary position through the width of the bridge (P_η), multiplying the load (P) by the distribution factor η (Goodrich and Puckett, 2000).

$$P_\eta = \eta \cdot P \tag{4}$$

The distribution factors are analogous to the transverse load distribution. Several methods are available in the literature for the calculation of these coefficients (Massonnet & Bares, 1968; Guyon, 1946; Courbon, 1950). When the transverse load distribution coefficients are known, the method suggests that they be used, otherwise it recommends applying two transverse load distributions which are upper and lower approximations of the accurate transverse load distributions (Fig. 5). One of these, which is unknown, is a conservative estimation of the transverse load distributions. Vigh and Kollar (2007) report the recommended value of these limits for four different bridge typologies (two girder system, multiple girder system, solid slab, box). These values were obtained by Janko (1998) by considering several real bridge superstructures.

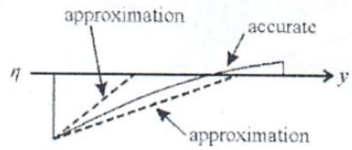


Fig. 5: Upper and lower approximation of transverse load distributions (Vigh and Kollar, 2007)

The method points out that since a comparison among loads is performed, the numerical value of the transverse load distributions are irrelevant and only the shapes are important.
The main advantage of the permitting procedure based on this last method, is the minimal input data of the bridge, which are:

- the span of the bridge;
- the width of the bridge;
- type of superstructures of the bridge.

2.3 Current procedure

Nowadays the conservative procedure adopted by the APT's DoT foresees three possible transit restrictions based on the total gross weight of the overweight loads:

1) **Free travel up to 44 tons**: the vehicle can travel freely with no traffic restriction – but restrictions on time and number of trips may be applied;
2) **Travel with traffic restriction and vehicle speed limit to 30 km/h up to 60 tons**: the road is closed to free traffic, the vehicle is required to cross the bridge in the center of the roadway and the maximum vehicle speed is 30 km/h.
3) **Travel with traffic restriction and vehicle speed limit to 10 km/h beyond 60 tons**: the road is closed to free traffic, the vehicle is required

to cross the bridge in the center of the roadway and the maximum vehicle speed is 10 km/h.

These restrictions are based on empirical information from past crossing of loads that caused no apparent damage to the bridges. However, these restrictions do not consider the real capacity of the bridges. In the next sections I analyze critically this categorization of restrictions and in Chapter 3 I explain a simplified method that takes into account the capacity of the bridge.

The possible methodologies that can be considered in the analyses are reported in the following. The differences among the methodologies are in the safety level that the manager wants to obtain and accept.

1) The overweight load does not cause effects that are more severe than the original bridge design code loading (the condition satisfies both the substantial and formal requirements of the design code);
2) the overweight load causes effects that are more severe than the original bridge design code loading but the exact knowledge of the total gross weight of the overweight load leads to an estimation of a reliability index equal to or higher than that foreseen by the design code (the condition satisfies the substantial but not the formal requirements of the design code);
3) the manager accepts that the overweight load crosses the bridge with a reliability index lower than that foreseen by the design code (that is, with a higher risk), (the condition does not satisfy either the substantial or formal requirements of the design code).

Table 1 shows the possible methodologies of analysis and travel conditions. The aim of the present thesis is to analyze only the methodology that satisfies both the substantial and the formal requirements of the design code (first row of Table 1) because the APT wants to be protected by legal liabilities involved in possible catastrophic events. The methodologies that do not satisfy the formal requirements of the design code are not discussed in the present work (second and third row of Table 1).

Table 1: Possible methodologies of analysis and travel conditions

Model / Methodology	Free travel	Bridge crossed in the center of the roadway	Limitation of the speed
As per design code	YES	YES	YES
With the principle of the design code	NO	NO	NO
Not accepted by the design load	NO	NO	NO

2.4 Effect of vehicle velocity

In the categorization explained in Section 2.3, the travel restrictions on overweight loads are based on the vehicle speed. This is justified by assuming that a higher speed entails a higher dynamic response coefficient. In fact, since the demand on the bridge caused by a dynamic load is higher than the same load applied statically, intuitively one can think that a higher speed entails a higher dynamic response coefficient of the bridge. An examination of the technical literature (Cantieni 1983, Paultre et al. 1992, Brady et al. 2006, Senthilvasan et al. 2002) shows that in reality the speed of an overweight vehicle is not directly proportional to the dynamic coefficient, which depends more on other factors such as (Cantieni 1983):

- the length of the bridge;
- the roadway irregularities;
- the stiffness of the deck;
- the dynamic features of the vehicle.

There is no sense in limiting the speed of the overweight load, if there is no evidence that by reducing speed, the dynamic coefficient and consequently the stress on the bridge are also reduced. Moreover, under certain conditions, a lower speed causes higher stress on the bridge (*Fig. 6*).

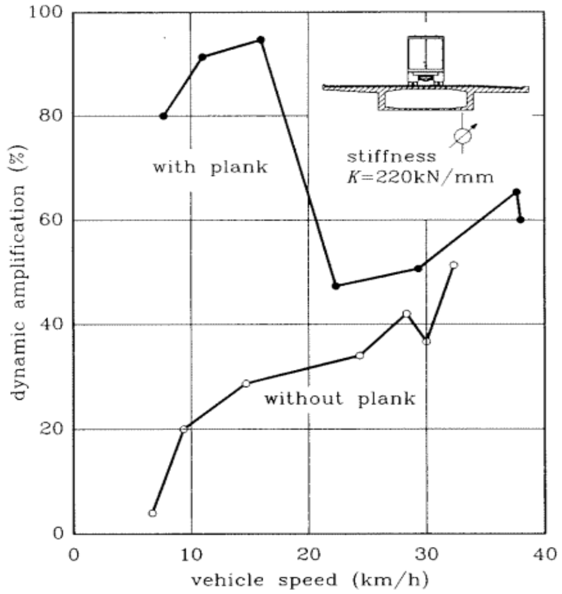


Fig. 6: Amplification effect as a function of the vehicle velocity (Cantieni, 1983)

2.5 Travel conditions

Based on the observations in Section 2.4, I decided to disregard speed restrictions, and to assume the following two travel conditions to be used in the transit permit issuing process:

1) free travel;
2) road closed to free traffic and bridge crossing in the center of the roadway without speed limits.

As discussed above, the limitation of the speed produces no benefits and so will not be taken into account in the discussion. Therefore, in accordance with Section 2.3, the possible methodologies of analysis and travel conditions become those shown in Table 2.

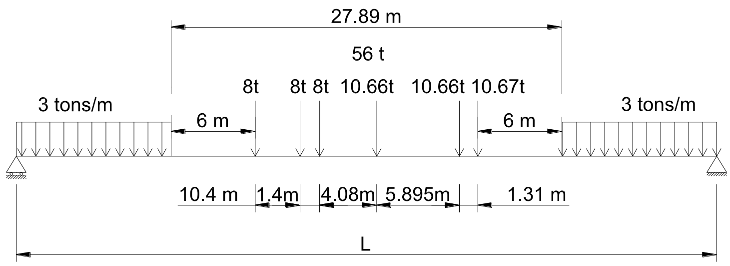
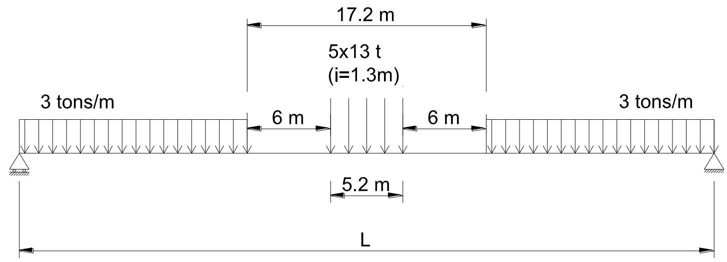
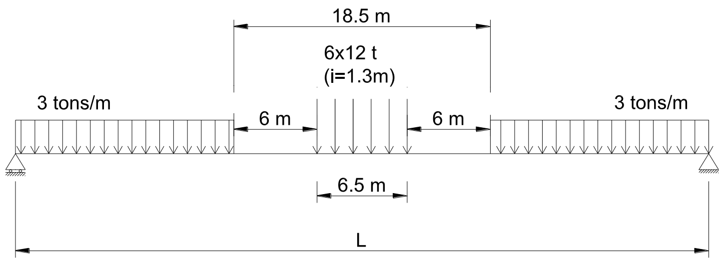
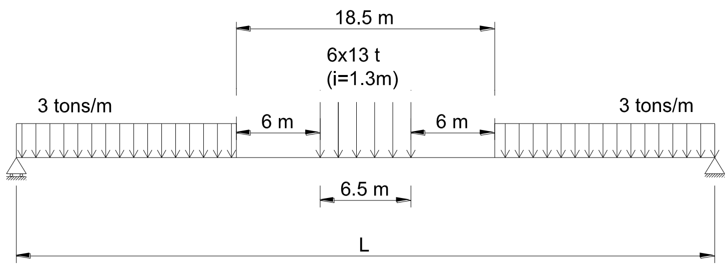
Table 2: Possible methodologies of analysis and travel conditions considered in the present thesis

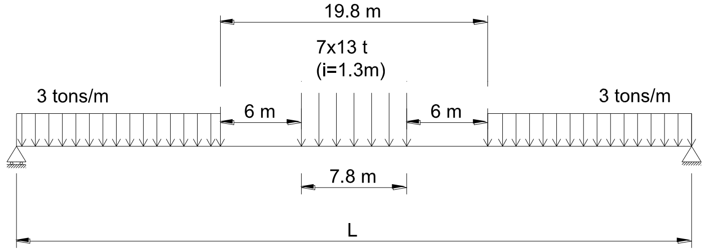
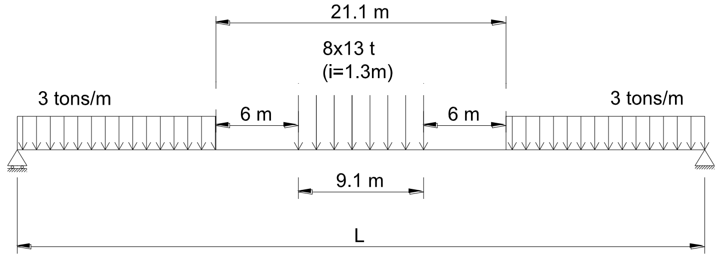
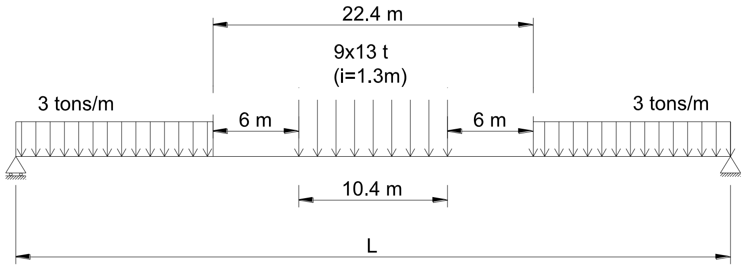
Model / Methodology	Free travel	Bridge crossed in the center of the roadway	Limitation of the speed
As per design code	YES	YES	NO
With the principle of the design code	NO	NO	NO
Not accepted by the design load	NO	NO	NO

2.6 APT overweight vehicle models

In general, permits for overweight vehicles are issued by transportation agencies based on the total gross weight, the maximum axle load, the distribution of axle weights and the axle spacing (see for example Federal Highway Administration (2012)). One of the main problems is the high variability of axle loads and spacing. A set of predefined overweight load configurations, the APT loads, is therefore defined; these configurations are more conservative than other axle combinations having the same gross weight. If a bridge is sufficient for a given APT load, it is automatically adequate for any load less than or equal to the APT load. Currently, the Italian highway code (D.L. 30/04/1992) places the following limits on the free movement of vehicles: maximum total weight 44 tons (440 kN); maximum axle load 13 tons (130 kN); minimum axle spacing 1.3 m. The APT requires that these axle weight and spacing limits be respected for extra-legal vehicles, whatever their gross weight. Therefore, the APT load model reproduces a multi-axle load in the most unfavorable configuration considering different gross weights: a set of 130kN concentrated loads spaced at 1.3 m, applied on a 3.0 m wide lane. Similarly to the provisions of the 1990 Italian design code for bridges (D.M. 04/05/1990), the overweight vehicle model is applied in conjunction with a uniformly distributed load of 30kN m^{-1}, 6 m ahead of the first axle and 6 m behind the last axle. In addition the APT DoT required that two additional models be taken into account: 6 axles by 12 tons = 72 tons (720kN) and 56 tons (560kN) due to specific real overweight loads. All APT loads are shown in Table 3. The concentrated forces represent the axles of the overweight load, and the uniformly distributed load of 3 t/m represents the traffic in front of and behind the overweight load.

Table 3: APT loads

APT Load	APT Load pattern
56 tons	
5 axles by 13 tons 65 tons	
6 axles by 12 tons 72 tons	
6 axles by 13 tons 78 tons	

7 axles by 13 tons 91 tons	
8 axles by 13 tons 104 tons	
9 axles by 13 tons 117 tons	

2.7 Conclusions

The objective of the APT DoT is to provide simple and practical rules for deciding whether an overweight load permit can be issued or not, and if so, under what restrictions. The current procedure of the APT foresees conservative limits and takes into account the vehicle velocity. Examining the technical literature it is clear that there is no sense in limiting the speed of the overweight load as there is no evidence that by reducing speed, the stress on the bridge is reduced. From these considerations only two travel conditions are possible: free travel and crossing the bridge in the center of carriageway.

Representative loads (APT loads), which are more conservative than other overweight loads having the same gross weight, are defined. If a bridge is sufficient for a given APT load it is automatically adequate for any load less than or equal to the APT load. Following this approach it is not necessary to verify a bridge every time a different load distribution is acting. The definition of the representative loads has been reached by analyzing the current Italian code and considering the minimum axle spacing, and the maximum axle load of the vehicles foreseen by the code.

3 CRITERIA FOR RE-ASSESSING BRIDGES FOR OVERWEIGHT LOADS

3.1 Introduction

In principle, assessing a bridge to any of the APT load models and travel conditions would require full formal re-evaluation of its capacity, carried out by a professional engineer. In practice, direct re-assessment of the 1000 bridges in the territory is a very expensive task, and it is in many cases unnecessary. To guide the cost issue, the approach followed is to estimate the capacity of the stock using a simplified and conservative approach first, and then refining the analysis only if a higher load rating is required.

In this Chapter I show the procedure to evaluate whether a bridge can withstand the APT loads defined in Chapter 2. In particular, I illustrate the multi-level assessment protocol conceived to estimate the response of the bridge stock to overweight loads.

3.2 Assessment principle

The assessment procedure includes four simplified levels of refinement, from Level 0 to Level 3, as summarized in Table 4. All methods are based on the following principle:

The bridge is rated for an overweight load if it is demonstrated, even conservatively, that the overweight load does not cause effects that are more severe than the original bridge design code loading.

In practice the procedure aims to demonstrate that the new overweight load condition is not worse than the most critical load condition associated with the original design assumption or as-built situation, and therefore the overweight load does not reduce the original design safety level. It is believed that this principle protects the APT against legal liability in the case of a failure, as following this principle the APT authorizes travel only of those vehicles whose effects are less than or equal to those expected under the design assumptions.

Table 4: Level of refinement of bridge assessment under overweight loads

Assessment Level	Capacity Models	Calculation Models
Level 0	Bridge is assumed verified with no overstrength	Statically determinate condition is assumed
Level 1	As per design	As per design
Level 2		Refined model; load redistribution is allowed, provided that the ductility requirements are fulfilled.
Level 3	Material properties can be updated based on in-situ testing and observations	

The fundamental hypothesis of the assessments is reported in the following:

- the contemporary presence of more than one overweight load on the bridge is not allowed.

In the case of free travel we evaluate that the condition above is still valid, because the contemporary presence of more than one overweight load is unlikely. Moreover, if this situation of more than one vehicle is considered, the result of the method would be too conservative. The hypothesis is automatically satisfied in the case of bridge crossed in the center of the roadway.

3.3 Levels of assessment

As summarized in Table 4, three levels of refinement of bridge assessment are foreseen to verify the bridge under overweight loads.

Level zero assessment is a conservative estimate of the bridge capacity based on its geometry and the design code it was originally designed to. The fundamental hypothesis of Level 0 assessment is reported in the following:

- the bridge was built exactly to the nominal design load (in other words, with no overstrength).

The bridge is automatically rated for the overweight load if it can be demonstrated that the stresses it creates are everywhere lower than or equal to those considered at the design stage.
The APT approach:

1) replaces the design traffic lane with the APT load;
2) evaluates the difference in demand between the new and old loadings.

If the stress under the APT load is lower than the design load stress, it is logical to infer that the bridge is able to withstand the APT load. For girder bridges, shear and bending moment in the deck are assessed.
The basic assumption in Level 0 assessment is that the bridge was built with no shear or moment overstrength at any section of the deck: this assumption is extremely conservative and normally not realistic. Bridge members are normally slightly oversized and very often the designer assumes conservative capacity models. In addition the mechanical properties of the materials in the as-built situation could be better than those specified by the designer. When a bridge is not sufficient according to Level 0 assessment, the assessment proceeds to a higher level of refinement and is carried out by a professional engineer based on analysis of the available design documentation.
At Level 1 the evaluator is required to examine the design documents to verify whether the critical structural members are oversized. This level of assessment is based only on analysis of the existing documents, without revising assumed material properties or the calculation model.
At Level 2 the evaluator reassesses the bridge capacity using more refined models than those used by the original designer. Often, a capacity increase is achieved by considering inelastic behavior and spatial stress redistribution.
At Level 3 the evaluator can update the characteristic values of the variables used in the assessment, based on the results of material testing and observations. The procedure leaves the evaluator free to test the materials, without specifying the minimum number of samples or the type of test. However, if the evaluator wants to use the test results quantitatively, a Bayesian probabilistic update technique should be applied. This can be done even if the design documentation is not available: in this case, an accurate survey of bridge geometry and extensive material sampling are needed.

3.4 Procedures for assessment

Although the definition of levels of assessment is general, the procedure for assessment is different in the case of girder bridges and arch bridges. The reason lies in the different resistance mechanisms of the two typologies. For arch bridges, in contrast with girder bridges, the capacity is independent of the material strength (for example the compressive strength of the masonry) but depends on the equilibrium of the forces acting upon it. In fact, if the form of the arch bridge is equal to the funicular form generated by the external forces, its capacity tends to infinity. Therefore, in Section 3.4.1 and 3.4.2 the procedure for assessment of girder and arch bridges is shown separately. In these sections only the procedure of Level 0 assessment is reported. The other levels depend on the specific verifications of the bridge and these are shown in the case studies of Chapter 4.
In the assessment levels we assume that the widths of the design load and the APT load are identical (as suggested also by Vigh and Kollar, 2006). The Italian codes (see for example Circolare 1962 or Circolare 1952) foresee two typology of design loads:

- civil loads (width=3.0m);
- military loads (width=3.5m).

Therefore, in our case, the width of the APT load can be 3.0m or 3.5m.

3.4.1 Girder bridges

The procedure for assessment involves an estimation of the capacity of the bridge, using Level 0 assessment first, and then refining the assessment level if a higher assessed capacity is required. Level 0 analysis is simple for the free travel condition. In this case the design situation is compared to a load condition where one traffic lane is replaced with the APT load. As discussed above, the method is based on the following principle:

The bridge is rated for an overweight load if it is demonstrated, even conservatively, that the overweight load does not cause effects that are more severe than the original bridge design code loading.

Since the method compares effects, the following terms can be neglected in the calculations:

- dead load of the bridge;
- live loads outside the lane replaced.

The reason is that these loads are the same for both the overweight and the APT loadings, and so their effects are irrelevant to the evaluation and can be disregarded. In fact, if S_0 is the demand caused by the overweight load, which can represent for example the bending moment or shear stress, S_{APT} is that caused by the APT load, and G is that caused by the dead load, the increase in the demand ΔS is independent of the dead load, and is equal to (5):

$$\Delta S = (S_{APT} + G) - (S_0 + G) = S_{APT} - S_0 + G - G = S_{APT} - S_0 \quad (5)$$

Similarly, the effects of live loads outside the lane replaced are identical in the two situations, so these live loads can also be disregarded too. In fact, only one traffic lane is replaced by the APT load and consequently the demand caused by any live loads is the same in the two situations (6).

$$\Delta S = (S_{APT} + S_2 + S_3) - (S_0 + S_2 + S_3) = S_{APT} - S_0 + S_2 - S_2 + S_3 - S_3$$

$$\Delta S = S_{APT} - S_0 \quad (6)$$

where S_1 and S_2 are the demands caused by the second and the third traffic lane respectively.
In contrast, partial factors (γ) or dynamic coefficients (Φ) considered at the design stage influence the increase in the demand (7).

$$\Delta S = [G \cdot \gamma_G + (S_{APT} \cdot \gamma_S + S_2 \cdot \gamma_S + S_3 \cdot \gamma_S) \cdot \Phi]$$

$$-[G \cdot \gamma_G + (S_0 \cdot \gamma_S + S_2 \cdot \gamma_S + S_3 \cdot \gamma_S) \cdot \Phi] = \gamma_S \cdot \Phi \cdot (S_{APT} - S_0) \quad (7)$$

However, these values do not change the sign of the result but only its value. This means that the result of the verification (positive or negative) is identical even if partial factors or dynamic coefficients are not taken into account. For this reason the comparison is conducted using the nominal load value, disregarding any another coefficient. If we need to quantify the extent of bridge understrength with respect to the overweight load as absolute value, it is also necessary to consider the partial factors or the dynamic coefficients. In contrast, if this deficiency is evaluated as a relative value (*ΔS/S0*), as in Section 3.5, these coefficients can also be neglected (8).

$$\frac{\Delta S}{S_0} = \frac{[G \cdot \gamma_G + (S_{APT} \cdot \gamma_S + S_2 \cdot \gamma_S + S_3 \cdot \gamma_S) \cdot \Phi]}{\gamma_S \cdot \Phi \cdot S_0}$$

$$- \frac{[G \cdot \gamma_G + (S_0 \cdot \gamma_S + S_2 \cdot \gamma_S + S_3 \cdot \gamma_S) \cdot \Phi]}{\gamma_S \cdot \Phi \cdot S_0} = \frac{\gamma_S \cdot \Phi \cdot (S_{APT} - S_0)}{\gamma_S \cdot \Phi \cdot S_0} \quad (8)$$

$$\frac{\Delta S}{S_0} = \frac{(S_{APT} - S_0)}{S_0}$$

Moreover, when the shape of the design load and APT load are similar, the static scheme of the bridge can also be neglected in the analyses.
Let us consider a simply supported beam and a fixed beam as shown in Fig. 7 and in Fig. 8.

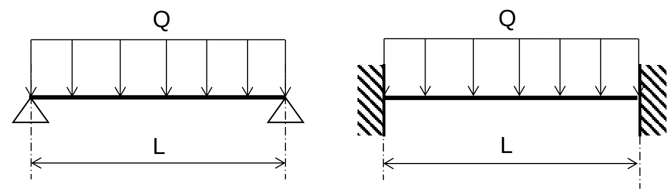


Fig. 7: Distributed load on a)simply supported beam and b)fixed beam

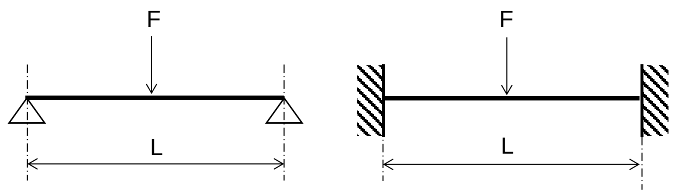


Fig. 8: Concentrated load on a)simply supported beam and b)fixed beam

The diagram of the bending moment of the fixed beam can be seen as a shift of that obtained for the simply supported beam. This is valid not only for distributed loads but for each type of load (e.g. concentrated loads) (Fig. 9 and Fig. 10).
This means that when the original design load produces static stresses higher than the new overweight vehicle in a simply supported span, the design stresses will be higher with a statically indeterminate boundary condition. Since the method

foresees a comparison between the effects caused by design and overweight loads, the static scheme, whether continuous or simply supported, is irrelevant, so a simply supported condition can always be assumed.

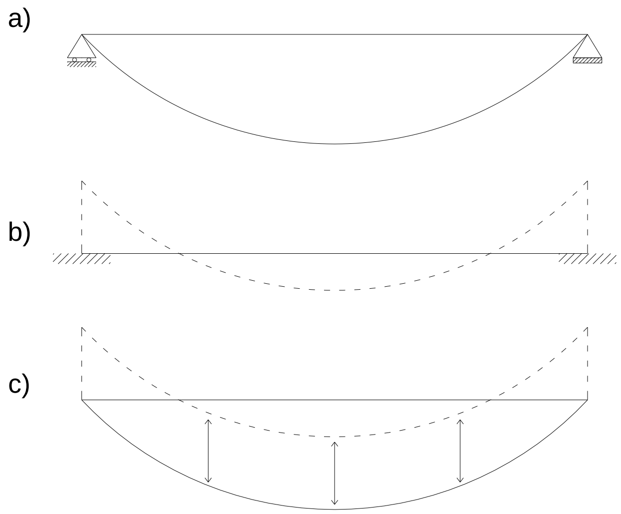


Fig. 9: Bending moment diagram caused by a distributed load in the case of a)simply supported beam, b)fixed beam, c)comparison.

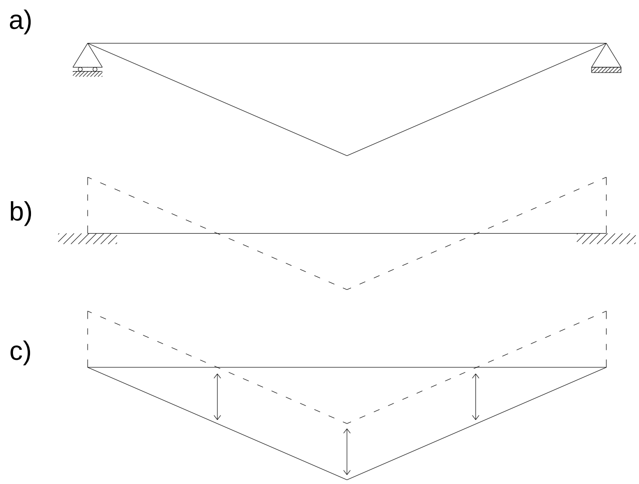


Fig. 10: Bending moment diagram caused by a concentrated load in the case of a)simply supported beam, b)fixed beam, c)comparison.

In conclusion, in the case of free travel of an overweight load, the assessment is made by comparing the effects of the original design and APT loads on a single lane simply supported beam.

The parameters that control the assessment are:

- the span length L,
- the design code adopted by the designer,

which in turn is related to the date of construction of the bridge.
In practice, Level 0 assessment foresees the calculation, for each section of the beam, of the maximum bending moment and shear caused by the design load. Subsequently, the design year of the bridge must be known, to obtain the design loads provided by the design code. Then the same procedure is applied to the APT load. If in all sections of the deck the bending moment and the shear caused by APT load are equal to or lower than those caused by the design load, the verification is positive (Fig. 11).

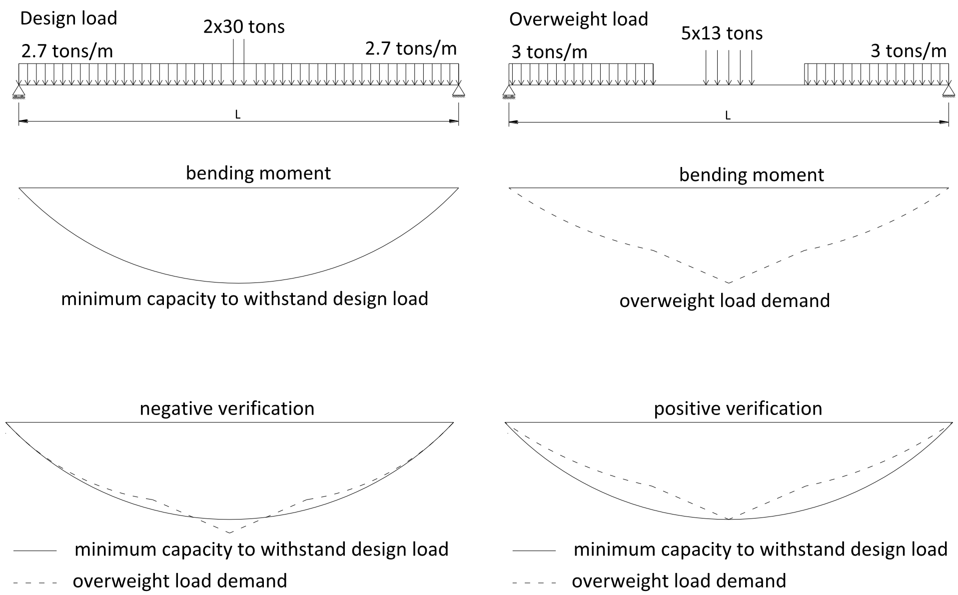


Fig. 11: Example of Level 0 assessment

In the case of restricted travel, the effects from the original traffic lanes must be compared to the single lane APT load applied in the center of the carriageway. In this case, to compare the effects, we must take account of the transverse load redistribution between the various girders using, for example, the Massonnet-Guyon-Bares theory (Massonnet C., & Bares R., 1968).

Fig. 12 illustrates the procedure for determining whether the effect of the overweight load centered on the carriageway is more or less critical than any combination of design traffic lanes. Fig. 1a shows the cross section of the bridge deck loaded with a two-lane design condition, with the Massonnet-Guyon-Bares transverse distribution coefficients multiplied by the corresponding load highlighted. The effect of the second design load configuration is illustrated in Fig. 12b, while Fig. 12c shows the envelope of design conditions. The maximum allowable APT load, applied in the center of the carriageway, is that which has effects less than or equal to the envelope of the design load effects (Fig. 12d). Therefore, knowing the transverse distribution coefficients of the design load, the maximum allowable APT load in the center of the carriageway can be determined for any deck width or any bridge.

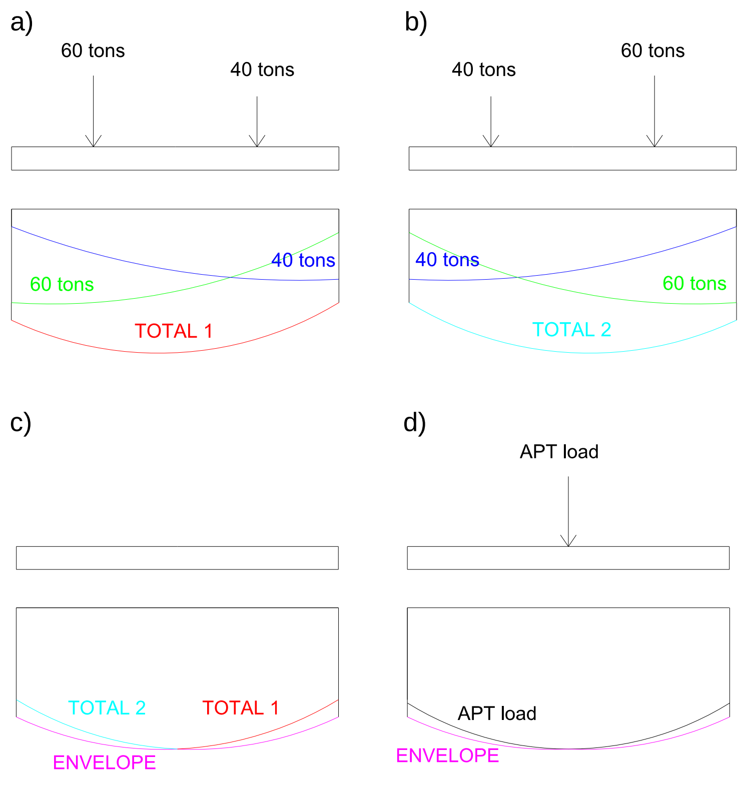


Fig. 12: a) Massonnet-Guyon-Bares coefficients multiplied by the relative load, first load design condition b) second load design condition, c) envelope of design condition, d) comparison with the effect of an APT overweight load.

In summary, in the case of restricted traffic, Level 0 assessment requires knowledge of:

- the transverse distribution coefficients of the design load;
- the span length;
- the design code.

As an alternative to the Massonnet-Guyon-Bares method, the Courbon method can be used to calculate the transverse distribution coefficients of the design load. The Courbon method is a specific case of the Massonnet-Guyon-Bares method (Petrangeli, 1996) in which the hypotheses are the following:

- the torsional stiffness of the deck is equal to zero;
- the longitudinal stiffness of the deck tends to infinity.

The method is not always in favor of safety, and therefore the final analyses should be performed using Massonnet-Guyon-Bares method. However, the Courbon method is a valid tool to estimate the lack of capacity of bridges, and with this aim the results shown in Section 6.4 were obtained using this method.
When evaluating a bridge deck made with a concrete slab, the Courbon coefficient is equal to (9):

$$r(x) = \left(1 + \frac{6e}{L} - \frac{12ex}{L^2}\right) \tag{9}$$

where $r(x)$ is the transverse distribution coefficient as a function of the transverse coordinate x (x=L/2 in the center of the carriageway), e is the load eccentricity from the center of the carriageway, and L is the width of the deck.
Fig. 13 shows the Courbon transverse distribution of load as a function of one moving concentrated force along the transverse section of the deck slab.
It can be noted from equation (9) and Fig. 13 that $r(x)$ is equal to one in the center of the carriageway (x=L/2) for each value of eccentricity e. This means (Fig. 14) that the minimum value of the envelope caused by a number of loads is in the center of the carriageway. Moreover it can be noticed that the minimum value of the envelope is equal to the sum of the loads. In the case of a bridge crossed in the center of carriageway, the allowable APT load is equal to the sum of the design loads (Fig. 14).

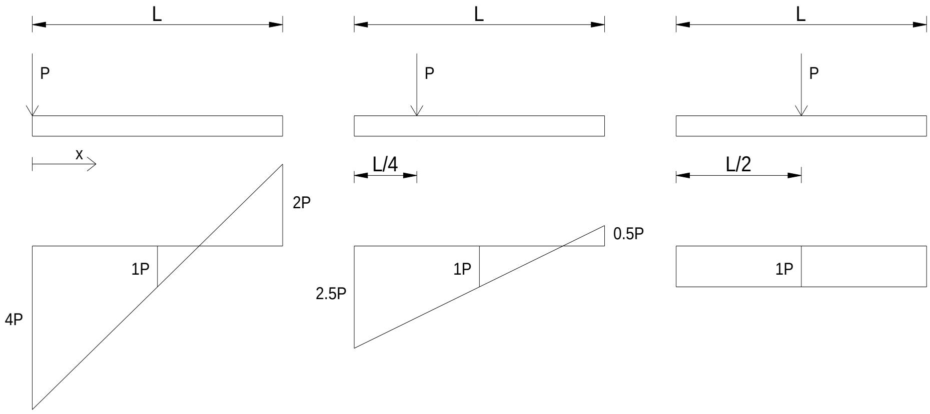


Fig. 13: Courbon transverse distribution of load as a function of one moving concentrated force

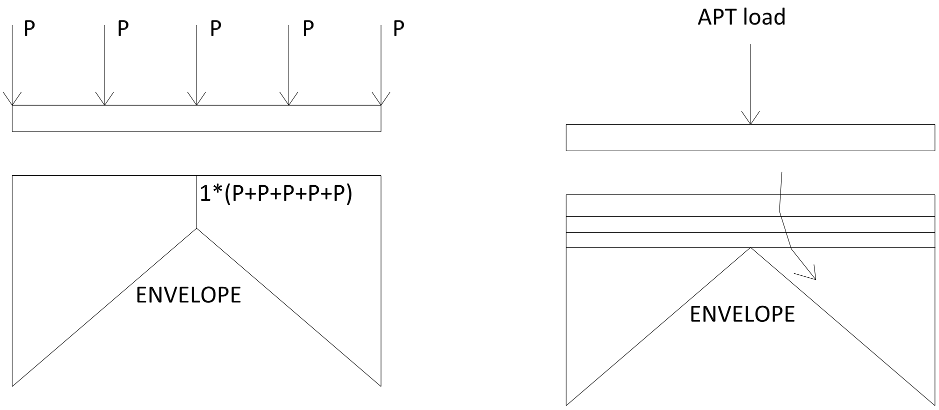


Fig. 14: Maximum allowable APT load in the case of bridge crossed in the center of carriageway

$$APT_{load} = sum\ of\ design\ load = 1 \cdot (P + P + P + P + P) = 5P \qquad (10)$$

Equation (10) is valid only in the case of absence of the traffic divider. When a traffic divider is present the bridge is crossed in the center of the semi-carriageway and it is necessary to use the formulation explained in Section 3.4 replacing the Massonnet-Guyon-Bares coefficients with the Courbon coefficients (Fig. 15).

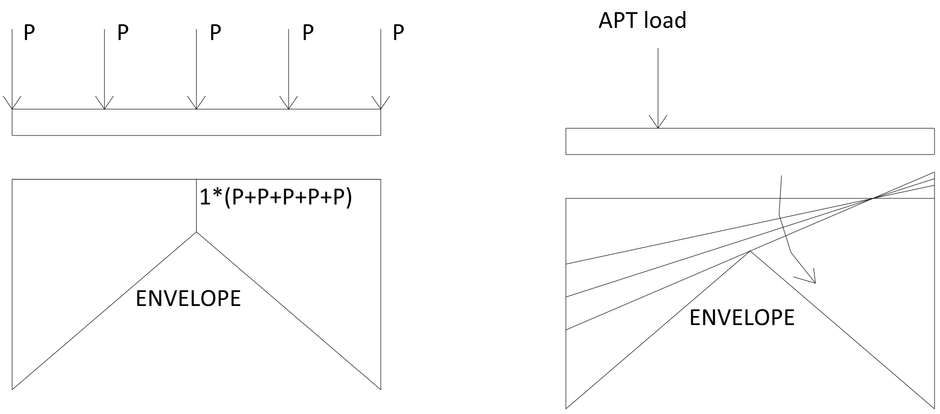


Fig. 15: Maximum allowable APT load in the case of bridge crossed in the center of carriageway with traffic divider

3.4.2 Arch bridges

For arch bridges, the approach shown in Section 3.4.1 has to be modified as the resistance mechanisms of the two types of bridges is different. For girder bridges, the procedure involves comparing, for each section of the beam, the maximum bending moment and shear caused by the design load and by the APT load. This procedure cannot be implemented for arch bridges because their capacity depends only on the equilibrium of the forces acting upon them (Heyman, 1982). The principle defined in Section 3.2 is still valid but the approach must be changed. In fact, the capacity of the arch element is independent of the material strength but depends on the equilibrium of the forces acting upon it. If the form of the arch bridge is equal to the funicular form generated by the external forces, its capacity tends to infinity. This means that the capacity depends on the form of the load (axle spacing and axle loads) and not on the total gross weight (Fig. 16). Fig. 16 shows thrust lines generated by a uniform load q and q+q by supposing the form of the arch equal to the funicular form (in this case a parabolic shape). It can be noticed that in both cases the thrust line is identical.

The verification of the arch element can be performed by checking that the thrust line is always within the middle third. This ensures that the cross-sections of the arch are always compressed. Heyman, (1982) stated that to verify the arch element it is sufficient demonstrate that there exist a satisfactory line of thrust that balances the given loading. In other words it is sufficient to find a thrust line in equilibrium with the external loading *which lies everywhere within the arch ring* (Heyman, 1982). This principle is used in Level 2 assessment, where the equilibrium analysis of the arch is performed.

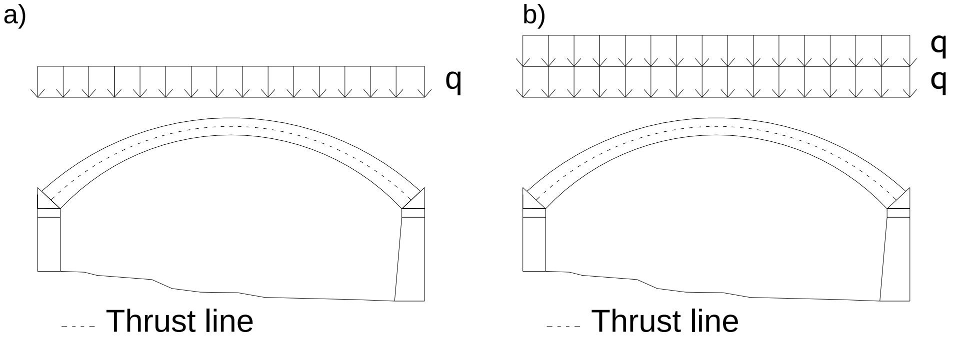


Fig. 16: a)thrust line generated by a uniform load q, b)thrust line generated by a uniform load q+q

For Level 0 assessment it is sufficient to compare the eccentricity of the thrust line caused by the design load with that caused by the APT load. If in each section of the arch, the eccentricity caused by the design load is greater than that caused by the APT load, the bridge is safe. In fact, the eccentricity caused by the design load represents the minimum thickness of the arch. An example of Level 0 assessment for arch bridges is reported in the following.

In the voussoir arch of Fig. 17, the connections between the arch rib and the abutments can be considered made by frictionless pins as reported in the literature (for example Heyman, 1982).

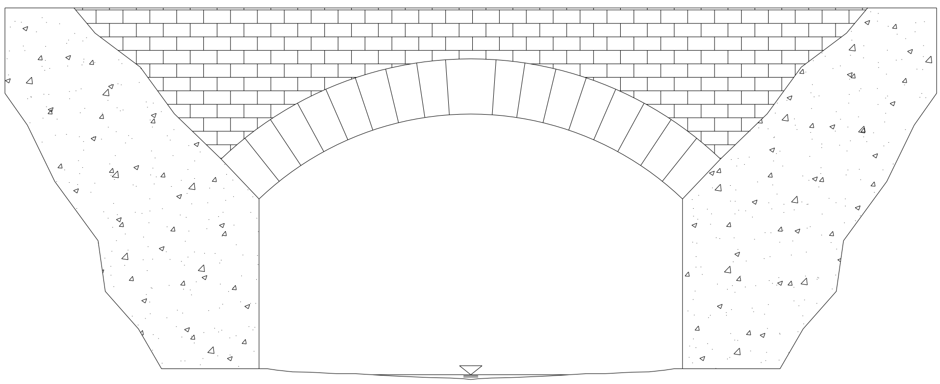


Fig. 17: Side view of voussoir arch bridge

If the loading F is located in the correspondence of the crown (Fig. 18), the total bending moment M can be determined by performing the difference between the bending moment caused by external forces ($R{\cdot}x$) and by the abutment thrust ($H{\cdot}y$) (11).

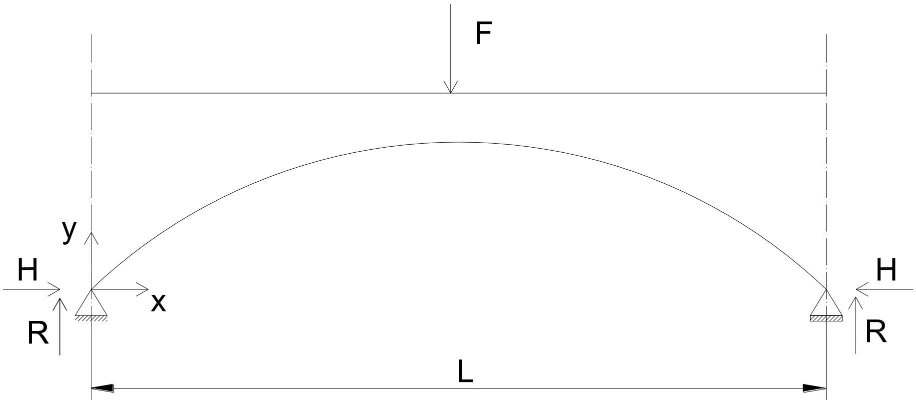


Fig. 18: Schematization of the arch bridge

$$M = -H \cdot y + R \cdot x \tag{11}$$

This means that the total bending moment (Fig. 19c) can be determined graphically by minimizing the difference between the bending moment caused by external forces (Fig. 19a) and a moment with the same shape of the arch (Fig. 19b). In fact, the shape of the arch represents the bending moment caused by the abutment thrust.

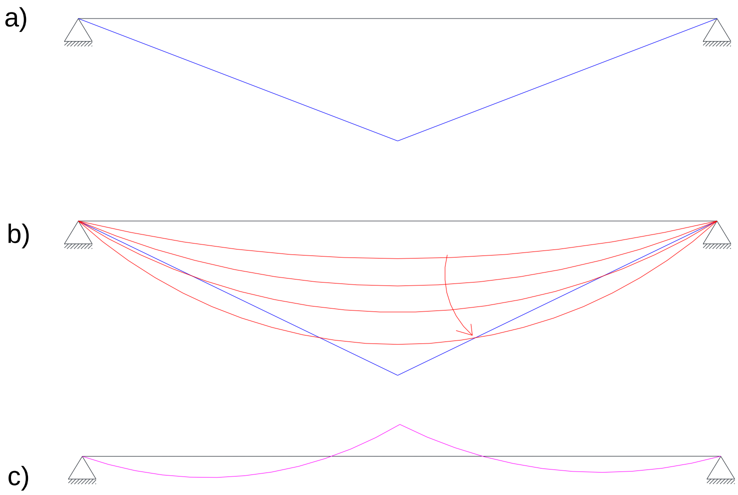


Fig. 19: Bending moment in the arch rib, a)caused by one concentrated force, b)caused by the horizontal component of the abutment thrust c)total bending moment.

Similarly to girder bridges, Level 0 assessment foresees the calculation, for each section of the arch rib, the envelope of the bending moment (Fig. 21c) caused by the design load (considering the worst position of the load (see for example Fig. 20)).

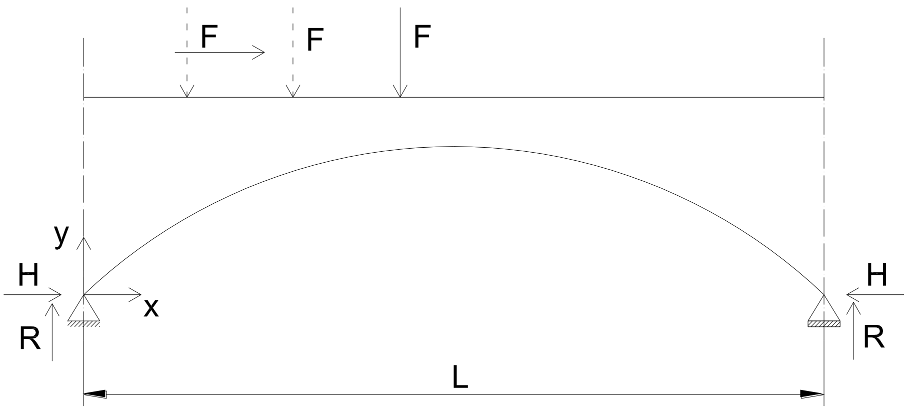


Fig. 20: Schematization of the arch bridge with a moving load

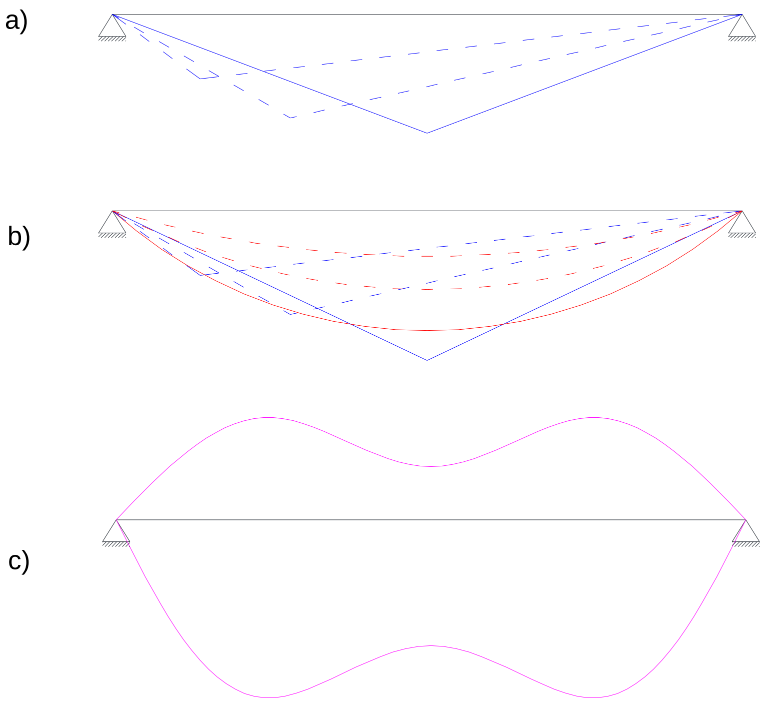


Fig. 21: Bending moment in the arch rib, a)caused by one moving concentrated force, b)caused by the horizontal component of the abutment thrust c)envelope of the total bending moment.

The same procedure is applied to the APT load. If in all sections of the deck the bending moment caused by APT load is equal to or lower than that caused by the design load, the verification is positive.
The approach discussed above requires the knowledge of the shape of the arch. In the case of the APT bridge stock these data are not known and then an approximation of the arch shape must be done to apply Level 0 assessment. In this assessment a parabolic shape of the arch was chosen.
For Level 2 assessment this is not necessary because the exact shape can be determined performing a geometric survey of the arch.
It is important to point out that for arch bridges, Level 1 assessment makes no sense because by maintaining the same calculation model as per design the result obtained would involve an increase either in the abutment thrust or in the eccentricity and therefore the same results as the Level 0 assessment would be obtained. Moreover, applying Level 3 assessment it is not possible to obtain a decrease in the lack of capacity shown after Level 0 or Level 2 assessments. The reason is that the mechanical properties of materials are not involved in the resistance mechanisms of arch bridges. In fact, the capacity is independent of the material strength but depends on the equilibrium of the forces acting upon it.

3.5 Lack of capacity of substandard bridges

To classify substandard bridges, we need to quantify the extent of their deficiency. A useful index here is the *critical live load multiplier* η, defined as the ratio between the stresses caused by the APT load and those due to the design load (12).

$$\eta = \frac{S_{APT}}{S_0} \tag{12}$$

where S_0 is the effect of the live load and S_{APT} is the effect of the APT load.
For girder bridges there will be a critical load multiplier for bending moment and shear stress, and these indices are calculated for each section of the span. The coefficient depends on the location of the section where the ratio is calculated. Therefore, the number of *critical live load multipliers* is equal to the number of sections where the comparison is performed. However, the most significant values are the maximum critical live load multiplier ($max\eta$) and the critical live load multiplier calculated where the stress is maximum (η_{max}). The latter is defined as the ratio between the maximum stress caused by the APT load and that due to the design load, and is independent of the location where the maximum stresses

are. η_{max} gives an in depth understanding of the deficiency of the bridge even if the location of the sections where the comparison is done is different. In fact, in most cases, these sections are close to each other and the capacity is piecewise constant. This means that even if the locations of the maximum stresses are not the same, we can assume this to be the case in order to estimate the response of the bridge. We can therefore calculate the critical live load multipliers for bending moment and shear stress. For each bridge the following indexes were calculated:

- maximum critical live load multiplier for shear stress ($max\eta_t$);
- maximum critical live load multiplier for bending moment ($max\eta_m$);
- critical live load multiplier for maximum shear stress ($\eta_{t,max}$);
- critical live load multiplier for maximum bending moment ($\eta_{m,max}$).

As discussed in Section 3.4.1, the parameters that control the assessment are the span length and the date of construction of the bridge. Thus, the safety factors, the dynamic coefficient and the static scheme of the bridge are irrelevant in the comparison. This is to say that it is possible to express the critical live load multipliers as a function of these parameters. In fact, Fig. 22 illustrates the critical live load multipliers as a function of the design code adopted by the designer, the span length, and the APT load (in the case of free travel). In Fig. 22 only two design codes are included (D.M. 1990 and Circolare 1962).

a)

SPAN (m)		DM_04_05_1990											
		56 t		5x13 t		6x13 t		7x13 t		8x13 t		9x13 t	
		Max η	ηmax	Max η	ηmax	Max η	ηmax	Max η	ηmax	Max η	ηmax	Max η	ηmax
1	V	0.533	0.533	0.650	0.650	0.650	0.650	0.650	0.650	0.650	0.650	0.650	0.650
	M	0.533	0.533	0.650	0.650	0.650	0.650	0.650	0.650	0.650	0.650	0.650	0.650
2	V	0.604	0.576	0.737	0.702	0.737	0.702	0.737	0.702	0.737	0.702	0.737	0.702
	M	0.604	0.533	0.737	0.650	0.737	0.650	0.737	0.650	0.737	0.650	0.737	0.650
3	V	0.604	0.586	0.737	0.737	0.737	0.737	0.737	0.737	0.737	0.737	0.737	0.737
	M	0.654	0.582	0.823	0.732	0.823	0.732	0.823	0.732	0.823	0.732	0.823	0.732
4	V	0.586	0.558	0.737	0.711	0.737	0.711	0.737	0.711	0.737	0.711	0.737	0.711
	M	0.595	0.585	0.749	0.737	0.749	0.737	0.749	0.737	0.749	0.737	0.749	0.737
5	V	0.586	0.546	0.755	0.755	0.755	0.755	0.755	0.755	0.755	0.755	0.755	0.755
	M	0.572	0.563	0.803	0.718	0.803	0.718	0.803	0.718	0.803	0.718	0.803	0.718
6	V	0.586	0.540	0.819	0.819	0.819	0.819	0.819	0.819	0.819	0.819	0.819	0.819
	M	0.558	0.551	0.832	0.780	0.832	0.780	0.832	0.780	0.832	0.780	0.832	0.780
7	V	0.568	0.535	0.867	0.867	0.886	0.886	0.886	0.886	0.886	0.886	0.886	0.886
	M	0.549	0.544	0.883	0.841	0.883	0.841	0.883	0.841	0.883	0.841	0.883	0.841
8	V	0.558	0.533	0.900	0.900	0.950	0.950	0.957	0.957	0.957	0.957	0.957	0.957
	M	0.543	0.540	0.917	0.881	0.945	0.900	0.948	0.900	0.948	0.900	0.948	0.900
9	V	0.551	0.550	0.924	0.924	0.997	0.997	1.031	1.031	1.031	1.031	1.031	1.031
	M	0.549	0.536	0.944	0.910	0.993	0.956	1.025	0.984	1.025	0.984	1.025	0.984
10	V	0.573	0.573	0.940	0.940	1.029	1.029	1.085	1.085	1.108	1.108	1.108	1.108
	M	0.570	0.534	0.961	0.932	1.026	0.998	1.083	1.051	1.101	1.051	1.101	1.051
11	V	0.594	0.594	0.947	0.947	1.049	1.049	1.121	1.121	1.164	1.164	1.178	1.178
	M	0.592	0.532	0.974	0.949	1.055	1.031	1.133	1.103	1.164	1.124	1.171	1.132
12	V	0.607	0.607	0.951	0.951	1.060	1.060	1.145	1.145	1.203	1.203	1.236	1.236
	M	0.606	0.530	0.986	0.962	1.080	1.058	1.172	1.144	1.222	1.185	1.232	1.213
13	V	0.620	0.620	0.953	0.953	1.065	1.065	1.159	1.159	1.230	1.230	1.277	1.277
	M	0.618	0.534	0.994	0.973	1.100	1.079	1.200	1.178	1.268	1.234	1.298	1.280
14	V	0.633	0.633	0.956	0.956	1.068	1.068	1.166	1.166	1.247	1.247	1.306	1.306
	M	0.632	0.546	0.996	0.982	1.115	1.097	1.219	1.206	1.301	1.275	1.351	1.336
15	V	0.642	0.642	0.958	0.958	1.071	1.071	1.171	1.171	1.257	1.257	1.326	1.326
	M	0.641	0.566	0.997	0.990	1.123	1.113	1.231	1.230	1.324	1.310	1.389	1.383
16	V	0.647	0.647	0.959	0.959	1.073	1.073	1.174	1.174	1.262	1.262	1.338	1.338
	M	0.646	0.582	0.997	0.995	1.127	1.124	1.248	1.248	1.341	1.338	1.421	1.421
17	V	0.649	0.649	0.961	0.961	1.074	1.074	1.176	1.176	1.266	1.266	1.345	1.345
	M	0.649	0.594	0.997	0.995	1.132	1.129	1.259	1.259	1.360	1.357	1.448	1.448
18	V	0.650	0.650	0.963	0.963	1.075	1.075	1.177	1.177	1.269	1.269	1.349	1.349
	M	0.654	0.603	0.997	0.994	1.134	1.131	1.265	1.265	1.372	1.369	1.468	1.468
19	V	0.650	0.649	0.964	0.964	1.076	1.076	1.178	1.178	1.270	1.270	1.353	1.353
	M	0.660	0.614	0.997	0.993	1.133	1.130	1.266	1.266	1.378	1.375	1.480	1.480
20	V	0.650	0.648	0.965	0.965	1.076	1.076	1.178	1.178	1.271	1.271	1.355	1.355
	M	0.663	0.623	0.997	0.992	1.132	1.128	1.264	1.264	1.380	1.378	1.487	1.487
21	V	0.650	0.648	0.967	0.967	1.077	1.077	1.178	1.178	1.271	1.271	1.355	1.355
	M	0.664	0.631	0.997	0.991	1.130	1.126	1.261	1.261	1.379	1.376	1.489	1.489
22	V	0.650	0.649	0.968	0.968	1.077	1.077	1.178	1.178	1.271	1.271	1.356	1.356
	M	0.664	0.638	0.997	0.990	1.128	1.124	1.258	1.258	1.376	1.373	1.488	1.488
23	V	0.651	0.651	0.969	0.969	1.077	1.077	1.178	1.177	1.271	1.270	1.356	1.355
	M	0.662	0.645	0.997	0.990	1.126	1.122	1.254	1.254	1.372	1.369	1.484	1.484
24	V	0.653	0.653	0.970	0.970	1.077	1.076	1.178	1.176	1.271	1.268	1.356	1.354
	M	0.661	0.651	0.997	0.989	1.124	1.120	1.251	1.251	1.368	1.365	1.479	1.479
25	V	0.655	0.655	0.971	0.971	1.077	1.076	1.178	1.175	1.271	1.267	1.356	1.352
	M	0.661	0.657	0.997	0.989	1.122	1.118	1.247	1.247	1.363	1.361	1.474	1.474

b)

SPAN (m)		CIRC_1962											
		56 t		5x13 t		6x13 t		7x13 t		8x13 t		9x13 t	
		Max η	ηmax	Max η	ηmax	Max η	ηmax	Max η	ηmax	Max η	ηmax	Max η	ηmax
1	V	0.561	0.561	0.684	0.684	0.684	0.684	0.684	0.684	0.684	0.684	0.684	0.684
	M	0.561	0.561	0.684	0.684	0.684	0.684	0.684	0.684	0.684	0.684	0.684	0.684
2	V	0.561	0.552	0.684	0.672	0.684	0.672	0.684	0.672	0.684	0.672	0.684	0.672
	M	0.561	0.561	0.684	0.684	0.684	0.684	0.684	0.684	0.684	0.684	0.684	0.684
3	V	0.598	0.598	0.752	0.752	0.752	0.752	0.752	0.752	0.752	0.752	0.752	0.752
	M	0.586	0.545	0.738	0.686	0.738	0.686	0.738	0.686	0.738	0.686	0.738	0.686
4	V	0.674	0.674	0.858	0.858	0.858	0.858	0.858	0.858	0.858	0.858	0.858	0.858
	M	0.673	0.656	0.847	0.825	0.847	0.825	0.847	0.825	0.847	0.825	0.847	0.825
5	V	0.716	0.716	0.990	0.990	0.990	0.990	0.990	0.990	0.990	0.990	0.990	0.990
	M	0.721	0.710	0.980	0.906	0.980	0.906	0.980	0.906	0.980	0.906	0.980	0.906
6	V	0.725	0.696	1.055	1.055	1.055	1.055	1.055	1.055	1.055	1.055	1.055	1.055
	M	0.750	0.742	1.061	1.051	1.061	1.051	1.061	1.051	1.061	1.051	1.061	1.051
7	V	0.725	0.668	1.081	1.081	1.106	1.106	1.106	1.106	1.106	1.106	1.106	1.106
	M	0.769	0.763	1.187	1.179	1.187	1.179	1.187	1.179	1.187	1.179	1.187	1.179
8	V	0.725	0.650	1.099	1.099	1.160	1.160	1.168	1.168	1.168	1.168	1.168	1.168
	M	0.782	0.778	1.277	1.270	1.298	1.297	1.298	1.297	1.298	1.297	1.298	1.297
9	V	0.725	0.658	1.108	1.105	1.191	1.191	1.232	1.232	1.232	1.232	1.232	1.232
	M	0.762	0.743	1.292	1.261	1.345	1.325	1.398	1.364	1.398	1.364	1.398	1.364
10	V	0.725	0.668	1.108	1.097	1.200	1.200	1.266	1.266	1.292	1.292	1.292	1.292
	M	0.731	0.717	1.275	1.252	1.356	1.341	1.438	1.412	1.438	1.412	1.438	1.412
11	V	0.715	0.675	1.108	1.077	1.201	1.193	1.275	1.275	1.323	1.323	1.339	1.339
	M	0.708	0.698	1.262	1.244	1.365	1.352	1.467	1.446	1.486	1.475	1.506	1.484
12	V	0.696	0.679	1.108	1.063	1.201	1.185	1.280	1.280	1.345	1.345	1.382	1.382
	M	0.691	0.683	1.253	1.238	1.371	1.361	1.490	1.472	1.535	1.525	1.580	1.562
13	V	0.687	0.687	1.108	1.056	1.201	1.180	1.284	1.284	1.363	1.363	1.415	1.415
	M	0.695	0.677	1.245	1.233	1.376	1.368	1.507	1.493	1.573	1.564	1.638	1.623
14	V	0.699	0.699	1.108	1.055	1.201	1.180	1.288	1.288	1.377	1.377	1.442	1.442
	M	0.698	0.681	1.239	1.225	1.380	1.369	1.521	1.505	1.603	1.591	1.685	1.667
15	V	0.704	0.704	1.107	1.051	1.201	1.175	1.288	1.284	1.379	1.379	1.454	1.454
	M	0.703	0.692	1.234	1.210	1.384	1.360	1.533	1.503	1.629	1.602	1.724	1.691
16	V	0.708	0.708	1.100	1.050	1.201	1.174	1.288	1.285	1.382	1.382	1.464	1.464
	M	0.708	0.699	1.220	1.195	1.375	1.350	1.530	1.500	1.636	1.607	1.742	1.707
17	V	0.711	0.711	1.093	1.053	1.198	1.177	1.289	1.289	1.387	1.387	1.474	1.474
	M	0.715	0.701	1.206	1.175	1.365	1.333	1.525	1.487	1.640	1.602	1.754	1.710
18	V	0.714	0.714	1.083	1.059	1.190	1.183	1.295	1.295	1.395	1.395	1.484	1.484
	M	0.721	0.703	1.195	1.159	1.358	1.319	1.521	1.475	1.643	1.596	1.765	1.712
19	V	0.717	0.717	1.067	1.066	1.190	1.190	1.303	1.303	1.405	1.405	1.496	1.496
	M	0.724	0.711	1.180	1.149	1.342	1.308	1.505	1.465	1.632	1.592	1.759	1.713
20	V	0.722	0.722	1.075	1.075	1.199	1.199	1.313	1.313	1.416	1.416	1.509	1.509
	M	0.726	0.718	1.170	1.143	1.331	1.300	1.492	1.457	1.623	1.588	1.755	1.714
21	V	0.728	0.728	1.086	1.086	1.209	1.209	1.323	1.323	1.428	1.428	1.523	1.523
	M	0.734	0.726	1.165	1.141	1.323	1.296	1.482	1.452	1.616	1.584	1.751	1.714
22	V	0.737	0.737	1.098	1.098	1.221	1.221	1.335	1.335	1.441	1.441	1.537	1.537
	M	0.745	0.736	1.163	1.141	1.320	1.296	1.477	1.450	1.612	1.583	1.747	1.715
23	V	0.746	0.746	1.110	1.110	1.233	1.233	1.348	1.348	1.455	1.455	1.552	1.552
	M	0.753	0.745	1.158	1.144	1.313	1.297	1.468	1.449	1.602	1.582	1.736	1.715
24	V	0.756	0.756	1.123	1.123	1.247	1.247	1.362	1.362	1.469	1.469	1.568	1.568
	M	0.760	0.753	1.156	1.144	1.309	1.295	1.462	1.447	1.596	1.579	1.729	1.711
25	V	0.768	0.768	1.137	1.137	1.260	1.260	1.376	1.376	1.484	1.484	1.584	1.584
	M	0.768	0.762	1.157	1.147	1.308	1.297	1.459	1.446	1.592	1.578	1.725	1.710

Fig. 22: Critical live load multipliers as a function of the design code [a)D.M. 1990 and b)Circolare 1962] adopted by the designer, the span length, and the APT load.

In Table 5 the association of the cell colors used in Fig. 22 with critical live load multiplier (η) is shown.

Table 5: Critical live load multiplier and assigned colors

Critical live load multiplier	Color
$\eta \leq 1$	
$1 < \eta \leq 1.1$	
$1.1 < \eta \leq 1.3$	
$1.3 < \eta \leq 1.6$	
$\eta > 1.6$	

Knowing the critical live load multipliers for each bridge, it is possible to define a priority ranking for future assessments. If η is slightly greater than 1, there is a high probability that by applying Level 1 assessment the verifications become positive. As η increases, this probability is smaller and smaller.

It is worth emphasizing that a critical live load multiplier η of less than 1.0 can have different impacts on the overall safety of the bridge, depending on the magnitude of the dead load. For example, a live load multiplier of 0.90 could be critical for short span steel girder bridges, where the live load is dominant, but is likely to be much less critical for a long span reinforced concrete bridge, where the dead load dominates.

An index that better reflects bridge understrength with respect to the overweight load is the *lack of capacity factor* α, defined as the percentage of additional capacity ΔR needed to safely carry the overweight load. For girder bridges this coefficient indicates how much the most critical bending moment or shear stress must increase for positive verification of the bridge. There is a direct relationship between indices α and η. If R is the capacity of the bridge at a specific limit state and S_{APT} is the overweight load demand, the lack in capacity α of the bridge can be expressed as follows (13):

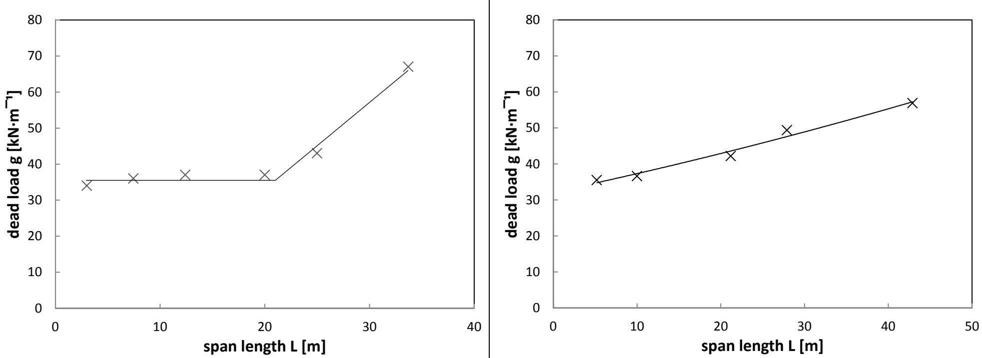
$$\alpha = \frac{S_{APT} - R}{R} = \frac{\Delta S}{R} = \frac{G/S_0 + \eta - (G/S_0 + 1)}{G/S_0 + 1} = \frac{\eta - 1}{G/S_0 + 1} \tag{13}$$

where G is the effect of the dead load and S_0 is the effect of the live load. Equation (13) shows that computing index α requires knowledge of the G/S_0 ratio for the bridge, and this in turn depends on the bridge characteristics as included in the design documentation, normally not known for a Level 0 assessment. To estimate index α at Level 0, a practical solution is to provide approximate expressions for

the dead loads of various bridge structures and span lengths. To do this, it is necessary to evaluate, for each main deck type, the weight of the bridge in a number of case studies (Table 6) and then to extrapolate the relative interpolating curve (Fig. 23a). This curve provides the weight of a lane width of 3 meters as a function of the bridge span.
The graphs in Fig. 23 show the dead load of a 3 m wide lane calculated for a sample of bridges of differing construction technologies, with proposed fitting curves that can be used to estimate the dead load once the bridge span is known.

Table 6: Weight of reinforced concrete bridges

Reinforced concrete girder bridge	
Span length [m]	P/L (lane of 3m) [kN/m]
3.0	34
7.4	36
12.4	37
20.0	37
25.0	43
33.7	67



a) reinforced concrete girder *b) precast reinforced concrete girder*

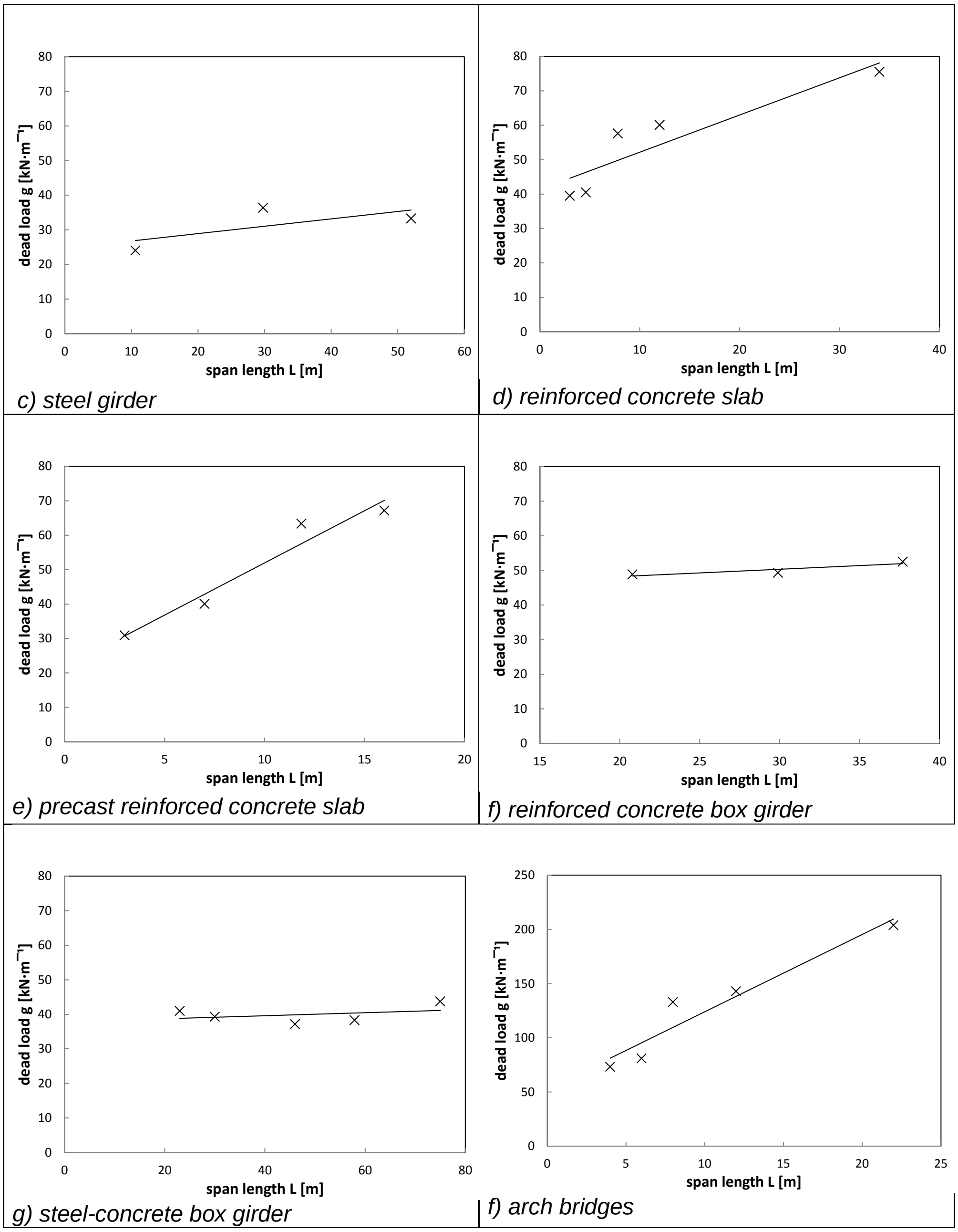


Fig. 23: Bridge span to dead load relationship for various bridge technologies: experimental samples and fitting curves

3.6 Risk interpretation of parameter α

This section provides the risk interpretation of parameter α in terms of the probability of failure of a substandard bridge caused by the APT load.

To determine the reliability index and the probability of failure caused by the APT load on a substandard bridge, it is necessary to evaluate the reliability of the design code. In other words, the aim is to determine the reliability index and the probability of failure of the bridge foreseen by the design code. The fundamental hypothesis is that the designer has strictly followed the design code. It is assumed that the probability of failure of the bridge that is implicit in the design code (P_{F0}) is the following (14):

$$P_{F0} = 10^{-5} \; P_{F0} = 10^{-5} \; P_{F0} = 10^{-5} \; P_{F0} = 10^{-5} \tag{14}$$

Consequently, the a priori reliability index (β_0) is the following (15):

$$\beta_0 = -\Phi^{-1}\left(P_{F0}\right) = -\Phi^{-1}\left(10^{-5}\right) = 4.26 \tag{15}$$

where Φ is the normal cumulative distribution function with mean value μ=0 and standard deviation σ=1.

In general the a priori reliability index can be expressed as follows (16):

$$\beta_0 = \frac{R_{50\%,0} - S_{50\%,0}}{\sqrt{\sigma_{R0}^2 + \sigma_{S0}^2}} \tag{16}$$

where $R_{50\%}$ and $S_{50\%}$ are the mean values of the normal distributions of capacity and demand respectively, σ_R and σ_S are the standard deviation of the normal distributions of capacity and demand respectively, and the subscript 0 indicates the design code.

The reliability index concerning the movement of the overweight load (β_{OL}) can be expressed as follows (17):

$$\beta_{OL} = \frac{R_{50\%,0} - S_{50\%,OL}}{\sqrt{\sigma_{R0}^2 + \sigma_{S0}^2}} \tag{17}$$

where $S_{50\%,OL}$ is the mean value of the normal distributions of the demand caused by the APT load.

The ratio between the two reliability indexes can be expressed as follows (18):

$$\frac{\beta_{OL}}{\beta_0} = \frac{\dfrac{R_{50\%,0} - S_{50\%,OL}}{\sqrt{\sigma_{R0}^2 + \sigma_{S0}^2}}}{\dfrac{R_{50\%,0} - S_{50\%,0}}{\sqrt{\sigma_{R0}^2 + \sigma_{S0}^2}}} = \frac{R_{50\%,0} - S_{50\%,OL}}{R_{50\%,0} - S_{50\%,0}} \tag{18}$$

If it is assumed that the ratio between the mean and the design value of the demand caused by the overweight load ($S_{d,OL}$) is equal to the ratio between the mean and the design value of the initial demand ($S_{d,0}$), we can write (19):

$$\frac{S_{50\%,OL}}{S_{d,OL}} = \frac{S_{50\%,0}}{S_{d,0}} \tag{19}$$

$S_{50\%,OL}$ is expressed as follows (20):

$$S_{50\%,OL} = S_{50\%,0}\frac{S_{50\%,OL}}{S_{d,0}} = S_{50\%,0}\frac{S_{50\%,OL}}{R_{d,0}} = S_{50\%,0}(1+\alpha) \tag{20}$$

where it is assumed that the design demand is equal to the design capacity $S_{d0}=R_{d0}$. Equation (18) can therefore be expressed as follows (21):

$$\frac{\beta_{OL}}{\beta_0} = \frac{\dfrac{R_{50\%,0}}{S_{50\%,0}} - \dfrac{S_{50\%,0}(1+\alpha)}{S_{50\%,0}}}{\dfrac{R_{50\%,0}}{S_{50\%,0}} - \dfrac{S_{50\%,0}}{S_{50\%,0}}} = \frac{\gamma_{m0} - (1+\alpha)}{\gamma_{m0} - 1} \tag{21}$$

where γ_{m0} *is* the mean safety factor (22) (23).

$$\gamma_{m0} = \frac{R_{50\%,0}}{S_{50\%,0}} = \frac{R_{50\%,0}}{R_{5\%,0}} \cdot \frac{R_{5\%,0}}{R_{d,0}} \cdot \frac{R_{d,0}}{S_{d,0}} \cdot \frac{S_{d,0}}{S_{95\%,0}} \cdot \frac{S_{95\%,0}}{S_{50\%,0}} \tag{22}$$

$$\gamma_{m0} = \gamma_R \cdot \gamma_S \cdot \frac{1 + k \cdot CoV_S}{1 - k \cdot CoV_R} \tag{23}$$

where γ_S and γ_R are the partial factor for the demand and capacity respectively, and the relationships between the characteristic and mean value of the capacity and demand are the following (24) (25):

$$R_{5\%} = R_{50\%} - k \cdot \sigma_R = R_{50\%}\left(1 - k \cdot CoV_R\right) \tag{24}$$

$$S_{95\%} = S_{50\%} + k \cdot \sigma_S = S_{50\%}\left(1 + k \cdot CoV_S\right) \tag{25}$$

where $R_{5\%}$ and $S_{95\%}$ are the characteristic values of the normal distributions of capacity and demand respectively, CoV_R and CoV_S are the coefficient of variation of the normal distributions of capacity and demand respectively, and k is the characteristic fractile factor equal to k=1.645.

Assuming the following values for the partial factor and for the coefficient of variation of the demand and capacity, the relationship between the failure probability and the lack of capacity is obtained (Fig. 24).

- CoV_R = 7.5%
- CoV_S = 15%
- γ_R = 1.2
- γ_S = 1.4

In Table 7 the association of the failure probability (P_f) with the lack of capacity (α) of the bridge is shown.

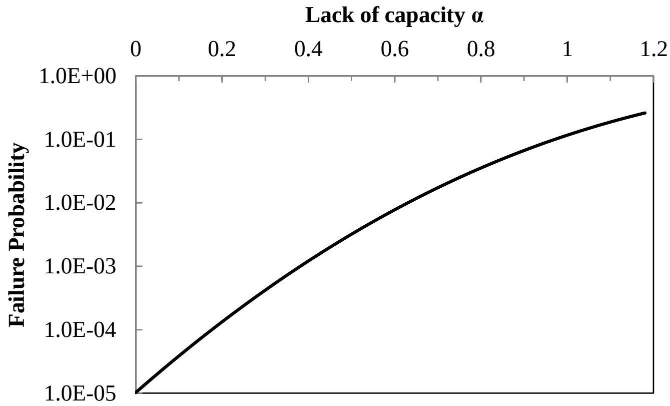


Fig. 24: Relationship between failure probability and α

Table 7: Failure probability and lack of capacity

Failure probabilty	Lack of capacity
$P_f = 10^{-5}$	$\alpha=0$
$10^{-5} < P_f < 4\cdot10^{-5}$	$0 < \alpha < 0,1$
$4\cdot10^{-5} < P_f < 2\cdot10^{-4}$	$0,1 < \alpha < 0,2$
$2\cdot10^{-4} < P_f < 5\cdot10^{-4}$	$0,2 < \alpha < 0,3$
$P_f > 5\cdot10^{-4}$	$\alpha > 0,3$

3.7 Conclusions

In order to limit costs, the assessment of the capacity of a bridge stock needs a simplified and conservative approach first, and then, if a higher load rating is required, a refining of the analysis. This Chapter has defined the assessment procedure to manage the problem of the overweight traffic, with various simplified levels of refinement, from Level 0 to Level 3. It has been shown that in the case of free road traffic, the result of the assessment depends only on the maximum span and on the year of bridge design (i.e. on the design loads of the design code). Furthermore it is completely independent of the mechanical conditions of the bridge. Moreover we obtained that in the case of a road closed to free traffic and bridge transit in the center of the roadway, transit of the overweight load on the bridge also depends on the transverse load distribution. The procedure has been split for girder and arch bridges because of different resistance mechanisms of the two typologies.

To quantify the extent of the deficiency of the substandard bridges, a classification has been made by defining the lack of capacity factor α. This index reflects bridge understrength with respect to the overweight load and has been defined as the percentage of the additional capacity needed to safely carry the overweight load. Finally, the risk interpretation of parameter α in terms of the probability of failure of a substandard bridge caused by the APT load has been provided.

4 APPLICATION TO CASE STUDIES

4.1 Introduction

In Chapter 3, the general method to verify bridges under overweight loads was shown. The assessment procedure involves three levels of refinement to improve the verification of the bridge. In this Chapter I apply the methodology to two case studies representative of the two typologies of bridges (girder and arch bridges) in order to test the method. Although Level 3 assessment was not considered in the case studies due to the costs involved in obtaining the updated material properties based on in-situ tests, a simulation of this assessment level is performed in order to show the methodology when in-situ tests are available.

4.2 Case studies

The two case studies analyzed are:

- *Fiume Adige* bridge, SP90 km 1.159;
- *Rio Cavallo* bridge, SS12 km 362.164.

As discussed above these two case studies are representative of the two typologies in which bridges can be classified. *Fiume Adige* is a girder bridge while *Rio Cavallo* is an arch bridge. It is worth pointing out that the methodology proposed is valid for all the typologies of bridges, which are clearly more than two (suspension bridges, cable-stayed bridges etc.). All types of bridges, except arch bridges, can be evaluated as they were girder bridges. In fact, the method foresees the comparison among demands caused by design loads and reference loads (APT loads). Therefore, as discussed in Section 3.4.1, when the original design load produces static stresses higher than the new overweight vehicle in a simply supported span, the design stresses will be higher with a statically indeterminate boundary condition. This is to say that the typology of the bridge is irrelevant to the comparison (except arch bridges), so a simply supported condition, choosing an appropriately span length, can always be assumed. For instance in the case of a cable-stayed bridge the length among two cables must be chosen for the calculation rather than the total span length, because the cables are actually supports.

4.2.1 Fiume Adige

The bridge is located in the Municipality of Rovereto, close to the town of Trento, Italy. The bridge is shown in Fig. 25. It is a four span prestressed concrete bridge, built in 1980, with the maximum span L=33.12m. The deck has an overall width of 9.50m and carries a 7.50m roadway and two 1.25m sidewalks. The deck structure of the 3 main spans is made up of 10 precast 1.44m double T girder elements per span, with a 0.18m cast-in-place concrete deck, while the shortest span is made up of 42 prefabricated elements pulled together with a cast-in-place concrete deck.

The main dimensions of the structure are reported in the following:

- bridge length: 106 m
- first span
    - length: 6.5 m:
    - girder spacing and number of girders: 0.21 m / 42 elements
    - total thickness of precast reinforced concrete slab: 0.45m
- other spans
    - length: 32.13m-33.12m
    - girder spacing and number of girders: 0.92 m / 10 elements
    - deck thickness: 0.18m
- deck width: 9.5 m
- carriageway width: 7.0 m

The design codes used by the bridge designer are the following:

- D.M. 26/03/1980 “Norme tecniche per l’esecuzione delle opere in cemento armato normale, precompresso e per le strutture metalliche.”
- D.M. 02/08/1980 “Criteri generali e prescrizioni tecniche per la progettazione, esecuzione e collaudo di ponti stradali.”

- CNR-UNI 10018/72 “Istruzioni per il calcolo e l’impiego di apparecchi di appoggio in gomma nelle costruzioni.”

a)

b)

Fig. 25: a) and b): “Adige Bridge”, Municipality of Rovereto (TN, Italy)

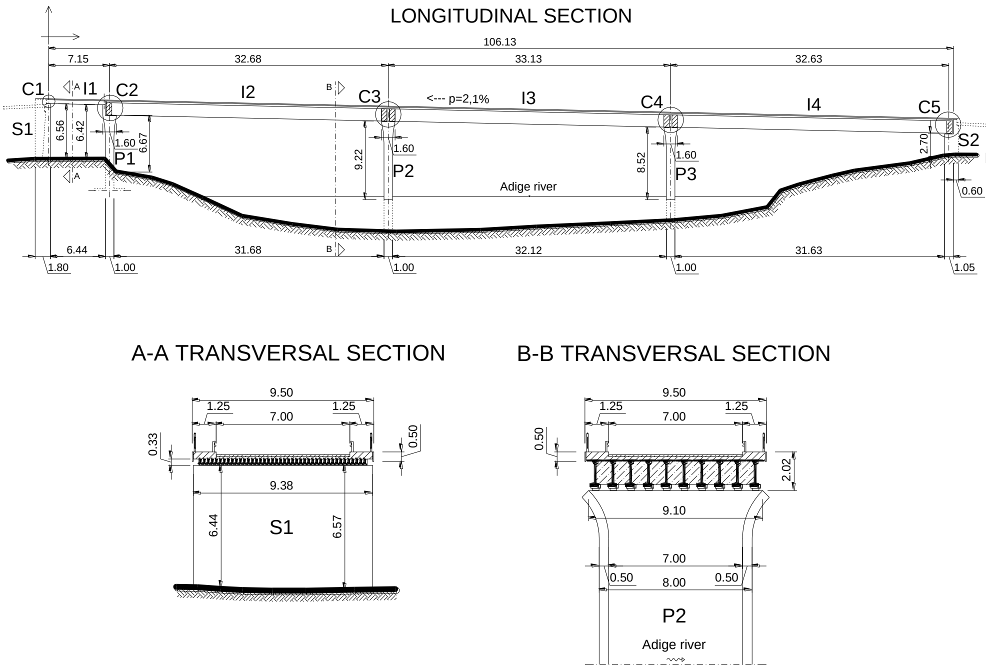


Fig. 26: Geometry of the Fiume Adige bridge

The lack of capacity (α) depends on the overweight load. Table 8 shows the lack of capacity for each APT load using Level 0 assessment. These values are calculated by a Matlab program (see Chapter 6) with the methodology described in Chapter 3.

Table 8: Results of Level 0 assessment

APT load	Level 0 assessment (α)
56 tons	0.00
5x13 tons	0.14
6x12 tons	0.16
6x13 tons	0.21
7x13 tons	0.27
8x13 tons	0.33
9x13 tons	0.39

The design loads used by the bridge designer are the following:

- permanent load of the deck (sum of structural and non-structural loads):
  - G = 14.457t/m.
- traffic loads:

- lane number 1: uniformly distributed loads q_{1a} = 4.509 t/m;
- lane number 2: uniformly distributed loads q_{1b} = 2.166 t/m;
- remaining area: uniformly distributed loads q_f=0.4t/m^2.

- the dynamic response coefficient of span 2, 3 and 4 is the following:
  - Φ= 1.211

The design loads and the dynamic response coefficient reported above refer only to spans 2, 3 and 4. This is because, for the sake of brevity, in the following I report the details of the calculations only for these spans.

Level 1 assessment

As mentioned in Section 3.3, in Level 1 assessment the evaluator is required to examine the design documents to verify whether the critical structural members are oversized. This level of assessment is based only on the analysis of the existing documents, without revising the assumed material properties or calculation model. The evaluator repeats the structural calculations, replacing the design traffic lane with the APT load. He/she calculates the effects caused by the new loads and verifies that they are less than the capacity of the bridge. If one verification is negative, the evaluator calculates the lack of capacity of the bridge (α), as reported in Section 3.5. The verifications of the following structural elements are performed:

- decks;
- columns;
- abutments;
- supports made of neoprene.

For each element a number of verifications are performed, such as the verification of the bending moment and shear stress. For the sake of brevity, I report the details of the calculations of the deck element only.

Free Travel

The design documentation includes the Massonnet-Guyon-Bares transverse load distribution. In the following it is reported the calculation of the most stressed beam that has an eccentricity of 1.410m from the center of the deck. In Fig. 27 the transverse position of loads and resulting Massonnet-Guyon-Bares transverse load distribution are represented, while in Table 9 their values are reported.

Table 9: Transverse redistribution of loads for free travel

Loads	Eccentricity (m)	K (Massonnet)	Q (t/m)
q_{f1} (Right side remaining area)	4.000	1.254	0.4
q_{1a}	1.750	1.563	4.509
q_{1b}	-1.750	0.761	2.166
q_{f2} (Left side remaining area)	-4.000	0.096	0.4

The total load on the most stressed beam can be calculated as follows (26):

$$Q_{tot} = \frac{G + \left[\left(q_{f1} \cdot K_{f1} + q_{1a} \cdot K_{1a} + q_{1b} \cdot K_{1b} + q_{f2} \cdot K_{f2}\right) \cdot \emptyset\right]}{n_{girders}} \tag{26}$$

where $n_{girders}$ is the number of prefabricated double T girder elements, equal to $n_{girders}$=10.

The design value of the bending moment (M_e) is the following (27):

$$M_e = \frac{Q_{tot} \cdot l^2}{8} = \frac{2.564 \cdot 32.125^2}{8} = 330.85\ t \cdot m \tag{27}$$

where l is the span length.

The resistance moment reported in the design documentation is as follows (28):

$$M_r = 620.56\ t \cdot m \tag{28}$$

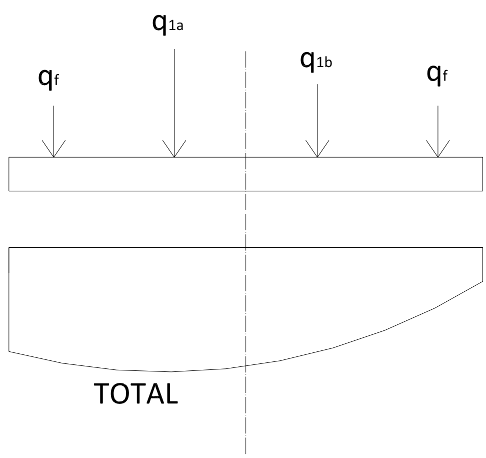


Fig. 27: Transverse position of loads and resulting Massonnet-Guyon-Bares transverse load distribution (free travel)

If the q_{1a} load is replaced with, for example the 6 x 13 t = 78 tons APT load (Fig. 27 and Fig. 28), the design bending moment on the most stressed beam is equal to (29):

$$M_{e\_6x13t} = 338.20 \ \ t \cdot m \tag{29}$$

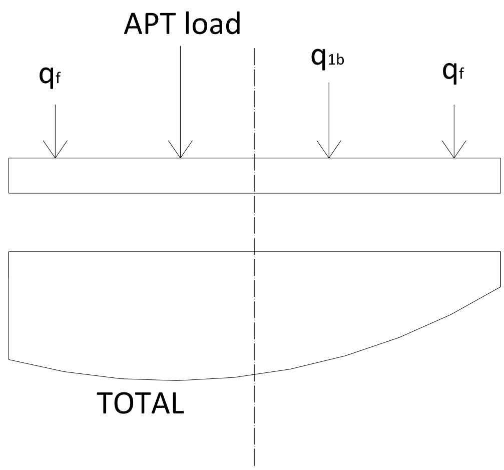


Fig. 28: Transverse position of loads and resulting Massonnet-Guyon-Bares transverse load distribution obtained replacing q_{1a} with APT load (free travel)

Table 10 shows the verifications of the bending moment on the most stressed beam for each APT load. The resistance moment reported was calculated by the bridge designer for the design documentation. The safety factor foreseen by the design code is equal to η_r =1.85, and then the verification is positive if the ratio between the resistance moment and design moment is greater than:

$$\eta_r = \frac{M_r}{M_e} \geq 1.85 \tag{30}$$

Table 10: Bending moment verification (free travel)

APT load	M_r (tm)	M_e (tm)	η_r	lack of capacity (α)
56t	620.56	294.49	2.107	0.00
5x13t	620.56	325.78	1.905	0.00
6x12t	620.56	330.19	1.879	0.00
6x13t	620.56	338.20	1.835	0.01
7x13t	620.56	350.62	1.770	0.05

8x13t	620.56	361.93	1.715	0.08
9x13t	620.56	373.21	1.663	0.11

The same procedure was followed to perform the shear verfication of the most stressed beam. The shear verification involves comparing the shear stress caused by the design load with the limit shear stress τ_{c1}=24 kg/cm^2(D.M. 26/03/1980). The shear stress is equal to the sum of two contributions: the first caused by the structural permanent loads considering only the prefabricated double T girder elements and the second caused by the non-structural permanent and traffic loads considering the whole cross-section (girder element + cast-in-place concrete deck). The first contribution is independent of the traffic loads and had been already calculated by the bridge designer. Therefore, only the second contribution had to be calculated replacing the design load with the APT load.
For the sake of brevity the results of the verification are reported without showing the calculations (Table 11).

Table 11: Shear verification (free travel)

APT load	total shear stress τ_e (Kg/cm²)	limit shear stress τ_{c1} (Kg/cm²)	lack of capacity (α)
56t	17.26	24	0
5x13t	17.91	24	0
6x12t	17.98	24	0
6x13t	18.12	24	0
7x13t	18.32	24	0
8x13t	18.51	24	0
9x13t	18.69	24	0

In addition to the ultimate verification here, I report also the serviceability verification which involves checking the stress at the top ($\sigma_{S,TOT}$) and bottom ($\sigma_{I,TOT}$) edge of the beam. The limits of compression (σ_c) and tension stress (σ_t) included in the design code are the following:

- σ_c < 0.38 Rck = 0.38·460 kg/cm² = 174.8 kg/cm²
- σ_t < 0.06 Rck = 0.06·460 kg/cm² = (-) 27.6 kg/cm²

Also in this case the total stress is obtained by the sum of two contributions: the first caused by the structural permanent loads considering only the girder elements and the second caused by the non-structural permanent and traffic loads considering the whole cross-section. Fig. 29 shows a part of the original design documentation where the red arrow indicates the effects of the traffic load that has to be replaced by the APT load. Table 12 shows the results of the verification.

4.4. AZIONI INTERNE E TENSIONI NEL CALCESTRUZZO AI LEMBI DELLA TRAVE PREFABBRICATA

	N (T)	M (TM)	A (CMQ)	WS (CMC)	WI (CMC)	SIGMA SUP. (KG/CMQ)	SIGMA INF. (KG/CMQ)
PRECOMPRESSIONE	345.73		3008.20			114.93	114.93
		-150.671		108844.4	-132362.1	-138.42	113.83
PESO TRAVE PREFABBRICATA		92.823		108844.4	-132362.1	85.28	-70.12
AL TAGLIO DELL'ARMATURA						61.78	158.63
PESO SOLETTA E TRAVERSI		55.148		108844.4	-132362.1	50.66	-41.66
CADUTE						3.57	-34.79
FINITURE		38.539		317417.2	-172885.5	12.14	-22.29
PONTE SCARICO						128.16	59.87
ACCIDENTALI		144.342		317417.2	-172885.5	45.47	-83.49
PONTE CARICO						173.64	-23.61
ACCIDENTALI PER TAGLIO MAX		92.131		317417.2	-172885.5	29.02	-53.29
PONTE CARICO PER TAGLIO MAX						157.19	6.56

Fig. 29: scan of a part of the original design documentation

Table 12: Serviceability verification (free travel)

APT load	$\sigma_{S,TOT}$ (kg/cm²)	lack of capacity (α)	$\sigma_{I,TOT}$ (kg/cm²)	lack of capacity (α)
56t	162.19	0.00	-2.60	0.00
5x13t	172.04	0.00	-20.69	0.00
6x12t	173.43	0.00	-23.25	0.00
6x13t	175.95	0.01	-27.88	0.01
7x13t	179.86	0.05	-35.06	0.04
8x13t	183.43	0.08	-41.61	0.07
9x13t	186.98	0.12	-48.13	0.11

The verifications are summarized in Table 13.

Table 13: Summary of Level 1 assessment (free travel)

APT load	Level 0 (α)	Bending moment Level 1 (α)	Shear stress Level 1 (α)	serviceability verification Level 1 (α)
56t	0.00	0.00	0.00	0.00
5x13t	0.14	0.00	0.00	0.00
6x12t	0.16	0.00	0.00	0.00
6x13t	0.21	0.01	0.00	0.01
7x13t	0.27	0.05	0.00	0.05
8x13t	0.33	0.08	0.00	0.08
9x13t	0.39	0.12	0.00	0.12

Bridge crossed in the center of the roadway

The verifications shown above have to be performed for traffic crossing the bridge in the center of the roadway as well. The differences with respect to the verification with free travel are the transverse position of the vehicle in the carriageway and the absence of the second traffic lane.

The following shows the calculation of the most stressed beam, which, in this case, is located in the center of the deck. Fig. 30 shows the transverse position of loads and resulting Massonnet-Guyon-Bares transverse load distribution while in Table 9 their values are reported.

Table 14: Transverse redistribution of loads for bridge crossed in the center of the roadway

Loads	Eccentricity (m)	K (Massonnet)	Q (t/m)
Right side remaining area	4.000	0.748	0.4
APT load	0.000	1.467	var
Left side remaining area	-4.000	0.378	0.4

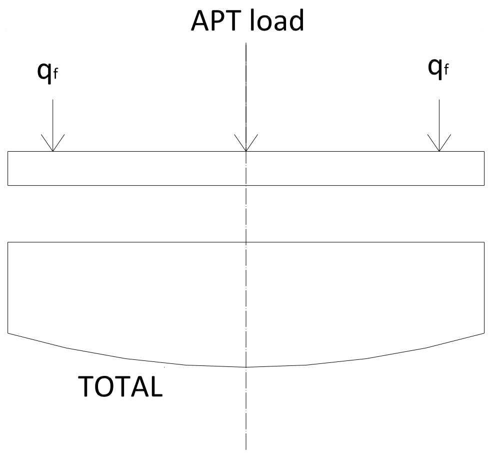


Fig. 30: Transverse position of loads and resulting Massonnet-Guyon-Bares transverse load distribution (bridge crossed in the center of the roadway)

For the sake of brevity, only the final results of the bending moment (Table 15) and the serviceability (Table 16) verification are reported.

Table 15: Bending moment verification (bridge crossed in the center of the roadway)

APT load	M_r (tm)	M_e (tm)	η_r	lack of capacity (α)
56t	620.56	262.81	2.36	0.00
5x13t	620.56	292.17	2.12	0.00
6x12t	620.56	296.32	2.09	0.00
6x13t	620.56	303.83	2.04	0.00
7x13t	620.56	315.49	1.97	0.00
8x13t	620.56	326.10	1.90	0.00
9x13t	620.56	336.69	1.85	0.00

Table 16: Serviceability verification (bridge crossed in the center of the roadway)

APT load	$\sigma_{S,TOT}$ (kg/cm²)	lack of capacity (α)	$\sigma_{I,TOT}$ (kg/cm²)	lack of capacity (α)
56t	152.20	0.00	15.72	0.00
5x13t	161.46	0.00	-1.26	0.00
6x12t	162.76	0.00	-3.66	0.00
6x13t	165.13	0.00	-8.01	0.00
7x13t	168.80	0.00	-14.75	0.00
8x13t	172.15	0.00	-20.89	0.00
9x13t	174.48	0.00	-27.01	0.00

As can be noticed in Table 15 and Table 16, the verifications are positive for each APT load. The shear verification is not reported as it was already verified for free travel.

Level 2 assessment

Level 2 assessments are needed when the verification at Level 1 is not satisfied. In this case only the bending moment verification is required at Level 2 as the shear verification was positive at Level 1.

As mentioned in Section 3.3, at Level 2 the evaluator reassesses the bridge capacity using more refined models than those used by the original designer. Often, a capacity increase is achieved by considering inelastic behavior and spatial stress redistribution. In this case the deck is made by 10 simple supported beams and the increase in the capacity is made possible by considering a different transverse redistribution of the loads. In fact, the bridge designer used an elastic transverse distribution of the loads and so it is possible to increase the capacity by considering a plastic transverse distribution (Fig. 31).

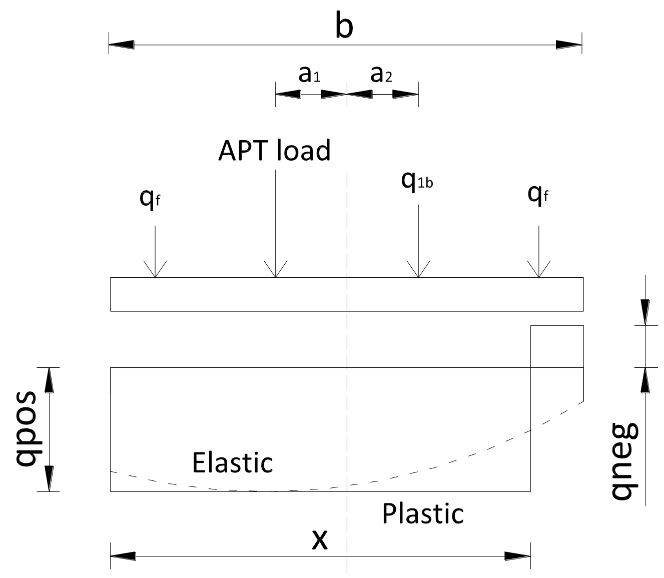


Fig. 31: Plastic and elastic transverse distribution.

The value of q_{neg} (Fig. 31) is equal to q_{neg} = 0 in favor of safety, the unknowns are x and $qpos$, which can be determined by imposing the equilibrium of the system. The maximum uniformly distributed load $q_{pos,max}$ on the deck is equal to (33):

$$q_{pos} \leq q_{pos,max} = \frac{\left[\frac{(M_r \cdot 8)}{L^2}/1.85\right]}{i} = 2.737\ t/m \tag{31}$$

where M_r is the resistance moment of the beam, 1.85 is the partial safety factor and i =0.95 is the spacing between two girders.

Imposing the equilibrium of the system (32) and (33):

$$q_{pos} \cdot x - q_{neg} \cdot (b - x) = \left(2q_f + q_{APT} + q_{1b}\right) \cdot \Phi + G \tag{32}$$

$$q_{pos} \cdot x \cdot \left(\frac{b - x}{2}\right) - q_{neg} \cdot (b - x) \cdot \left(\frac{x}{2}\right) = \left(q_{APT} \cdot a_1 - q_{1b} \cdot a_2\right) \cdot \Phi \tag{33}$$

where b = 9.50m is the width of the deck, ϕ = 1.211 is the dynamic response coefficient, q_f = 0.4 t/m, q_{neg}=0, q_{1b} = 2.166t/m, and G = 14.457 t/m is the sum of structural and non-structural permanent loads.

Here, in the equilibrium of the system equations the loads were imposed. Therefore, it is necessary to determine the APT uniformly distributed load (q_{APT}) that produces the same effects (in terms of maximum bending moment) as the real APT load (for example 6x13 tons APT load). If the 6x13 tons APT load is considered, the 6x13t APT uniformly distributed load is equal to $q_{APT,6x13t}$ = 4.813 t/m, and the following results are obtained:

- x=9.07 m
- $q_{pos,6x13t}$=2.589 t/m

where $q_{pos,6x13t}$ is the resultant value of q_{pos} considering the 6x13t APT uniformly distributed load.

The verification is positive (see equation (34)) and the new partial safety factor is obtained from equation (30):

$$\eta_{r,6x13t} = \frac{M_r}{M_{e,6x13t}} = \frac{M_r}{\left(q_{pos,6x13t} \cdot i \cdot \frac{l^2}{8}\right)} = \frac{620.563}{317.287} = 1.956 \tag{34}$$

Table 17 and Table 18 report the final results for each APT load and assessment level.

Table 17: Summary of the final results (free travel)

APT load	Level 0 (α)	Bending moment Level 1 (α)	Bending moment Level 2 (α)	Shear stress Level 1 (α)	Serviceability verification Level 1 (α)
56t	0.00	0.00	0.00	0.00	0.00
5x13t	0.14	0.00	0.00	0.00	0.00
6x12t	0.16	0.00	0.00	0.00	0.00
6x13t	0.21	0.01	0.00	0.00	0.01
7x13t	0.27	0.05	0.00	0.00	0.05
8x13t	0.33	0.08	0.03	0.00	0.08
9x13t	0.39	0.12	0.06	0.00	0.12

Table 18:Summary of the final results (bridge crossed in the center of the roadway)

APT load	Level 0 (α)	Bending moment Level 1 (α)	Shear stress Level 1 (α)	Serviceability verification Level 1 (α)
56t	0.00	0.00	0.00	0.00
5x13t	0.00	0.00	0.00	0.00
6x12t	0.00	0.00	0.00	0.00
6x13t	0.00	0.00	0.00	0.00
7x13t	0.02	0.00	0.00	0.00
8x13t	0.05	0.00	0.00	0.00
9x13t	0.08	0.00	0.00	0.00

Level 3 assessment

Level 3 assessment foresees that the evaluator can update the characteristic values of the variables used in the assessment, based on the results of material testing and observations. As discussed above, the procedure leaves the evaluator free to test the materials, without specifying the minimum number of samples or the type of test. However, if the evaluator wants to use the test results quantitatively, a Bayesian probabilistic update technique should be applied.

In this case study, Level 3 was not considered due to the costs involved in obtaining the updated material properties based on in-situ tests. However, a simulation of Level 3 assessment has been performed in order to show the methodology when in-situ tests are available.

Bayesian inference is a tool that is able to treat various types of information following a unified approach (Sonda et al., 2003). In our case Bayesian inference can be used to update the *a priori* probability distribution function using information obtained from experimental tests (*a posteriori*) In Section 5.6 a brief introduction to Bayesian statistics is given while in the following only the application to the updating of a random variable is described (Fig. 32).

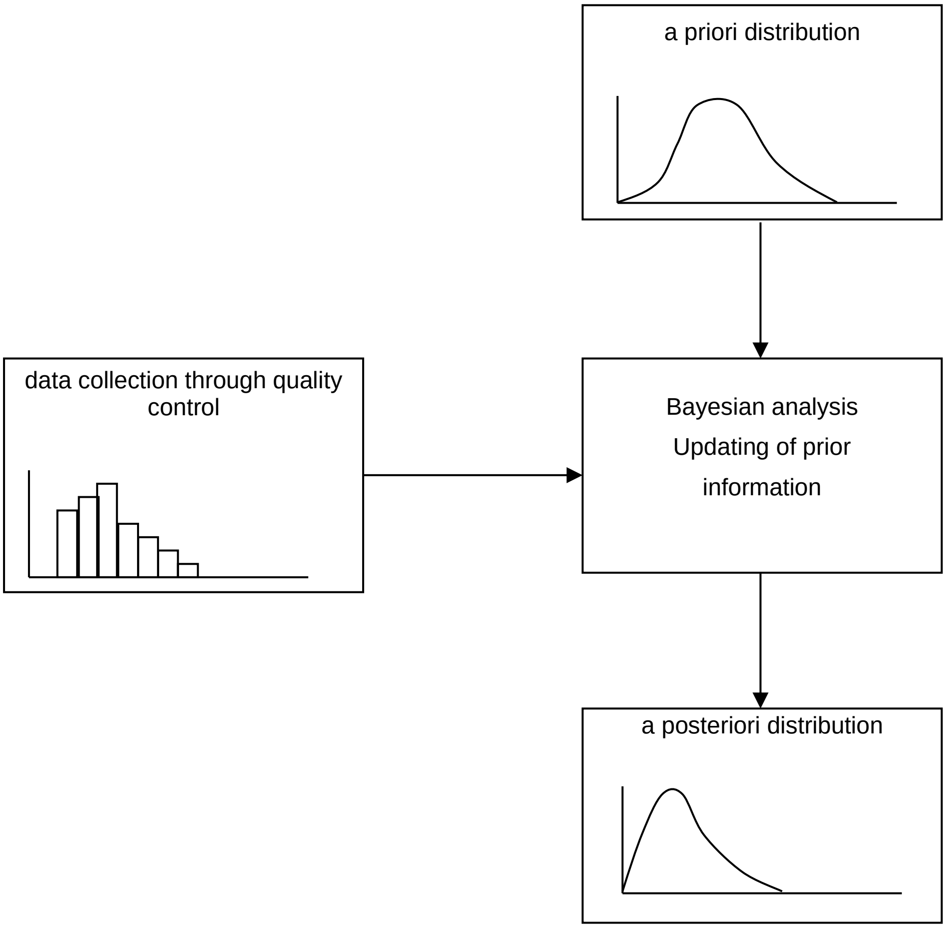


Fig. 32: Updating of a random variable (Sonda et al., 2003)

The basic assumption in the following is that all the random variables could be described by Gaussian likelihood functions.

The *a priori* information of the random variable *x* can be expressed as follows (35):

$$f_x(x) = N(\mu_0, \sigma_0) \tag{35}$$

where μ_0 is the mean value and σ_0 is the standard deviation of the *a priori* Gaussian likelihood function. This distribution take into account all the uncertainties that can be estimated *a priori,* including those regarding the mean value. If μ_1 represents the local uncertainties of the mean value μ_0, μ_1 can also be considered a random value (36):

$$g(\mu_1) = N(\mu_0, \sigma_2) \tag{36}$$

As can be noticed, the mean value is still equal to μ_0, while the standard deviation σ_2 is a part of σ_0, the standard deviation of the random variable x.
If μ_1 is known, the conditional probability of the random variable x can be expressed as follows (37):

$$f_x(x|\mu_1) = N(\mu_1, \sigma_1) \tag{37}$$

where (38):

$$\sigma_0^2 = \sigma_1^2 + \sigma_2^2 \tag{38}$$

Where a sample of n observations for the random variable x is available ($\bar{x}$), using Bayes' theorem the conditional probability of μ_1 can be expressed as follows (39):

$$g(\mu_1|\bar{x}) = \frac{f(\bar{x}|\mu_1) \cdot g(\mu_1)}{f_x(\bar{x})} \tag{39}$$

where $f(\bar{x}|\mu_1)$ is the likelihood and $f(\bar{x})$ is equal to (40):

$$f(\bar{x}) = \int_{-\infty}^{+\infty} f(\bar{x}|\mu_1) \cdot g(\mu_1) d\mu_1 \tag{40}$$

Having determined $g(\mu_1|\bar{x})$ from equation (39), the updated probability distribution function of the random variable x, is equal to (41):

$$f(x|\bar{x}) = \int_{-\infty}^{+\infty} f(\bar{x}|\mu_1) \cdot g(\mu_1|\bar{x}) d\mu_1 \tag{41}$$

From equations (35), (36) and (37), the following equation can be obtained (42):

$$g(\mu_1|\bar{x}) = N(\mu_3, \sigma_4) \tag{42}$$

where:

$$\mu_3 = \frac{(n \cdot x_m \cdot \sigma_2^2 + \mu_0 \cdot \sigma_1^2)}{(n \cdot \sigma_2^2 + \sigma_1^2)} \tag{43}$$

$$x_m = \frac{1}{n} \sum_{i=1}^{n} x_i \tag{44}$$

$$\sigma_4^2 = \frac{(\sigma_1^2 + \sigma_2^2)}{(\sigma_1^2 + n\sigma_2^2)} \tag{45}$$

The distribution function (42) is equal to equation (36) updated with the sample of n observations of the random variable x. The parameters μ_3 and σ_4 are the corresponding μ_0 and σ_2 assumed *a priori*.
Substituting equations (36) and (37) into (40), the updated distribution function of the random variable x can be obtained (46):

$$f_x(x|\bar{x}) = N(\mu_3, \sigma_3) \tag{46}$$

where (47):

$$\sigma_3^2 = \sigma_1^2 + \sigma_4^2 \tag{47}$$

The methodology discussed above can be used here in the *Fiume Adige* case study to update the properties of the materials. Level 3 assessment should be used only in the case of a negative verification of Level 2 assessment. In this case the bending moment verification provides negative results. The material that most influences the capacity of the resistance moment of a precast girder is the steel, as the concrete determines only the height of the neutral axis and so the improvement in the capacity would be very small. Therefore, it is useful to perform experimental tests only for the strands.

Let us suppose that the evaluator samples three strands and tests them with a tensile standard test. The characteristic value of strength of prestressing steel obtained from the design documentation is f_{tk}=1900 MPa. Let us suppose that the results of the tests are those reported in Table 19.

Table 19: Results of tensile standard tests of the strands

Sample	f_{tk} (MPa)
1	2021
2	1999
3	2219

We use the methodology discussed above and assume a Gaussian distribution function for both *a priori* and *a posteriori* distribution of strength of prestressed steel. We assume the following values that characterize the *a priori* distribution, as recommended by Zandonini et al. (2012):

- f_{ptk}=1900 MPa
- σ_0=74 MPa

which corresponds to a mean value of f_{pt0}=2021 MPa. Where there is no information on the value to assume for σ_1 and σ_2 Zandonini et al. (2012) advises choosing (48):

$$\sigma_1 = \sigma_2 \tag{48}$$

From equations (38) and (48) we obtain (49):

$$\sigma_1 = \sigma_2 = 52.32MPa \tag{49}$$

Substituting the values in the equations (43), (44), (45) and (47) we obtain (50), (51), (52) and (53):

$$\mu_3 = \frac{(nx_m\sigma_2^2 + \mu_0\sigma_1^2)}{(n \cdot \sigma_2^2 + \sigma_1^2)} = \frac{(3 \cdot 2080 \cdot 52.32^2 + 2021 \cdot 52.32^2)}{(3 \cdot 52.32^2 + 52.32^2)} = 2065\ MPa \tag{50}$$

$$x_m = \frac{1}{n}\sum_{i=1}^{n} x_i = \frac{2021 + 1999 + 2219}{3} = 2080\ MPa \tag{51}$$

$$\sigma_4^2 = \frac{(\sigma_1^2 + \sigma_2^2)}{(\sigma_1^2 + n\sigma_2^2)} = \frac{52.32^2 + 52.32^2}{52.32^2 + 3 \cdot 52.32^2} = 684.3\ \rightarrow\ \sigma_4 = 26.2\ MPa \tag{52}$$

$$\sigma_3^2 = \sigma_1^2 + \sigma_4^2 = 52.32^2 + 26.2^2 = 3421\ \rightarrow\ \sigma_3 = 58.5\ MPa \tag{53}$$

The updated characteristic value of strength of prestressing steel can be obtained from the equation (54):

$$f_{tk} = \mu_3 - 1.645 \cdot \sigma_3 = 2065 - 1.645 \cdot 58.5 = 1969\ MPa \tag{54}$$

It can be noticed that the increment in the nominal value that can be used for the calculations is equal to 3.6%. This means that in this case, Level 3 assessment shows a decrease in the lack of capacity of 3.6%.

4.2.2 Rio Cavallo

The bridge is located in the Municipality of Calliano, close to the town of Trento, Italy. The bridge is shown in Fig. 33. This is a four multiple arch bridge, built in 1940, with the span length *l*=12m. The carriageway is 11m in overall width. The arch thickness at the crown is 0.60m and at the abutment is 0.90m. The maximum thickness of the backfill is 1.9m.

The main dimensions of the structure are reported in the following and in Fig. 34:

Total bridge length:	55.4 m
Number of span:	4
Arch length:	12 m
Rise:	2.4 m
Arch thickness at the crown:	0.6 m
Arch thickness at the abutment:	0.9 m
deck width:	11 m

a)

b)

Fig. 33: a) and b): "Rio Cavallo", Municipality of Calliano (TN, Italy)

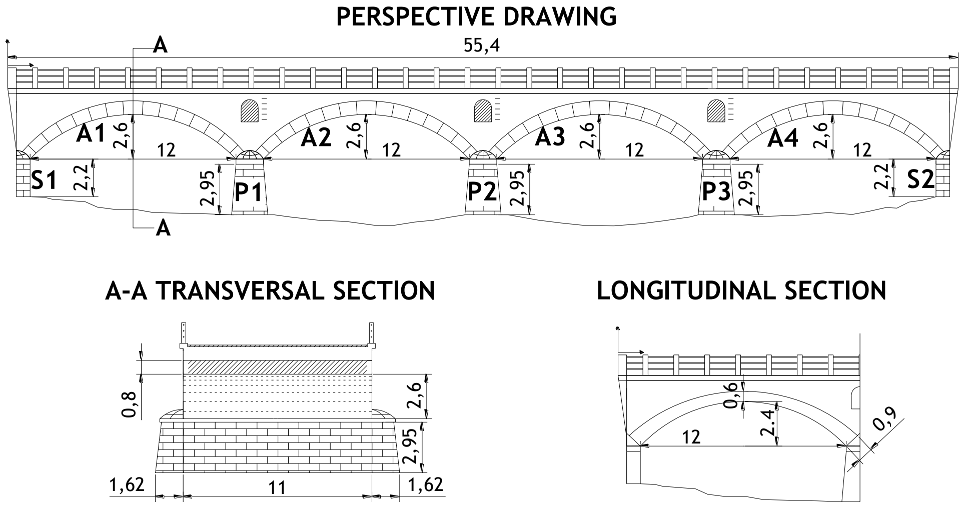


Fig. 34: Geometry of the Rio Cavallo bridge

Table 8 shows the lack of capacity for each APT load using Level 0 assessment. As for *Fiume Adige* bridge, these values are calculated by a Matlab program with the procedure described in Chapter 3.

Table 20: Results of Level 0 assessment of the Rio Cavallo bridge

APT load	Level 0 assessment (α)
56t	0.20
5x13 tons	0.33
6x12 tons	0.30
6x13 tons	0.33
7x13 tons	0.33
8x13 tons	0.33
9x13 tons	0.33

The design code used by the bridge designer is the following:

- MM.LL.PP. Normale n.8 15/09/1933

The bridge designer did not use the traffic loads foreseen by the design codes but imposed a distributed uniformly load of 1000 kg/m^2. He/she justified this choice saying that for the arch bridge this load has greater effect than the design load.

The permanent loads used by the bridge designer are the following:

- Arch weight: G_1 = 2100 kg/m^2
- Backfill weight: G_2 = 2100 kg/m^2

Level 1 assessment

The verification of the arch is carried out with an equilibrium analysis checking the most stressed cross-sections of the arch. This is the same procedure followed by the bridge designer. Level 1 assessment is based only on the analysis of the existing documents, without revising the assumed material properties or calculation model. Although the calculation model is reported in the documentation, it is impossible to simply replace the APT load with design load as the verifications are obtained graphically. Since the load distributions are different, the thrust line is different too. Automatically Level 1 assessment does not involve improvements in the capacity and it is also necessary to make some additional hypotheses to those made by the bridge designer. This means that Level 1 assessment is not useful for arch bridges and only Level 2 assessment can be performed to verify the bridge.

Level 2 assessment

The equilibrium analysis made by the bridge designer foresees the verification of the arch taking into account the loads acting on 1m of the arch width. For this reason the verifications of the arch taking into account the free travel or the bridge crossed in the center of the carriageway are the same. Assuming the same hypothesis, here only half of the axle weight of the APT load is considered. In fact the spacing between the two wheels is defined equal to 2m. This means that the concentrated force acting on an arch of 1m width is equal to half of the axle (13t/2=6.5t). Moreover, for the equilibrium analysis, the spread of the concentrated forces of the APT load in the backfill is neglected. In other words, in favor of safety, the load is considered as concentrated forces acting on the arch.
The bridge designer verified the arch with the following combination of the loads:

1) dead load (Fig. 35) that is equal to G_1+ G_2;
2) dead load and traffic load Q on the whole arch length (Fig. 36a);
3) dead load and traffic load Q on half arch length (Fig. 36b).

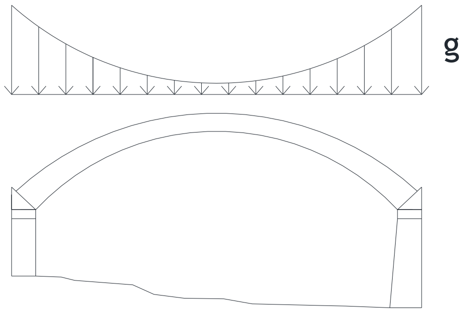


Fig. 35: Dead load

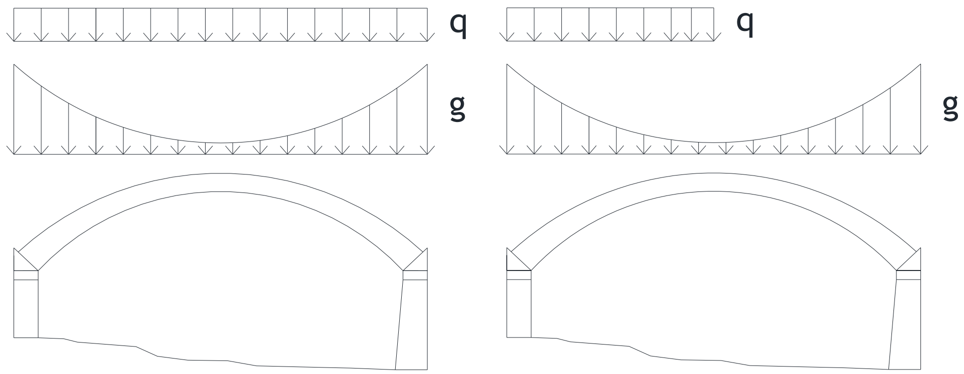


Fig. 36: a) Dead load and traffic load on the whole arch length, b) dead load and traffic load on half arch length

The same load combinations are considered, replacing the traffic load with the APT load. Clearly the first combination has not been performed in the following, as it remains the same as that computed by the bridge designer.

Firstly, the verification considering the APT load located on the whole arch length is performed. Only the verification of the 9x13t APT load is reported here. It should be noticed that if the 9x13t APT load is verified the other APT loads are verified too.

To verify the arch element it is sufficient demonstrate that there exist a satisfactory line of thrust that balances the given loading (Heyman, 1982). In other words it is sufficient to find a thrust line in equilibrium with the external loading *which lies everywhere within the arch ring* (Heyman, 1982).

To perform the equilibrium analysis of the arch, it is possible to study only half of the arch length, considering the symmetry of the arch. As already mentioned, the concentrated load is equal to half of the weight of one axle and only 1m width of the arch is studied. Therefore, the load is equal to (55):

$$Q=6.5\ t \tag{55}$$

At the crown the load acting is equal to half of *Q* since only half of the arch is studied. The numerical results and the equilibrium analyses (Fig. 37) are reported in the following:

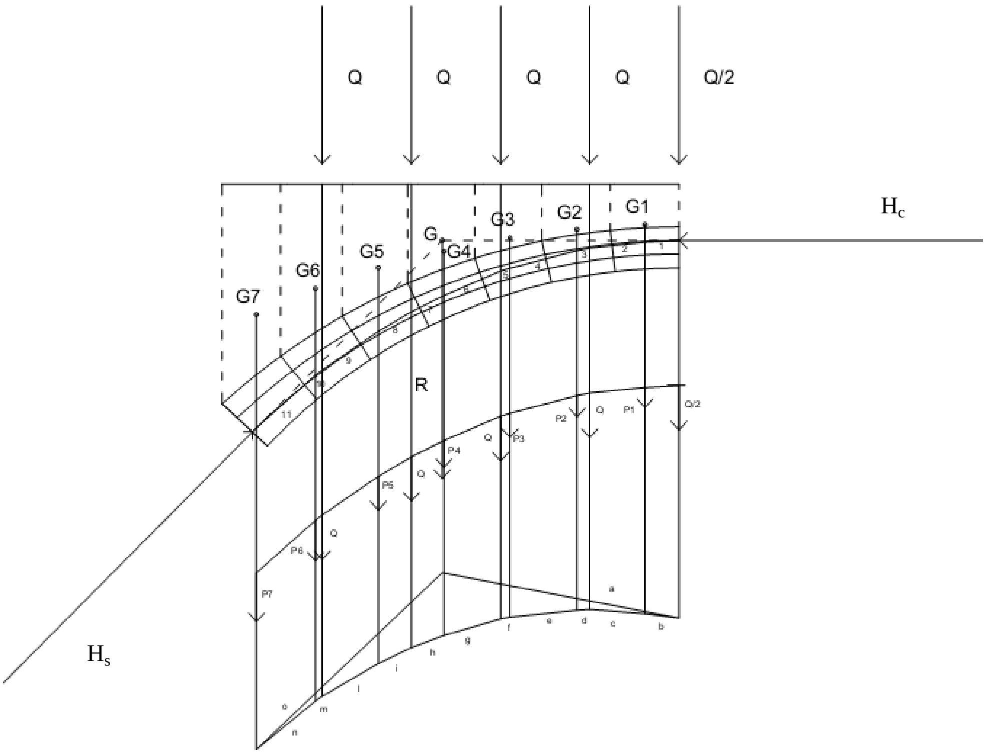


Fig. 37: Construction of the funicular polygon

- H_s = 85351 kg
- H_c = 60023 kg

where R is the resultant force of the loads, and H_s and H_c are the thrust at the abutment and at the crown respectively. As can be seen in Fig. 37, the thrust line is always within the middle third and consequently the cross-sections are always compressed. Following the same procedure reported by the bridge designer, the compression stresses ($\sigma_{I,c}$) at the crown and at the abutment ($\sigma_{I,s}$) are checked. The results are reported in the following:

$$\sigma_{I,c} = \frac{H_c}{A_c} + \frac{M_c y_c}{I_c} = \frac{60023}{6000} + \frac{10 \cdot 60023 \cdot 30 \cdot 12}{100 \cdot 60^3} = 20.00 \frac{kg}{cm^2} = 2.00\ MPa \qquad (56)$$

$$\sigma_{I,s} = \frac{H_s}{A_s} + \frac{M_s y_s}{I_s} = \frac{85351}{9000} + \frac{15 \cdot 85351 \cdot 45 \cdot 12}{100 \cdot 90^3} = 18.94 \frac{kg}{cm^2} = 1.89\ MPa \qquad (57)$$

where A is the area, I is the momentum of inertia, y is the distance between the baricenter and the compressive edge, and the subscripts c and s represent the cross-sections at the crown and abutment respectively.
The compression stresses are very small and therefore the verification is positive.

Secondly, the verification considering the APT load located on half of the arch length is performed. In this case the thrust line is not symmetrical and so the whole arch is studied. The same hypotheses are assumed as in the previous analyses. The numerical results and the equilibrium analysis (Fig. 38) are reported in the following:

- H_s = 83206 kg
- H_c = 54181 kg

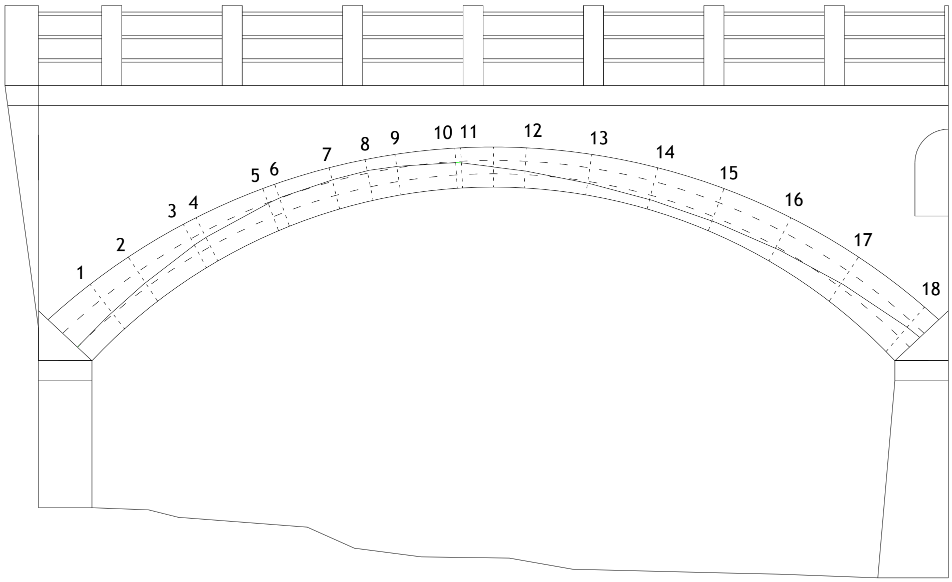


Fig. 38: Resulting thrust line with traffic load on half arch length

As can be seen in Fig. 37, the thrust line is not always within the middle third and consequently tension stress appears at the edge of some cross-sections. The thrust line, however, is within the thickness of the arch and consequently the equilibrium is guaranteed (Heyman, 1982). The results of the two most stressed cross-sections (called section 8 and 14 for their position in the arch) and the cross-sections at the crown and at the most unfavorable abutment are reported:

$$\sigma_{I,8} = \frac{F_8}{A_8} + \frac{M_8 y_8}{I_8} = \frac{54571}{6000} + \frac{14 \cdot 54571 \cdot 30 \cdot 12}{100 \cdot 60^3} = 21.83 \frac{kg}{cm^2} = 2.18 MPa \tag{58}$$

$$\sigma_{II,8} = \frac{F_8}{A_8} - \frac{M_8 y_8}{I_8} = \frac{54571}{6000} - \frac{14 \cdot 54571 \cdot 30 \cdot 12}{100 \cdot 60^3} = -3.65 \frac{kg}{cm^2} = -0.37 MPa \tag{59}$$

$$\sigma_{I,14} = \frac{F_{14}}{A_{14}} + \frac{M_{14} y_{14}}{I_{14}} = \frac{57249}{6600} + \frac{19 \cdot 57249 \cdot 33 \cdot 12}{100 \cdot 66^3} = 23.65 \frac{kg}{cm^2} = 2.37 MPa \tag{60}$$

$$\sigma_{II,14} = \frac{F_{14}}{A_{14}} - \frac{M_{14} y_{14}}{I_{14}} = \frac{57249}{6600} - \frac{19 \cdot 57249 \cdot 33 \cdot 12}{100 \cdot 66^3} = -6.33 \frac{kg}{cm^2} = -0.63 MPa \tag{61}$$

$$\sigma_{I,c} = \frac{H_c}{A_c} + \frac{M_c y_c}{I_c} = \frac{54181}{6000} + \frac{5 \cdot 54181 \cdot 30 \cdot 12}{100 \cdot 60^3} = 13.54 \frac{kg}{cm^2} = 1.35\ MPa \tag{62}$$

$$\sigma_{II,c} = \frac{H_c}{A_c} - \frac{M_c y_c}{I_c} = \frac{54181}{6000} - \frac{5 \cdot 54181 \cdot 30 \cdot 12}{100 \cdot 60^3} = 4.54 \frac{kg}{cm^2} = 0.45\ MPa \tag{63}$$

$$\sigma_{I,s} = \frac{H_s}{A_s} + \frac{M_s y_s}{I_s} = \frac{83206}{9000} + \frac{15 \cdot 83206 \cdot 45 \cdot 12}{100 \cdot 90^3} = 18.49 \frac{kg}{cm^2} = 1.85\ MPa \qquad (64)$$

$$\sigma_{II,s} = \frac{H_s}{A_s} - \frac{M_s y_s}{I_s} = \frac{83206}{9000} - \frac{15 \cdot 83206 \cdot 45 \cdot 12}{100 \cdot 90^3} = 0 \frac{kg}{cm^2} = 0\ MPa \qquad (65)$$

where the subscripts 8 and 14 represent the sections 8 and 14 respectively.
The results show that the stresses in the arch are very small and therefore the verifications are positive. Also the shear stresses in the cross-sections of the arch are always low and therefore the verifications are satisfied.
For the sake of brevity the calculations of the verifications of the piers and abutments are not reported here. However, the total loads acting on these elements are lower than the design load and so the verifications are automatically positive.

In the following (Table 21 and Table 22) the final results for each APT load and assessment level are reported. As can be noticed, the verification is satisfied in both travel conditions (free travel and bridge crossed in the center of the roadway).

Table 21: Results of Level 0 assessment (free travel)

APT load	Level 0 assessment (α)	Level 2 assessment arch verification (α)	Level 2 assessment pier and abutment verification (α)
56t	0.20	0.00	0.00
5x13 tons	0.33	0.00	0.00
6x12 tons	0.30	0.00	0.00
6x13 tons	0.33	0.00	0.00
7x13 tons	0.33	0.00	0.00
8x13 tons	0.33	0.00	0.00
9x13 tons	0.33	0.00	0.00

Table 22: Results of Level 0 assessment (bridge crossed in the center of the roadway)

APT load	Level 0 assessment (α)	Level 2 assessment arch verification (α)	Level 2 assessment pier and abutment verification (α)
56t	0.20	0.00	0.00
5x13 tons	0.33	0.00	0.00
6x12 tons	0.30	0.00	0.00
6x13 tons	0.33	0.00	0.00
7x13 tons	0.33	0.00	0.00
8x13 tons	0.33	0.00	0.00
9x13 tons	0.33	0.00	0.00

4.3 Discussion of the results

In Section 4.2 the results of the application of assessment Levels 1 and 2 to the two case studies have been reported. For the girder bridge *Fiume Adige,* both assessment levels have been applied while for the arch bridge *Rio Cavallo* only Level 2 assessment has been performed because, as discussed above, there is no sense in the application of Level 1 assessment for this type of bridge.

The results obtained for the *Fiume Adige* show that the precision of Level 1 assessment reveals a higher level of capacity than that given by the simplified model in Level 0 assessment (i.e. verify potential reserve of capacity by examining the design documentation). In fact, in the case of free travel and of the 9x13t APT load, the decrease in parameter α is 27% (see Table 23). This value represents the overstrength of the bridge with respect to the design loads. The reason of the large decrease is that there are many more reinforcing bars than necessary. The critical verification at Level 0 assessment was the shear verification but the design code involves a minimum transverse reinforcement greater than necessary. It is worth emphasizing that this result could not be obtained a priori as the extent of the excess reinforcing bars is not known. For the limit state where Level 1 assessment provided negative results (i.e. the capacity is still insufficient), Level 2 assessment was performed. Refining the models used by the bridge designer, an additional decrease in the lack of capacity (α) has been obtained. For the bending moment a plastic transverse distribution of the load instead of elastic has been used. The additional decrease from Level 1 to Level 2 assessment in the lack of capacity was of 6% (Table 23). For shear, more refined models have not been used as the shear verification is positive in Level 1 assessment.

Table 23: Results of the case study Fiume Adige

Free Travel	Level 0 (α)	Level 1 (α)	Level 2 (α)
APT load 9x13t	0.39	0.12	0.06

It should be noticed that the decrease in the lack of capacity between Level 1 and Level 2 assessment is possible only for the limit states where an ultimate analysis or a modification of the static model is possible. In general the serviceability verifications cannot be considered in Level 2 as the elastic hypotheses of the bridge designer cannot be changed.

The results obtained for the *Rio Cavallo* show that the bridge can withstand the new APT loads. The verifications implied a complete re-assessment of the elements of the bridge (arch, piers and abutments). All the verifications were performed graphically drawing the funicular polygon in the elements. Safety is calculated by checking that the thrust line is always within the elements and the stress in each cross section is low enough. As can be seen in Table 24 the verifications for the 9x13=117t APT load are positive. The large decrease in the lack of capacity can be explained by the significant intrinsic geometrical factor of safety of arch bridges. This is guaranteed by the high value of permanent loads of arch bridges and by the hypothesis of the design. The assumption that the material has an infinite compressive strength (Heyman, 1982) implies that the stress are very low and in general an increase in the live load does not cause structural problems to the bridge.

Table 24: Results of the case study Rio Cavallo

Free Travel	Level 0 (α)	Level 2 (α)
APT load 9x13t	0.33	0.00

In both case studies, for the travel condition of the bridge being crossed in the center of the roadway, the bridges are acceptable for each APT overweight vehicle model. The reason is that the deck width is sufficient as there is more than one line load and consequently that the vehicle crossing the bridge alone entails the neglect of one or more line loads in the verifications.

4.4 Conclusions

In this Chapter Level 1 and 2 assessment of the girder bridge *Fiume Adige* and the arch bridge *Rio Cavallo* have been applied in order to test the method. The Chapter has shown how to apply the procedure to two real cases and the necessity to apply the method to each design verification performed by the bridge designer.

In the case of the girder bridge *Fiume Adige*, the results have shown that, performing Level 1 assessment, all the APT loads less than or equal to the 6x12t are acceptable, while performing Level 2 less than or equal to the 7x13t (considering only the collapse verifications). Level 0 analysis had shown that only the 56t APT load was acceptable for the bridge. A simulation of Level 3 assessment was shown using Bayesian probabilistic technique to update the characteristic values of the variables used in the assessment, based on the results of material testing and observations.

In the case of the arch bridge *Rio Cavallo* the results have shown that all the APT loads, in both travel conditions, can cross the bridge without reducing the original design safety level. Also in this case Level 0 analysis had shown that only the 56t APT load was acceptable for the bridge.

In conclusion, Level 1 and 2 assessment can lead to good decreases in terms of the lack of capacity of the bridge that were, for example, equal to 33% in the case of 9x13t APT load (free travel condition, Fiume Adige bridge).

Once again, these results could not be obtained a priori as the overstrength of the bridge and the design structural details are not known.

5 DECISION MAKING

5.1 Introduction

Decision making under uncertainty is about making choices whose consequences are not completely predictable, because events will happen in the future that will affect the consequences of actions taken now (Parmigiani & Inoue, 2009).

In Chapters 3 and 4 the criteria for re-assessing bridges for overweight loads and the application to two case studies were shown. The application of Level 1 and 2 assessment were made automatically, without considering whether it was the most rational choice. In fact, if the objective is to allow the travel of an overweight load without reducing the original design safety level, when Level 0 assessment shows that the lack of capacity is very high, the most rational choice could be the full formal re-assessment of the bridge made by a structural engineer with possible retrofit strengthening rather than proceeding with assessment levels 1, 2 and 3.

The aim of this Chapter is to define a decisional model to support the manager in his/her choices. Therefore, the objective is to obtain a methodology to determine the limit values of the lack of capacity (α), which make it more cost effective to employ a certain assessment level (Level 1, 2 or 3) or a full formal re-assessment with possible retrofit strengthening. In other words the goal is to propose a decision making process to guide the manager in making the most cost effective decision when selecting the appropriate assessment level.

In particular, in Section 5.2 and 5.3, I report the basic concepts of decision theory and of expected utility theory. In Section 5.4 I define a general decision tree for girder and arch bridges, and in Section 5.5 I show how to optimize the thresholds of the lack of capacity α in order to minimize the expected costs. This aim is pursued in order to guide the decision regarding the overweight load made by the transportation agencies. In addition, in Section 5.6, a methodology to update parameters on the basis of a number of case studies is proposed. Bayesian statistics is used to implement this methodology.

5.2 Basic concepts

The symbology used in this Chapter is taken from the master's thesis of Cappello (2013), who studied in depth some of the aspects included in the present thesis by applying expected utility theory with the aim of demonstrating its advantages.

The decision tree (Fig. 39) is the appropriate representation of any process involving decisions. In Fig. 39 the action a_1 starts from the decision node (a square), and leads to the chance node (a circle), which in turn leads to two possible consequences θ_1 or θ_2. The probabilities associated with the states θ_1 and θ_2 are the values of $p_{\theta 1}$ and $p_{\theta 2}$ respectively. $z\theta_1|a_1$ and $z\theta_2|a_1$ are the outcomes observed after the occurrence of the situations θ_1 and θ_2, and $u(z\theta_1|a_1)$ and $u(z\theta_2|a_1)$ are the utilities assigned to these outcomes.

In a general decision process it is possible to have many actions a_j and the set of these actions is called A. Similarly, the set of the states θ_i is represented by the symbol Θ. One can imagine that the outcome $z\theta_i|a_j$ corresponding to the condition θ_i, observed after the action a_j is chosen, is often, but not always, an amount of money. The set of all the outcomes is called Z.

Once the utility function $u(z\theta_i|a_j)$ is known, the utility associated with an outcome $z\theta_i|a_j$ can be calculated. The utility function represents the utility that a decision maker wants to associate to an outcome. There can be several utility functions and the result, i.e. the value of the utility, depends on which one is chosen. Different functions $u(z\theta_i|a_j)$ can be useful to represent different cases of decision making process.

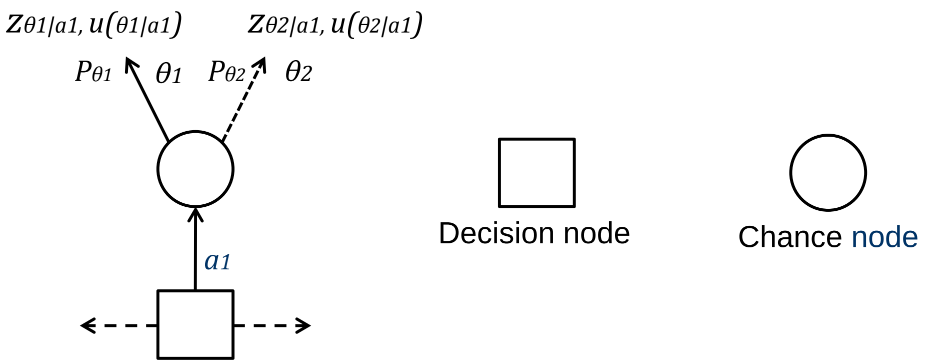


Fig. 39: Example of a decision tree and symbols used

5.3 Expected utility theory

The expected utility theory is a quantitative evaluation of the utility of the outcomes. It provides the probability that each outcome will occur (Jordaan, 2005). The action chosen is the one that has the greatest expected utility.

The expected utility assigned to an action can be considered as the sum of the utility of the outcomes (Cappello, 2013). The expected value can be calculated as follows (66).

$$\mathcal{U}(a) = \sum_{z \in Z} u(z) \cdot p(z) \tag{66}$$

where the probability of the outcome $p(z)$, if the action a is chosen, is equal to (67):

$$p(z) = \int_{\theta : a(\theta) = z} p(\theta) \cdot d\theta \tag{67}$$

As the higher $U(a)$ the better, the optimal action a^*, also called Bayes action, is the one that maximizes $U(a)$ (68):

$$a^* = argmax\,\mathcal{U}(a) \tag{68}$$

which corresponds to the maximum expected utility (Parmigiani & Inoue, 2009).

If the Bayes action is chosen using losses, it is necessary to define the loss function as the opposite of the utility (69):

$$L_u(\theta, a) = -u(z) \tag{69}$$

where the space $(\theta, a) \in (\Theta, A)$. Unlike the utility, the definition of the *loss function* predicts that at least one action should be with zero loss (Cappello, 2013). As a consequence, the *regret loss function* $L(\theta, a)$ has been defined (70), obtained as a transformation of the utility-derived loss function (Cappello, 2013).

$$L(\theta, a) = L_u(\theta, a) - \min_{a \in A}[L_u(\theta, a)] \tag{70}$$

Defining the expected loss as follows:

$$\mathcal{L}(a)=\sum_{z\in Z} L(z)\cdot p(z) \tag{71}$$

the Bayes action a^* can be determined as follows (72), using the same principle of the *expected utility* already applied (68):

$$a^* = \operatorname{argmin} \mathcal{L}(a) \tag{72}$$

5.4 Decision tree

In this Chapter, I develop a method that can support the decision-making process for the APT bridge stock. In particular, the aim is to obtain the limit values of the lack of capacity (α) which makes it more convenient to employ a certain assessment level. The various simplified assessment levels that are available for the APT, as a consequence of a negative verification, were shown in Table 4.

Firstly, the hypothesis is to use a single level tree diagram (Fig. 40), that is, to assume that when an assessment level has been taken, the only possible results are either the positive verification of the bridge or its replacement. The decision tree in Fig. 40 is valid only for girder bridges, as applying Level 1 and 3 assessment to arch bridges is not useful (see Chapter 3 and 4). The methodology explained in the following is only a simple example to introduce the problem of the determination of the limit values of the lack of capacity. After this example the final decision tree and a more rigorous discussion will be shown.

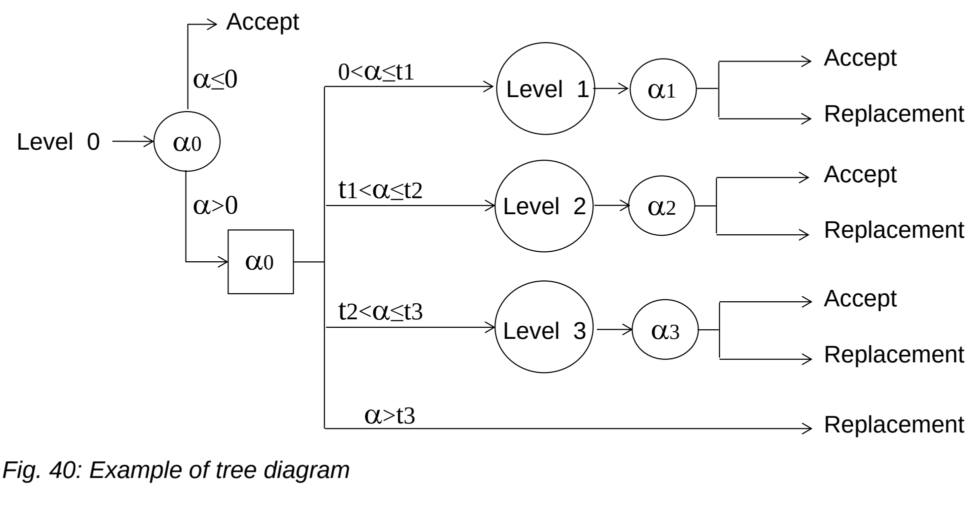


Fig. 40: Example of tree diagram

Table 25 shows the cost of each type of assessment level as a function of the total bridge cost (C_c). These are plausible values chosen by the author only in order to explain the method. In the final decision tree the details of costs will be shown and adapted to the real needs of the APT (see Chapter 6).

Table 25: Building cost percentage of different assessment levels.

Assessment Level	**Cost**
Level 1	$0.5\%C_c$
Level 2	$1.5\%C_c$
Level 3	$10\%C_c$
Replacement	$100\%C_c$

A basic principle of the expected utility theory is here used to determine the appropriate assessment level. Assuming a lognormal probability distribution for the decrease in the lack of capacity from one assessment level to another, the inverse cumulative distribution function describes the probability that, as a result of a given assessment level, the verification becomes positive (73).

$$P = 1 - \Phi\left(\frac{1}{\beta} ln\left(\frac{\alpha}{\alpha_{mean}}\right)\right) \tag{73}$$

where β is the lognormal standard deviation that incorporates aspects of uncertainty; and α_{mean} is the decrease in the lack of capacity after using a certain assessment level. We imposed β equal to 0.7.

Table 26 lists the values of α_{mean} that correspond to a probability of 50% that the verification becomes positive. These values are the decrease in the lack of capacity after the assessment level (see equation (74)). The values were chosen by the author a priori. The mean decrease between two consecutive assessment levels ($\Delta\alpha$) is reported in Table 27.

$$\alpha_{i+1} = \alpha_i - \Delta\alpha_{i+1} \tag{74}$$

Table 26: Coefficient α corresponding to different assessment levels.

Assessment level	α_{mean} (α_i)
Level 1	0.08
Level 2	0.12
Level 3	0.20
Replacement	∞

Table 27: Mean decrease between two consecutive assessment levels.

Assessment level	$\Delta\alpha_{mean}$ ($\Delta\alpha_i$)
Level 1	0.08
Level 2	0.04
Level 3	0.08
Replacement	∞

Fig. 41 shows the inverse cumulative distribution function obtained using the values shown in Table 26. It represents the probability of obtaining a positive verification after using a certain assessment level as a function of the lack of capacity (α).

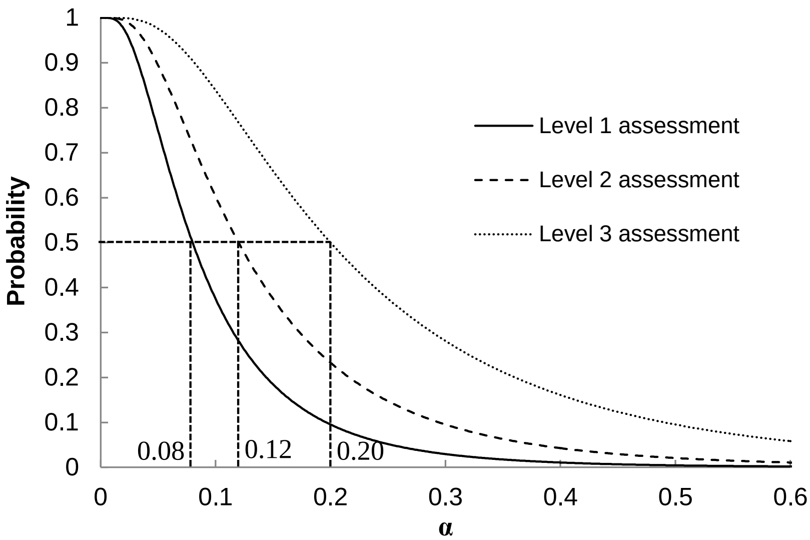


Fig. 41: Probability of obtaining a positive verification after using a certain assessment level as a function of α.

Once the probability corresponding to a positive verification using a given assessment level is obtained, the next step is to calculate the loss function. In this case the loss function is equal to the cost, which depends on both the assessment level and the consequence, according to equations (75) and (76).

$$L(Cost_{failure}) = Cost_{failure} = Cost_{assessment\_level} + C_c \quad (75)$$

$$L\left(Cost_{safe}\right) = Cost_{safe} = Cost_{assessment\_level} \tag{76}$$

The expected loss is equal to the following (77).

$$\mathcal{L}_i = Cost_{total_i} = Cost_{assessment\_level_i} + C_c\left(1 - P(\alpha_i)\right) \tag{77}$$

Using equation (77), the relations as a function of the bridge cost C_c (Fig. 42) can be built.

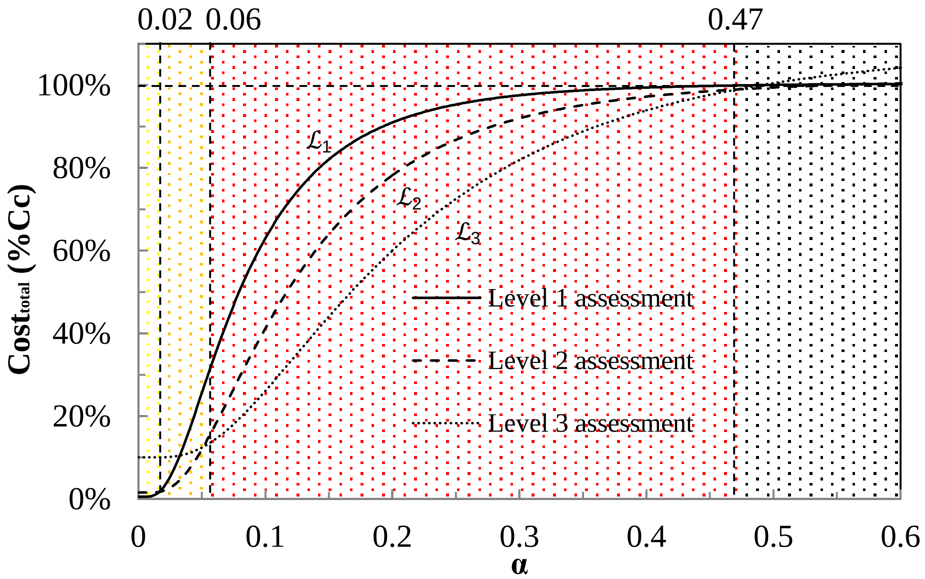


Fig. 42: Relationship between costs and lack of capacity

The lower curve, given a value of α, is the Bayes action, that which has the lower expected loss. Looking at the graph, it is therefore possible to determine the intervals in which it is more appropriate to choose a given assessment level. According to equation (77) it can be noted that there is a fixed cost (depending on the assessment level (Table 25)) and a variable cost that depends on the probability of obtaining a positive verification. The following values are obtained (Fig. 42):

- 0%<α<2% Level 1 assessment is most cost effective;
- 2%≤α<6% Level 2 assessment is most cost effective;
- 6%≤α<47% Level 3 assessment is most cost effective;
- α≥47% it is most cost effective to replace the bridge.

As discussed above, in this first step the hypothesis is to use a single level tree diagram, that is, to assume that when an assessment level has been applied, the only possible results are the positive verification of the bridge or its replacement. In reality it is possible that after an assessment level, the choice is to perform the subsequent assessment level. Moreover, according to the APT (see Chapter 6), there is no possibility of the bridge being replaced after Level 1. Replacement is possible only after Level 2 assessment as the hypothesis is that if the result of Level 1 assessment is negative, the only option is to perform Level 2 assessment. Furthermore, it is not possible to perform Level 3 assessment directly without having analyzed Level 1 and 2. These remarks are valid only for girder bridges as for arch bridges only Level 2 assessment is possible.
The possibility of utilizing a higher assessment level, in the case of a negative verification of a girder bridge, is illustrated in Fig. 43. The final decision tree for girder bridges (Fig. 43) has four decision node. The possible actions are:

- $a_{a,(i)}$: at Level i the APT load can cross the bridge, as can the overweight loads with lower total gross weight;
- $a_{r,(i)}$: at Level i the bridge should be replaced;
- $a_{n,(i)}$: the subsequent assessment level should be performed from Level i to Level $i+1$;

while the possible outcomes are:

- θ_s: the bridge withstands the overweight load;
- θ_f: the bridge collapses;
- θ_r: the bridge is replaced and the new bridge withstands the overweight load.

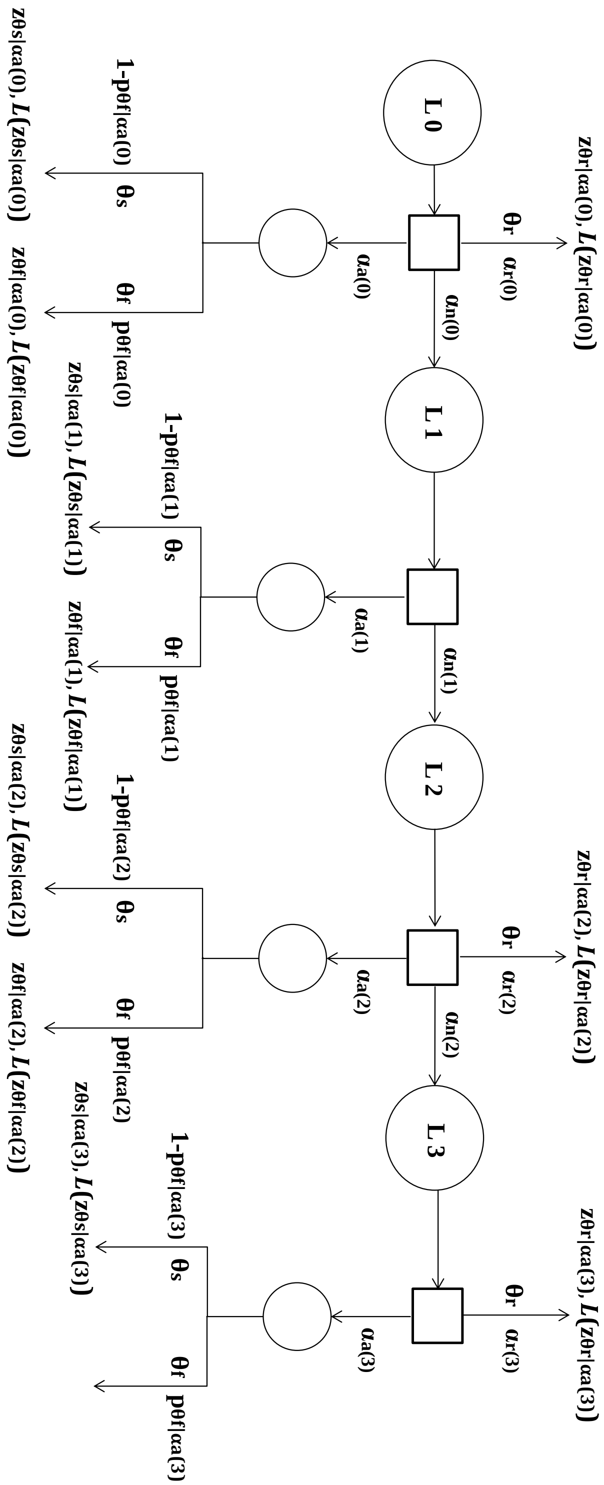


Fig. 43: Final decision tree for girder bridges with documentation

In Fig. 43 $p_{\theta_f|a_a,(i)}$ is the probability that the bridge collapses.
The symbols adopted are those used in a thesis by Cappello (2013), which as mentioned before, studied in depth some of the aspects included in this thesis. Cappello (2013) also studied the loss functions and the costs associated to the possible collapse of bridges. In the decision tree of Fig. 43 the outcomes take into account the possible collapse of the bridge after the decision to allow the crossing of the bridge.
Unlike the decision tree of Fig. 40, in this case the expected loss as a consequence of a given choice takes into account not only the cost of the assessment level but also the consequences of the collapse of the bridge.
After Level 0 assessment (L0) the decision maker has three options:

- $a_{a,(0)}$: the APT load can cross the bridge and the possible outcomes are:
  - the bridge collapses;
  - the bridge withstands the overweight load;
- $a_{r,(0)}$: the bridge is replaced and the new bridge withstands the overweight load;
- $a_{n,(0)}$: Level 1 assessment is performed and the result is α_1.

After Level 1 assessment (L1) the decision maker has two options:

- $a_{a,(1)}$: the APT load can cross the bridge and the possible outcomes are:
  - the bridge collapses;
  - the bridge withstands the overweight load;
- $a_{n,(1)}$: Level 2 assessment is performed and the result is α_2.

After Level 2 assessment (L2) the decision maker has three options:

- $a_{a,(2)}$: the APT load can cross the bridge and the possible outcomes are:
  - the bridge collapses;
  - the bridge withstands the overweight load;
- $a_{r,(2)}$: the bridge is replaced and the new bridge withstands the overweight load;
- $a_{n,(2)}$: Level 3 assessment is performed and the result is α_3.

After the final level of assessment (L3) the decision maker has two options:

- $a_{a,(3)}$: the APT load can cross the bridge and the possible outcomes are:

- the bridge collapses;
- the bridge withstands the overweight load;

- $a_{r,(3)}$: the bridge is replaced and the new bridge withstands the overweight load.

In the final decision tree for girder bridges, of Fig. 43, it has been supposed that the design documentation of the bridge exists. Although it should always exist, in many cases it does not (see Chapter 6). The decision tree needs to be modified when the design documentation is not available. Without design documentation Level 1 and Level 2 assessment are not possible, while Level 3 assessment can be done but with additional cost with respect to Level 3 with documentation. Firstly, the evaluator does not know the geometry of the bridge and a detailed survey on the bridge geometry is necessary. Secondly, the material properties are also unknown, and an extended campaign of in-situ testing and observations must be done. Finally, the construction date must be estimated and a simulation of the design process is necessary.
The decision tree for girder bridges without documentation is illustrated in Fig. 44:

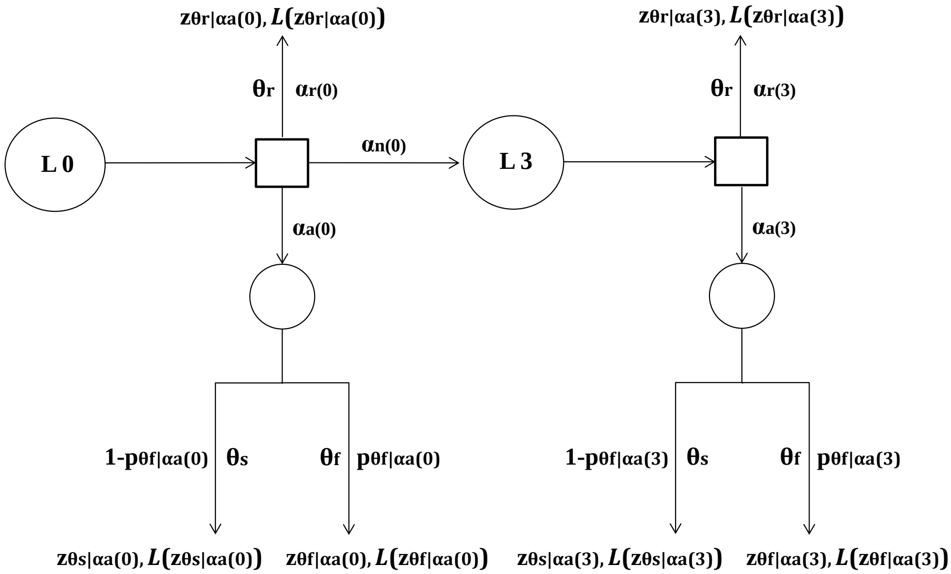


Fig. 44: Decision tree for girder bridges without documentation

The same symbols used in the decision tree for girder bridges with documentation are adopted.
After Level 0 assessment (L0) the decision maker has three options:

- $a_{a,(0)}$: the APT load can cross the bridge and the possible outcomes are:
    - the bridge collapses;
    - the bridge withstands the overweight load;
- $a_{r,(0)}$: the bridge is replaced and the new bridge withstands the overweight load;
- $a_{n,(0)}$: Level 3 assessment is performed and the result is α_3.

After Level 3 assessment (L3) the decision maker has two options:

- $a_{a,(3)}$: the APT load can cross the bridge and the possible outcomes are:
    - the bridge collapses;
    - the bridge withstands the overweight load;
- $a_{r,(3)}$: the bridge is replaced and the new bridge withstands the overweight load.

As discussed above, Level 1 and 3 assessment applied to arch bridges are not useful. The decision tree for arch bridges is composed of only Level 0 and 2 assessment. As for girder bridges, we have to take into account whether the design documentation exists. The difference between performing the Level 2 assessment with or without design documentation lies in the costs. The cost of Level 2 with and without documentation (henceforth called respectively Level 2A and Level 2B) are reported in Table 28 and Table 29 respectively:

Table 28: Costs for Level 3 assessment for arch bridges with design documentation

Item	**Cost**
First inspection with geometric survey, assessment of cracking, report	0.05%Cc
Calculation report	0.20%Cc
Total	0.25%Cc

Table 29: Costs for Level 3 assessment for arch bridges without design documentation

Item	**Cost**
First inspection with geometric survey, analysis of the cracking state, report	0.05%Cc
Geometric survey of the arch	0.20%Cc
In-situ test to check thicknesses	0.15%Cc
Calculation report	0.30%Cc
Total	**0.70%Cc**

The final decision tree for arch bridges is shown in Fig. 45

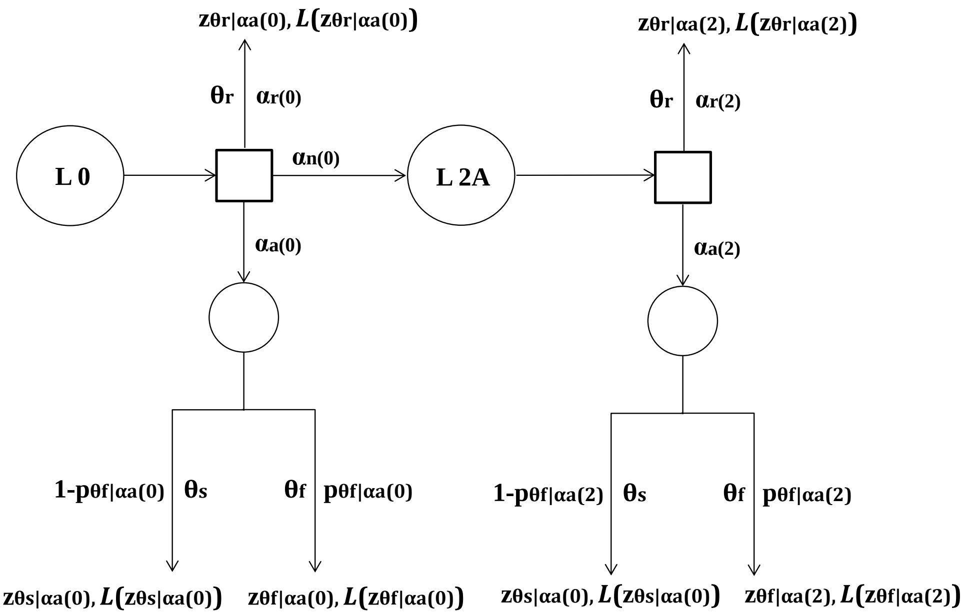


Fig. 45: Decision tree for arch bridges with design documentation

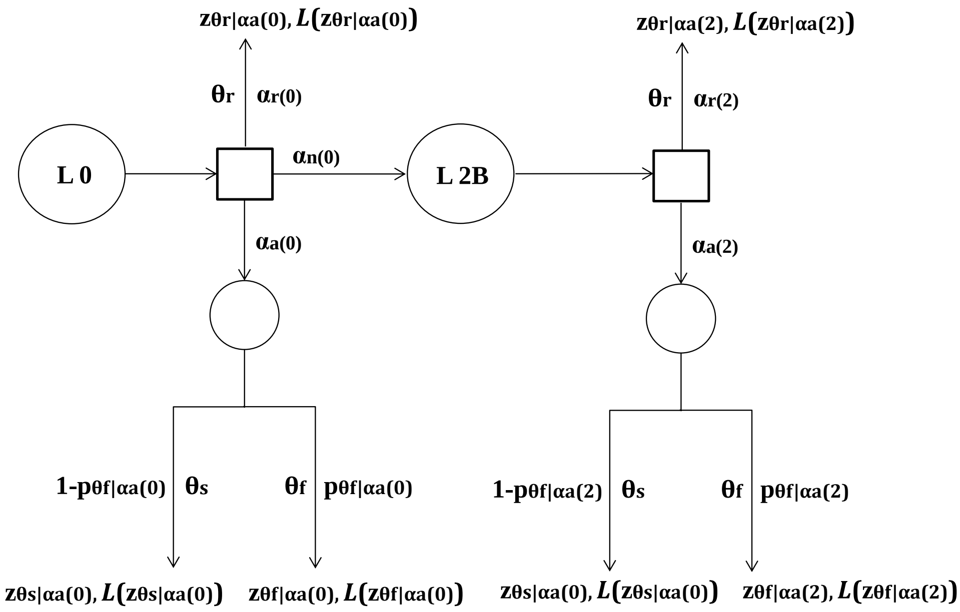


Fig. 46: Decision tree for arch bridges without design documentation

5.5 Optimization of alpha thresholds

The optimization of alpha thresholds has already been studied by Cappello (2013) who based his analyses and models on the present work. However, it is worth reporting the methodology of his thesis, which studied in depth the decision making part of this thesis. His thesis presents the results of alpha thresholds and Bayesian updating of parameters. In this Section and in Section 5.6 only the main results are reported, while a comprehensive discussion of these topics are available in the thesis by Cappello (2013). As Cappello's thesis regards only girder bridges, this Section reports the discussion on this typology. Therefore, the optimization of alpha thresholds proposed in this Section is found for girder bridges although the methodology is valid also for arch bridges. The optimization of parameters will be applied to the APT bridge stock in Chapter 6.

The aim is to obtain the values of the possible actions $a_{a,(i)}$, $a_{n,(i)}$, and $a_{r,(i)}$ which minimize the expected loss. To do this, it is firstly necessary to define the distributions for each stochastic variable $(\Delta\alpha)_{(i)}$ that represents the decrease in the lack of capacity shown after selected assessment level. According to Section 5.4 a lognormal distribution function is used and the mean values that correspond to a probability of 50% that the verification becomes positive are reported in Table 26. Secondly, the loss function should be defined and two loss functions have been chosen to model the perception loss of the decision maker. The loss functions chosen are reported in the following:

- Bernoulli's loss function;
- probability of failure as an outcome.

In Bernoulli's loss function the utility function proposed is as follows (78):

$$u(z) = log\left(\frac{z^c}{z_0}\right), \tag{78}$$

that corresponds to the following loss function (79):

$$l_u(z) = -u(z), \tag{79}$$

remembering that (80):

$$l(z) = l_u(z) - \min_{a\in A}[l_u(z)] \tag{80}$$

where z is money, z_0 is the maximum amount of money the APT is willing to spend and c is a coefficient to calibrate based on boundary conditions (see Cappello, 2013).

In the case of probability of failure as an outcome, the hypothesis is that the decision maker of the APT cannot accept a bridge collapse. In this case the loss is $+\infty$ if p_f=1, which is the worst possible condition. Consequently, this means that the probability of failure can be considered as an outcome. The expected loss is reported in the following (81):

$$L(a_a, \alpha) = c \cdot log\left(\frac{1}{1 - p_f(\alpha)}\right) \tag{81}$$

where c is a constant in millions of euro.

The costs of Table 25 were recalibrated after the effective assignments of the assessment levels to the evaluators (see Chapter 6). The new costs are reported in Table 30.

Table 30: Costs for Level 1 + Level 2 assessment for girder bridges.

Item	**Cost**
Level 1: First inspection with geometric survey, report	0.08%Cc >750 €
Additional cost for Level 2 assessment	+0.12%Cc
Level 1 + Level 2	0.20%Cc >750 €

Table 31: Costs for Level 3 assessment for girder bridges with design documentation

Item	**Cost**
First inspection with geometric survey, report	0.05%Cc
Sampling and tests on the materials involved	0.30%Cc
Integration of the calculation report	0.15%Cc
Total	**0.50%Cc**

Table 32: Costs for Level 3 assessment for girder bridges without design documentation

Item	**Cost**
Complete and detailed geometry survey	0.30%Cc
Sampling of slab	0.15%Cc
Inspection of the reinforcement bars	0.30%Cc
Sampling of the concrete of the girders	0.45%Cc

Sampling of the steel strands of the girders	0.60%Cc
Integration of the calculation report	0.50%Cc
Total	**2.30%Cc**

The cost for Level 0 assessment is equal to zero as the analysis is done automatically using information contained in the APT-BMS database (see Chapter 6).

The results of the optimization of alpha thresholds for a bridge with design documentation, performed by Cappello (2013), are reported in Table 33. According to the APT (see Chapter 6) Cappello (2013) fixed the value of $a_{a,(0)}$ at $a_{a,(0)}$ =0.04.

Table 33: Results of alpha thresholds for decision tree for girder bridges with documentation

Alpha thresholds	Bernoulli's loss function	Probability of failure as an outcome
$a_{a,(0)}$	0.01	0.01
$a_{n,(0)}$	0.55	0.28
$a_{a,(1)}$	0.23	0.17
$a_{a,(2)}$	0.29	0.19
$a_{n,(2)}$	0.55	0.28
$a_{a,(3)}$	0.52	0.25

For a bridge without design documentation the results are shown in Table 34:

Table 34: Results of alpha thresholds for decision tree for girder bridges without documentation

Alpha thresholds	Bernoulli's loss function	Probability of failure as an outcome
$a_{a,(0)}$	0.04	0.04
$a_{n,(0)}$	1.07	1.00
$a_{a,(3)}$	0.48	0.33

The total costs obtained when applying the decision trees defined in Section 5.4 are equal to € 29,200,000 (Cappello (2013)). These costs also consider the re-construction cost of the bridge when, after the re-assessment, the verification is negative.

5.6 Bayesian updating of parameters

Bayesian statistics helps to solve problems that involves inductive logic (Fig. 47), which is where the cause can be determined by observing its effects (Cappello, 2013). For instance, it provides the probability that a fair coin is used if, after ten tosses, the number of heads is six (Sivia, 2006). In contrast, deductive logic predicts only the effects of the cause. Fig. 47 represents the difference between inductive and deductive logic.

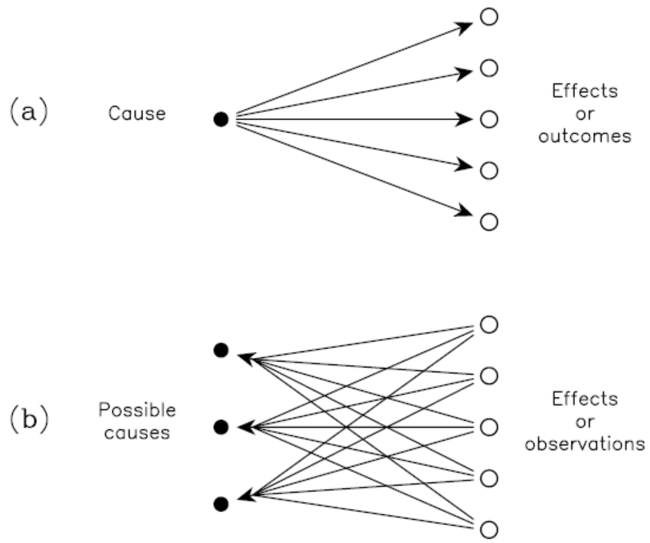


Fig. 47: A schematic representation of deductive logic, and inductive logic (Sivia, 2006).

Bayesian statistics is able to update prior distribution of parameters on the basis of new observations (Bolstad, 2007). In our case the use of Bayesian statistics is useful in order to update the distributions of the stochastic variables $\Delta\alpha$ using the results of a number of case studies.

The Bayes' theorem (85), which is the single tool of Bayesian statistics (Bolstad, 2007), can be obtained as follows:

$$Pr(x, y) = Pr(x|y) \cdot Pr(y) \qquad (82)$$

where $Pr(x,y)$ is the probability of observing both x and y, $Pr(y)$ is the probability of observing y, and $Pr(x|y)$ is the probability of observing x given y. Since (83)

$$Pr(x, y) = Pr(y, x) \qquad (83)$$

the following equations can be written (84) (85):

$$Pr(x|y) \cdot Pr(y) = Pr(y|x) \cdot Pr(x) \tag{84}$$

$$Pr(x|y) = \frac{Pr(y|x) \cdot Pr(x)}{Pr(y)} \tag{85}$$

where $Pr(y/x)$ is the likelihood probability function, $Pr(x)$ is the prior probability function, $Pr(y)$ is the evidence and $Pr(x/y)$ is the posterior probability function.
As reported in Section 5.4 a lognormal distribution function has been used for each stochastic variable $(\Delta\alpha)_{(i)}$. This means that we can write the associated normal distribution function with mean $\mu_{ln(\Delta\alpha),i}=ln[(\Delta\alpha)_{(i)}]$ and standard deviation $\sigma_{ln(\Delta\alpha)i}$=0.40 as follows (86):

$$pdf\left[(\Delta\alpha)_{(i)}\right] = \frac{1}{\sqrt{2\pi\sigma^2_{ln(\Delta\alpha),i}}} exp\left\{-\frac{1}{2}\left[\frac{ln(\Delta\alpha)_{(i)} - \mu_{ln(\Delta\alpha),i}}{\sigma^2_{ln(\Delta\alpha),i}}\right]^2\right\} \tag{86}$$

where $\sigma_{ln(\Delta\alpha)i}$=0.40 results from β =0.7 as imposed in Section 5.4. Equation (86) represents the a priori distribution of $(\Delta\alpha)_{(i)}$. According to Section 5.4 the standard deviation $\sigma_{ln(\Delta\alpha)i}$ is the same for all assessment levels, while the mean $\mu_{ln(\Delta\alpha),i}$ is reported in Table 35.

Table 35: Mean decrease from one assessment level to another.

Assessment level	$\boldsymbol{\Delta\alpha_{mean}}$ ($\boldsymbol{\Delta\alpha_i}$)	$\mu_{ln(\Delta\alpha),i}$
Level 1	0.08	-2.526
Level 2	0.04	-3.219
Level 3	0.08	-2.526

If the bridge is without design documentation (Level 3 assessment is carried out after Level 0 assessment), equation (86) is still valid but in this case the mean value of $\mu_{ln(\Delta\alpha),3}$ is equal to the sum of $\mu_{ln(\Delta\alpha),i}$, which is $\mu_{ln(\Delta\alpha),3}=ln[0.08+0.04+0.08]= ln[0.20]$. The distributions of $(\Delta\alpha)_{(i)}$ are shown in Fig. 48.

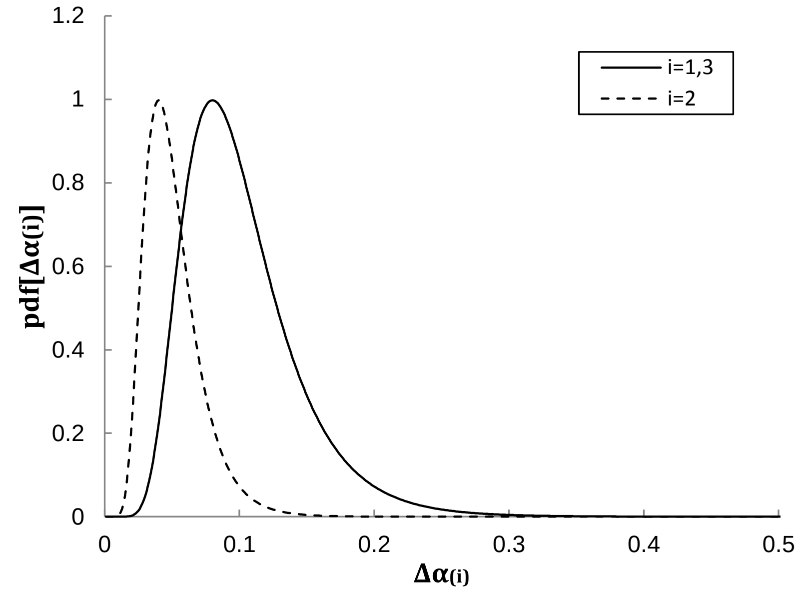


Fig. 48: Distributions of $(\Delta\alpha)_{(i)}$

The Bayes' theorem (85) can be rewritten as follows (Cappello, 2013) (87):

$$g(\mu^*_{ln(\Delta\alpha),i}, \sigma^*_{ln(\Delta\alpha),i}|y) = f_l \cdot f_p \tag{87}$$

where g is the posterior distribution without the constant $Pr(y)$, f_p is the prior distribution, and f_l is the likelihood distribution defined as follows (Cappello, 2013) (88):

$$f_l = \prod_{k=1}^{N} normpdf\left(ln\left[(\Delta\alpha)_{(i)}\right]_k | \mu^*_{ln(\Delta\alpha),i}, \sigma^*_{ln(\Delta\alpha),i}\right) \tag{88}$$

where equation (89)

$$normpdf\left(ln\left[(\Delta\alpha)_{(i)}\right]_k | \mu^*_{ln(\Delta\alpha),i}, \sigma^*_{ln(\Delta\alpha),i}\right) \tag{89}$$

provides the probability density of a normal distribution with mean $\mu^*_{ln(\Delta\alpha),i}$ and standard deviation $\sigma^*_{ln(\Delta\alpha),i}$ at the point $ln[(\Delta\alpha)_{(i)}]_k$.
The prior distribution according to equation (86) is reported in the following (90):

$$f_p = normpdf\left(\mu^*_{ln(\Delta\alpha),i} \middle| \mu_{ln(\Delta\alpha),i}, CoV_\mu \cdot \left|\mu_{ln(\Delta\alpha),i}\right|\right) \cdot normpdf\left(ln\left(\sigma^*_{ln(\Delta\alpha),i}\right) \middle| ln\left(\sigma_{ln(\Delta\alpha),i}\right), CoV_{ln\sigma} \cdot \left|ln\left(\sigma_{ln(\Delta\alpha),i}\right)\right|\right) \quad (90)$$

where the values of $\mu_{ln(\Delta\alpha),i}$ are presented in Table 26 and $\sigma_{ln(\Delta\alpha),i}$=0.4. As can be noted, f_p is a prior distribution of the parameters. The prior distribution is obtained by multiplying two normal distributions as $\mu^*_{ln(\Delta\alpha),i}$ and $\sigma^*_{ln(\Delta\alpha),i}$ are independent (Cappello, 2013). Cappello (2013) imposed the coefficients of variation as follows (91) (92):

$$CoV_{\mu^*} = 0.70 \quad (91)$$

$$CoV_{ln\sigma^*} = 0.80 \quad (92)$$

which represent respectively the coefficient of variation of the mean and the coefficient of variation of the standard deviation of the prior distribution.
Solving equation (87) with a Markov Chain Monte Carlo (MCMC) method, it is possible to obtain the updated values of $\Delta\alpha$ ($\mu^*_{ln(\Delta\alpha),i}$) and β ($\sigma^*_{ln(\Delta\alpha),i}$). MCMC methods are numerical methods developed to obtain samples from the posterior distribution. The Metropolis-Hastings algorithm, which will be used in Section 6.8 for the updating of parameters, is an MCMC method. The steps of Metropolis-Hastings algorithm are reported below (Bolstad, 2010):

1) start with an initial value $\theta^{(0)}$;
2) do from n=1 to n:
    - draw θ' from $q(\theta^{(n-1)},\theta')$;
    - calculate the probability $\alpha(\theta^{(n-1)},\theta')$;
    - draw u from $U(0,1)$;
    - if u< $\alpha(\theta^{(n-1)},\theta')$ then let $\theta^{(n)}= \theta'$, or else let $\theta^{(n)}= \theta^{(n-1)}$.

where $q(\theta,\theta')$ is a candidate distribution that generates a candidate θ' given a starting value θ, $U(0,1)$ is a uniform distribution, and $\alpha(\theta^{(n-1)},\theta')$ is the acceptance probability that is equal to (93):

$$\alpha(\theta,\theta') = min\left\{1, \frac{g(\theta'|y)q(\theta',\theta)}{g(\theta|y)q(\theta,\theta')}\right\} \tag{93}$$

where $g(\theta/y)$ is the posterior distribution.

5.7 Conclusions

The aim of this Chapter was to define a decisional model to support the manager in his/her choices. Different decision trees have been defined for arch and girder bridges with and without design documentation. The separation of decision trees is necessary due to the different costs of the assessment levels with or without design documentation and the different assessment methodologies for arch and girder bridges. Furthermore, a methodology has been defined to determine the limit values of the lack of capacity (α) that make it more cost effective to employ a certain assessment.

In addition, a methodology has been proposed to optimize these thresholds of the lack of capacity α, and a method to update these parameters when a number of case studies are available. These methodologies will be applied to the APT bridge stock in Chapter 6.

6 APPLICATION TO THE APT BRIDGE STOCK

6.1 Introduction

As discussed in Chapter 2, transportation agencies do not know how to respond to transit applications on their bridges. In particular, The Department of Transportation of the Autonomous Province of Trento adopts a conservative procedure to release permits based on empirical information from past crossings of loads that caused no apparent damage to the bridges. Chapter 3 illustrated a methodology that takes into account the real capacity of the bridges. Moreover a decisional model to support the manager in his/her choices was defined.

In this Chapter, I apply both the procedure defined in Chapter 3 and the decisional model shown in Chapter 5 to the APT bridge stock.

In Section 6.2 I report a brief introduction to the APT's Bridge Management System (APT-BMS) while in Sections 6.3 I report the strategic objectives to assess/retrofit the APT bridge stock to allow free or restricted travel of minimum overweight load vehicles over the entire network. Then, in Section 6.4 the consequent results obtained by applying Level 0 assessment are shown. Section 6.5 shows the final decision tree adopted by APT in order to guide their choices. Based on this decision tree a prediction of re-assessment results and costs is performed in Section 6.6.

In the last Sections the optimization of the thresholds of the lack of capacity (Section 6.8) based on the results of 20 APT bridge case studies (Section 6.7) is performed.

6.2 Bridge Management System

Since 1998, following a policy of decentralization that has relocated state responsibilities to the provinces, the number of bridges managed by the

Department of Transportation of the Autonomous Province of Trento, a medium-sized public agency, has increased from 412 to approximately 1000 (Bortot et al., 2009).

As discussed in Section 1.1, in order to allow effective planning of maintenance policies and to meet the new requirements, the Department of Transportation of the APT has begun to collaborate with our Department of the University of Trento (DIMS), to develop a comprehensive Bridge Management System (APT-BMS) (Bortot et al., 2009).

The objective was to develop a management tool which was able to determine present and future needs for maintenance, reinforcement and replacement of APT bridges (Zonta et al., 2007). Moreover the aim was to provide a prioritization system to guide the APT in the effective utilization of available funds. The result has been the development of a management system fully available on the web (www.bms.provincia.tn.it). Fig. 49 shows the main components and the information flow of the APT-BMS. The system is based on a set of components: database, cost, deterioration, repair and maintenance models, inspection data, and decision making. Each component consists of a procedure package and is divided into modules. The modules can operate on a project level (single bridge) or network level (bridge stock) (Zonta et al., 2007).

The database consists of:

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

Inventory data includes all the information and a simplified model of the bridge, representing the logical distribution of its elementary units (Zonta et al., 2007).

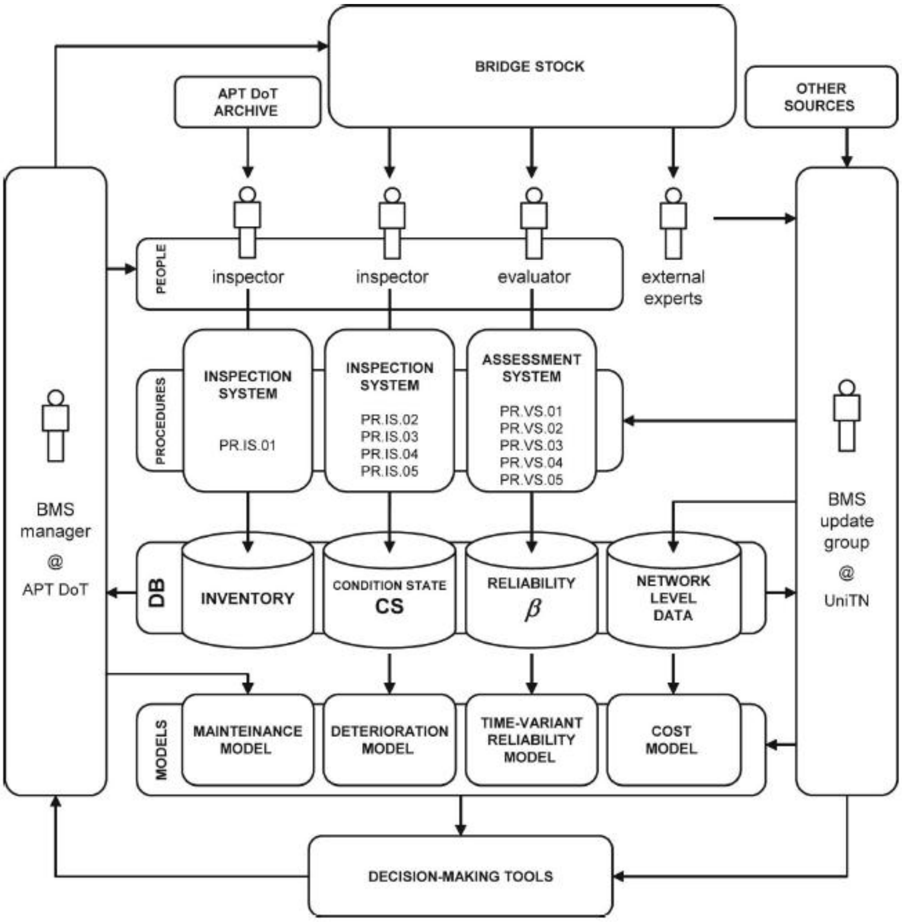


Fig. 49: Main components and information flows of the APT-BMS (Zonta et al., 2007)

The model consists of structural units (SU) which in turn are divided into standard elements (SE), most of which coincides with AASHTO CoRe elements (AASHTO, 1997) (Fig. 50). This division allows the definition of the Condition State evaluated during inspections.

The data of the Condition State provide information about the deterioration of standard elements and their intensity (Bortot et al., 2009). The condition state of the bridge is detected by the evaluators, following the procedures of the APT-BMS. The condition state is represented by an integer. The minimum value, which represents the optimum situation, is equal to 1 while the maximum can vary to 3, 4 or 5 depending on the type of standard element. This methodology was defined in order to maintain the same evaluation models used by one of the most common management systems, PONTIS (Thompson, 1998) (Bortot et al, 2007).

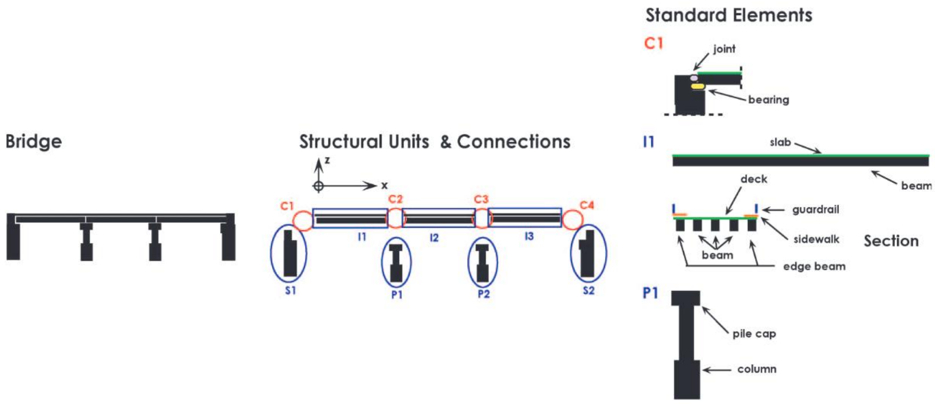


Fig. 50: Example of bridge representation as implemented in the APT-BMS (Zonta et al., 2007)

In addition to this heuristic condition index, another condition index, the *Apparent Age* (AA) index, is introduced in the APT-BMS reflecting the specific view of the agency on the concept of bridge deterioration. The AA index calculates the most likely age of the bridge, given the condition state of its elements. The index, compared to the real age, shows whether the bridge deterioration is normal, or if the structure has suffered abnormal degradation (Zonta et al., 2008).
Moreover, the APT-BMS involves a specific section for the formal assessment of the reliability of bridge structures. 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. Further information on reliability and the APT-BMS is given in Zonta et al., (2007); .
In the APT-BMS there are three types of inspections:

- Inventory inspections;
- Routine inspections;
- Special inspections.

Inventory inspections collect all data required for the operation of the system and if possible include attachments, such as pictures, design documents, FEM models, and AutoCad files, while routine inspections are used to update the condition state of the bridge. Routine inspections are divided into simplified and principal inspections depending on the level of detail and the frequency of the assessment. Special inspections are performed only when, a routine inspection identifies structural anomalies, or the inspector was unable to evaluate one or more elements of the bridge.

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 risk due to a number of unacceptable events in the whole network in a specific timespan (for example: in the next t_L=5 years)* (Zonta et al., 2009).
The ranking of any intervention using a priority index α can be expressed as follows (94):

$$\alpha = \frac{R_{TOT}(t_L)}{C} \tag{94}$$

where $R_{TOT}(t_L)$ is the total risk of unacceptable events over the time span $(0,t_L]$ when the intervention is not undertaken, and C is the cost of bridge reconstruction (Zonta et al., 2009).
The *unacceptable events* considered in the APT-BMS are:

- failure of a principal element;
- failure of a secondary element;
- pile collapse due to scour;
- accidents due to sub-standard guardrails;
- loss of life due to an earthquake.

Assuming that the unacceptable events E_i are non-correlated, the total cumulative-time risk can be evaluated as the sum of risks associated with each event considered (95):

$$R_{TOT}(t_L) = \sum_i R_i(t_L) = \sum_i P_E(t_L) \cdot P_{X|E} \tag{95}$$

where R_i is the risk associated with the unacceptable event E_i, P_E is the *probability of occurrence* of the *unacceptable event* and $P_{X|E}$ is the magnitude of the expected damage if the event occurs.

The system operates entirely on the web, and is based on an SQL database, accessible over the Internet. The database can be modified through a web-based user-friendly interface. Inspectors and evaluators can find the appropriate procedures on the same web application, and can upload the results of condition assessments or of safety evaluations. The manager can view the result of the analysis on the same web-interface. The system is continuously updated by our research group at the University of Trento.

The system is supported by a number of separate servers which are hosted on different machines and can be managed by different actors (Zonta et al., 2006). In its initial configuration the system includes (Fig. 51):

- a database server;
- a web server application;
- the data analysis application server.

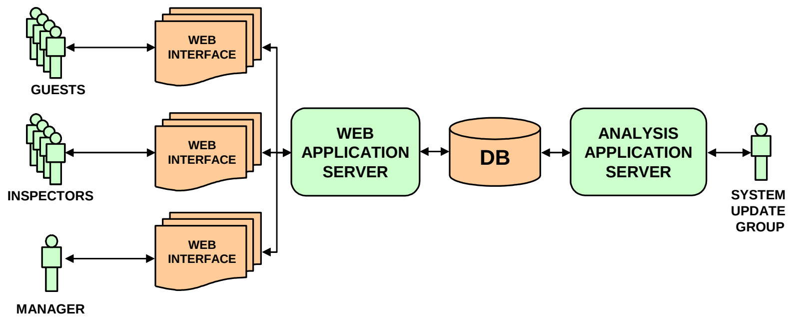


Fig. 51: Architecture of the APT-BMS in the single agency configuration (Zonta et al., 2006).

The web application consists in a user-friendly portal that provides access for data management to users. Users are:

- system managers (full control on the stock information);
- inspectors and evaluators (access limited to bridges and inspections under their responsibility);
- guests.

One of the most interesting aspects of the APT-BMS is its fully web-based operation. The various site sections include: *Inventory*, *Inspections*, *People*, *Roads* and *Network*.
The *Inventory* is the first section of the web application (Fig. 52) and includes a basic search engine, a result list including the important characteristics of the bridges, and a multi-tab window showing information on the selected bridge (Zonta et al., 2006). Inventory data include all the information regarding the bridge, namely identification, administrative issues, geographical location, construction and previous retrofits, and also a number of multimedia attachments (e.g. pictures, documents, FEM models, AutoCAD files). The tab *II livello* shows

the elementary model of the bridge, broken down into Standard Units, and Standard Elements. Moreover, an advanced search engine and a complete report generator are included in the section (Zonta et al., 2006).

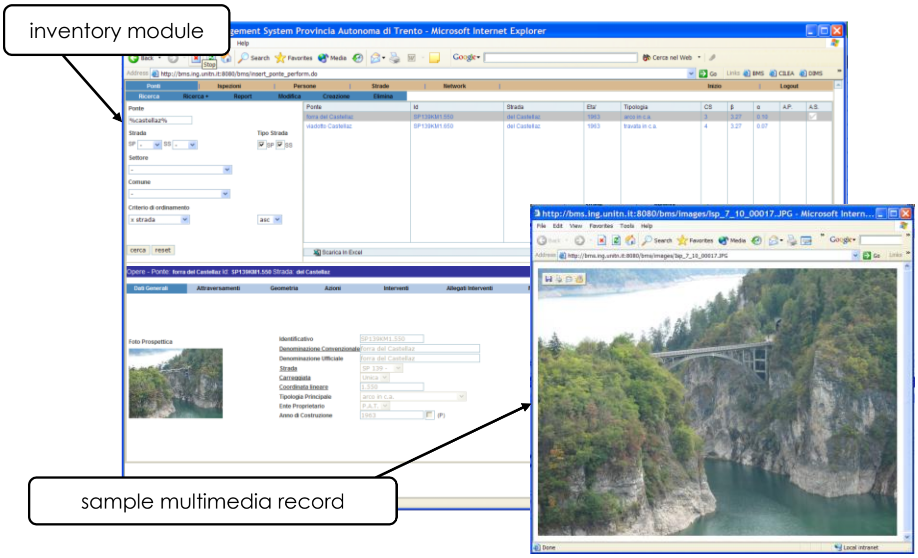


Fig. 52: Display of the Inventory section (Zonta et al., 2006).

The *Inspections* section allows the manager to organize the inventory, routine inspections, and safety evaluations (Fig. 53). The manager assigns tasks to each inspector/evaluator, and validates the results uploaded by them at the end of their job. In the case of an inspection that foresees a condition assessment, the evaluator assigns a condition index to each Standard Element of the bridge. When there are anomalies, the evaluator inserts a detailed description and a number of digital pictures. For safety assessment, the evaluator fills in a table with the results, where each limit state considered in the evaluation has an associated load rating factor or a reliability index (Fig. 54). After entering data, the evaluator should attach a detailed report of the calculations done, including all electronic files used during the analysis (e.g. the input file of a Finite Element program).

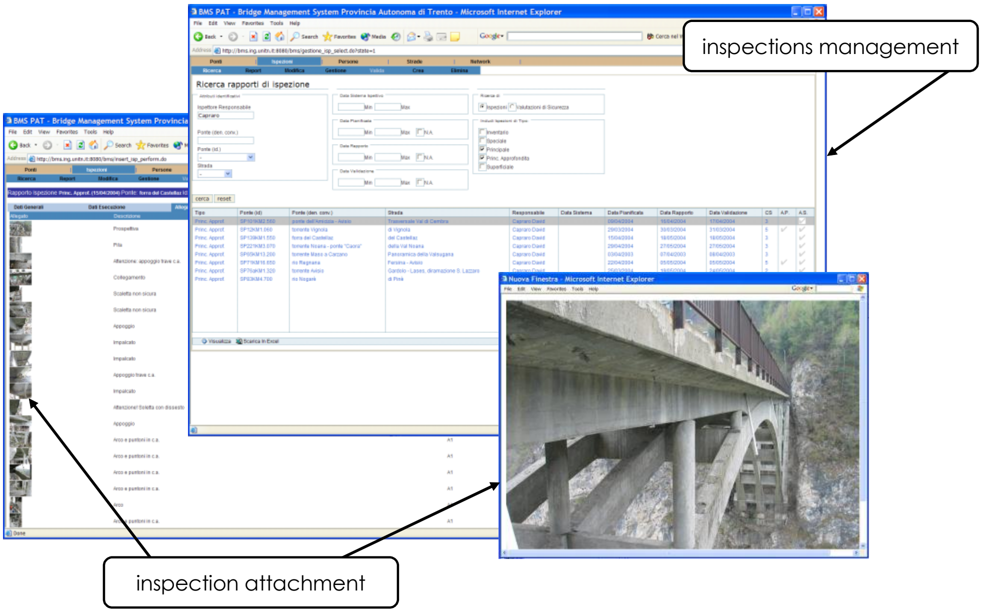


Fig. 53: Display of Inspections section (Zonta et al., 2006).

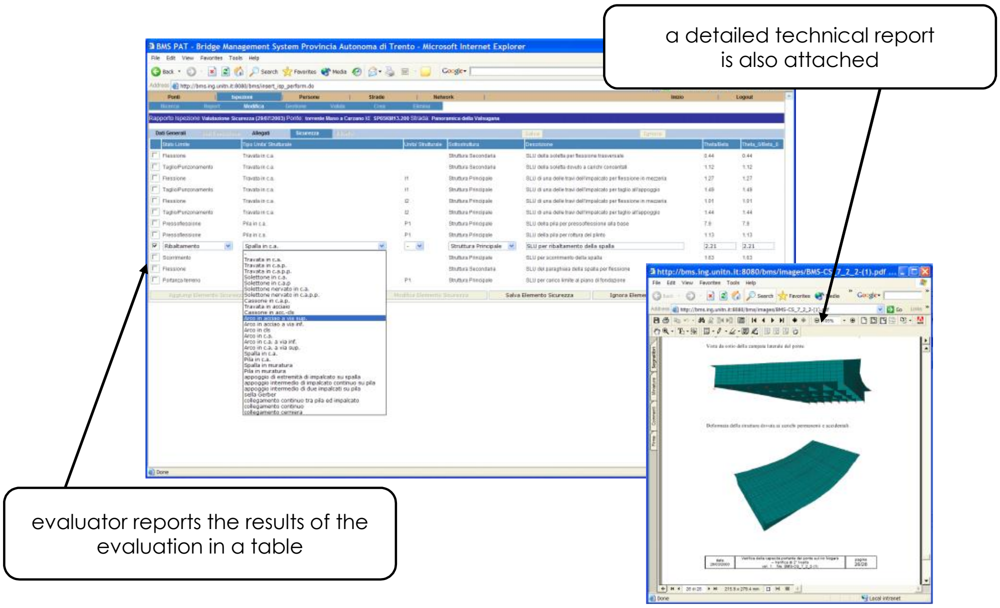


Fig. 54: Display of the evaluations section (Zonta et al., 2006).

In the *People* and *Roads* sections the manager can list the people involved in the process (including the application users), and the APT road network, respectively. In the *Network* section there are the results of the risk analysis of all the unacceptable events of the bridge stock. The users can visualize the results on Google Earth maps.

6.3 Strategic objective

The APT has defined a ten-year objective to assess/retrofit its bridge stock to allow free or restricted travel of minimum overweight load vehicles over the entire network. In particular, the road system has been divided into two different networks: a strategic network and a non-strategic network. The strategic network (see Table 36) includes all inter-regional highway links, with 574 km of highways, 264 girder bridges and 87 arch bridges; the non-strategic network is the remaining 1836 km of local road links, including 428 girder bridges and 174 arch bridges (Fig. 55).

Table 36: Strategic network

Links	Start kilometer	End kilometer
SP59	-	-
SP71	4.000	40.000
SP72	-	-
SP76	-	-
SP79	10.200	13.200
SP80	-	-
SP83	0.000	7.700
SP90 tronco1	3.200	14.000
SP90 tronco dir Ala	-	-
SP90 tronco2	-	-
SP90 dir SS12	-	-
SP90 dir Rover	-	-
SP90 tronco3	5.700	7.510
SP101	-	-
SP104	-	-
SP118	3.700	6.367
SP232	-	-
SP232 dir	-	-
SP235	0.000	17.400
SP254 Rupe	-	-
SS12	326.181	350.300
	357.300	383.000
	387.300	401.275

SS12svincoli	-	-
SS42	147.846	170.900
SS43	0.000	30.650
SS45bis	112.000	116.800
	119.100	154.160
SS47	73.014	130.8 (ex 131.800)
ex SS47 Martignano	-	-
SS48	16.430	20.400
	35.000	63.800
SS48 var Predazzo	-	-
SS50	61.057	74.400
SS237	-	-
SS239	-	-
SS240	0.000	16.800
SS240 dir	-	-
SS249	96.500	101.130
SS347	0.000	11.780

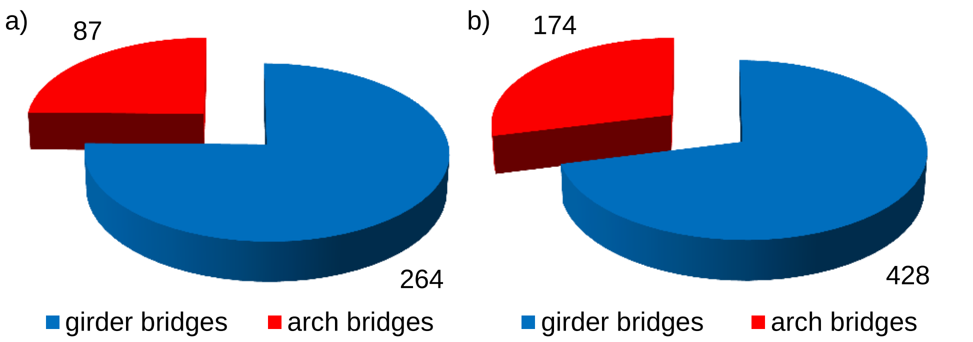


Fig. 55: Girder and arch bridges on a)strategic and b)non-strategic network

The mid-term objective for the strategic network is to allow the free travel of 6-axle overweight vehicles with a maximum axle weight of 12 tons (120 kN) and total weight 72 tons (720 kN), and the restricted travel (bridge crossed in the center of the roadway) of 8-axle overweight vehicles with a maximum axle weight of 13 tons (130 kN) and a total weight of 104 tons (1040 kN). Similarly, for non-strategic bridges, the targets are the free travel of any 56 ton vehicles and the restricted travel of 72 ton vehicles.

Table 37: Strategic objective

	Free travel	Center of roadway
Strategic network	6x12t=72t	8x13t=104t
Non-strategic network	56t	6x12t=72t

6.4 Level 0 assessment results

Level 0 assessment involves the verification of the bridge assuming no overstrength. As already discussed in Chapter 3, in practice the assessment consists in the calculation, for each section of the deck, of the maximum bending moment and shear caused by the design load (or the eccentricity of the thrust line in the case of arch bridges) and the subsequent comparison with the APT loads.
In the case of free travel, the APT-BMS database contains the required data to perform the assessment of each bridge, which are the following:

- maximum span length;
- year of the bridge design (required to infer the design code).

The assessment in the case of crossing the bridge in the center of the roadway would require the Massonet-Guyon-Bares coefficients to evaluate the transverse distribution of the loads. As an alternative, all the geometrical and mechanical details of the deck must be known. Currently, the APT-BMS database does not contain this information although it has been planned to enter the values of the Massonnet-Guyon-Bares coefficients for each bridge in the next two years. For this reason, the transverse distribution of the loads has been calculated using the Courbon method, which requires only the deck width as input data, which is contained in the APT-BMS database.
The analysis of Level 0 assessment was done automatically using the information contained in the APT-BMS database (see Chapter 6). A Matlab® program (Matlab v. 7.0.1) was implemented which automatically reads the geometrical data of each bridge and calculates, in each section of the bridge, the maximum bending moment and the maximum shear stress caused by the design loads and the APT loads. The design loads are calculated by inferring the design code from the year of the design of the bridge. The following design codes are implemented in the program:

- Normale n.8 15/09/1933
- Normale n.6018 09/06/1945 and n.772 12/06/1946
- Circolare n.820 15/03/1952
- Circolare n.384 14/02/1962
- D.M. 02/08/1980
- D.M. 04/05/1990
- D.M. 14/01/2008

For bridges designed before 1933, the program implements the traffic loads included in Jorini (1918), which reports the reference loads to be used in the design of bridges.

Fig. 56 and Fig. 57 show the flowcharts implemented in the Matlab program in the case of free travel and bridge crossed in the center of the carriageway respectively. The details and a comprehensive discussion of the Matlab program can be found in the master's thesis of Lastei (2011).

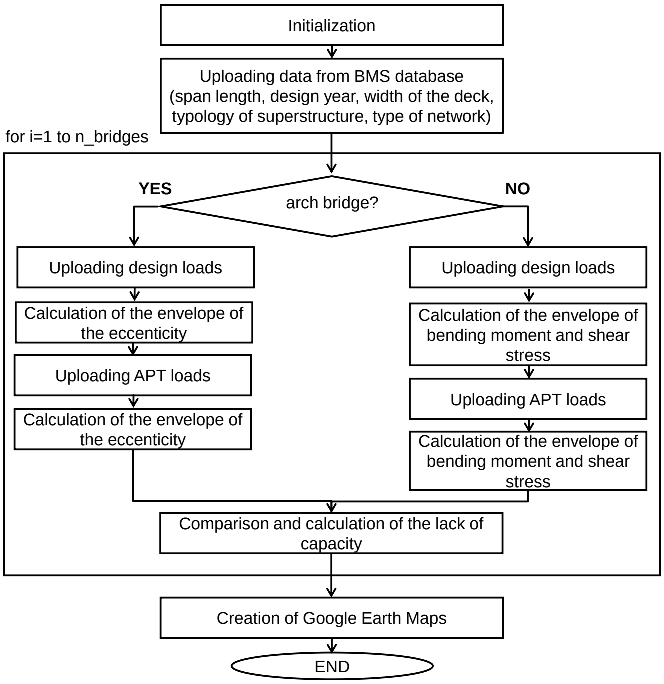


Fig. 56: Flowchart of the Matlab program implemented for Level 0 assessment in the case of free travel

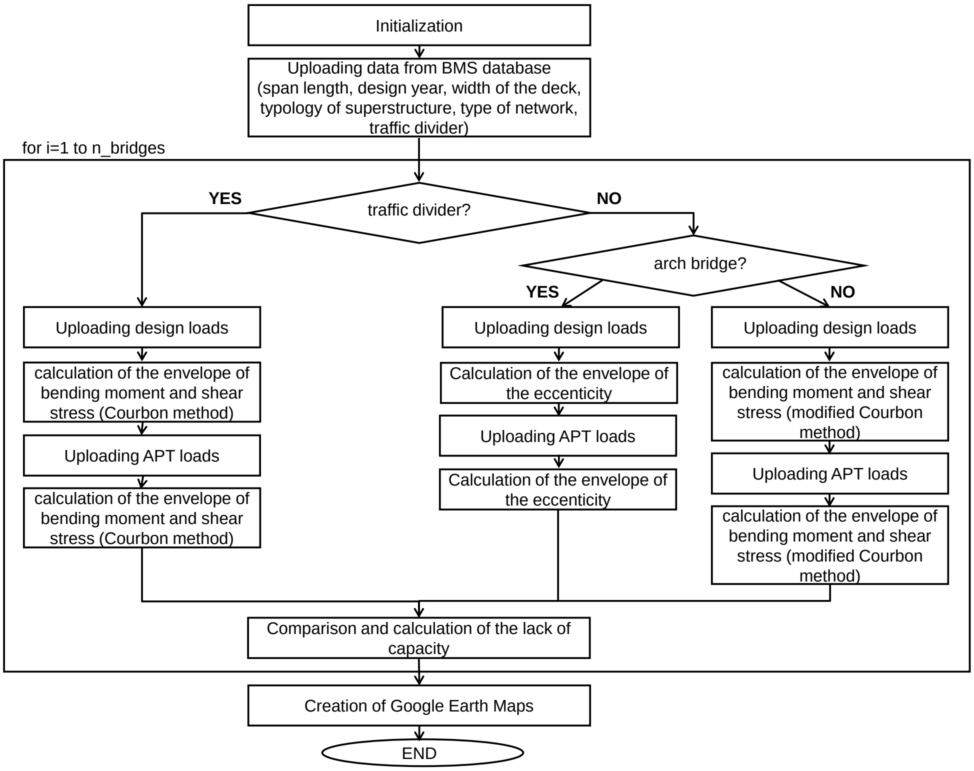


Fig. 57: Flowchart of the Matlab program implemented for Level 0 assessment in the case of bridge crossed in the center of the carriageway

Fig. 58 shows the results of Level 0 assessment using the desired overweight loads for both the strategic and the non-strategic network. Each dot on the map corresponds to one bridge, while the dot color encodes the assessment result in terms of the index α with the following meanings:

Table 38: Lack of capacity and assigned colors

Lack of capacity	Color
$\alpha = 0$	green
$0 < \alpha < 0.1$	yellow
$0.1 < \alpha < 0.2$	orange
$0.2 < \alpha < 0.3$	red
$\alpha > 0.3$	black

Fig. 58a plots the distribution of the lack of capacity in the case of strategic network of the bridges located in the APT, using the higher value of alpha between that relative to free travel (with the 6x12t APT load) and that a relative to bridge crossed in the center of the roadway (with the 8x13t APT load). Fig. 58b shows the results for the non-strategic network with the desired overweight loads.

a) b)

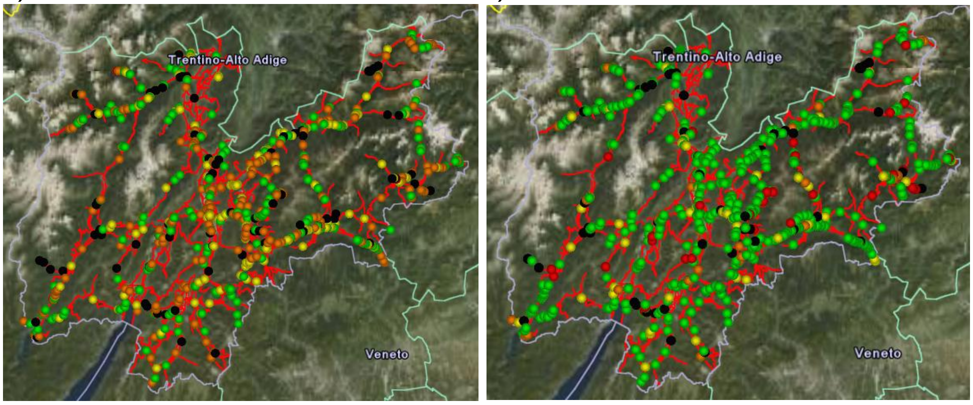


Fig. 58: deficiencies for target APT load models on a)strategic and b)non-strategic networks (maps)

a) b)

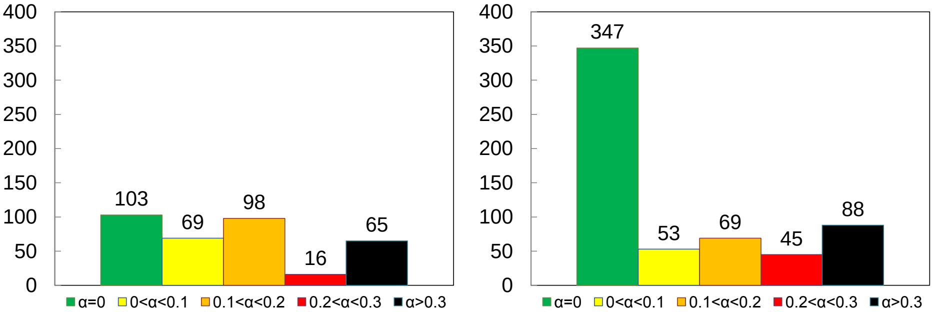


Fig. 59: deficiencies for target APT load models on a)strategic and b)non-strategic networks

The analysis of the strategic bridges shows that the verification is positive for 29% (103/351 bridges) of the bridges of the APT stock. For these bridges, the APT can authorize the transit of overweight loads up to 72 tons in the case of free travel, and up to 104 tons in the case of bridge crossed in the center of the roadway. For non-strategic bridges, Level 0 assessment is positive for 58% (347/602).

The remaining bridges (71% for strategic and 42% for non-strategic network) have been classified as sub-standard, in order to define a priority ranking for specific verification or for future reinforcement or replacement.

The results of assessment levels were inserted in the APT-BMS database through three tables (Table 39, Table 40, and Table 41). In particular, the table

Transitabilità (Table 39) contains the results of all the assessments (Level 0, Level 1, Level 2, Level 3) and from this table the data are uploaded to the web application.

Table 39: Example of the new table Transitabilità implemented in the APT-BMS

Oid	Oid_B	Tip_APT load	Trav_ Cond	α	Tip_Design _code	Ass_L	Lim_state	Tip_Us	Des
1	1	1	1	0	3	0	3	6	-
2	1	2	1	0.23	3	0	4	10	-
3	1	3	1	0.23	3	1	2	5	-
4	1	4	1	0.28	3	1	6	4	-
5	1	5	1	0.34	3	2	7	2	-
6	1	6	1	0.53	3	3	5	1	-
7	1	7	1	0.55	3	3	5	7	-
8	1	1	2	0	3	0	9	6	-
9	1	2	2	0	3	0	5	4	-
10	1	3	2	0	3	1	1	9	-
11	1	4	2	0	3	1	4	11	-
12	1	5	2	0.10	3	1	8	4	-
13	1	6	2	0.15	3	1	3	3	-
14	1	7	2	0.25	3	1	1	2	-

Oid is a progressive number, while Oid_B is the Oid number of the bridge. Tip_APT_load, Trav_Cond, and α are the APT load, the travel condition, and the lack of capacity corresponding to the assessment. Tip_Design_code is the design code used in the evaluation, Ass_L is the assessment level used in the evaluation, and Lim_state is the limit state that caused the highest lack of capacity, Tip_Us is the element where this value of α is calculated, and Des is a detailed description of the verification.

The results of Level 0 assessment are automatically implemented in the database, while for the other assessment levels the evaluator can insert the data directly from the web application.

The results of assessment levels for each bridge are shown in the web application in the new Tab *Transitabilità* (Fig. 60).

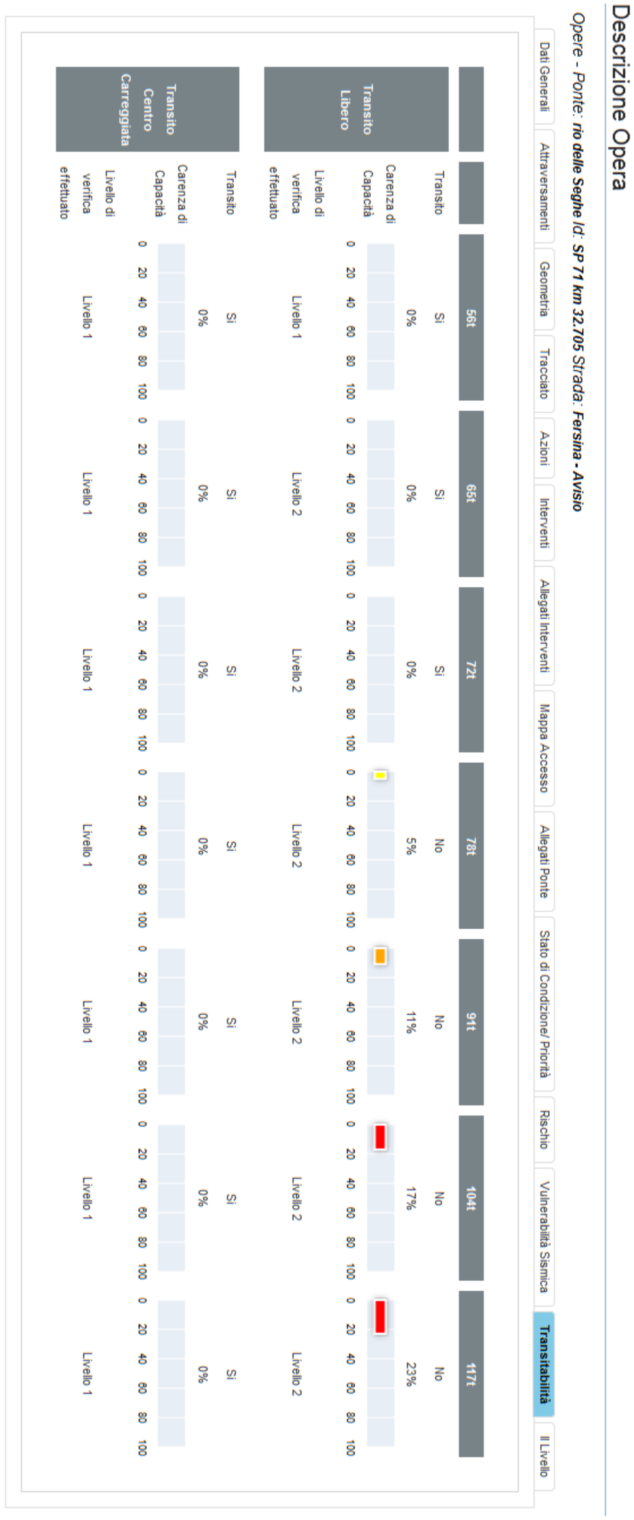


Fig. 60: New tab Transitabilità (rio delle Seghe bridge) (www.bms.provincia.tn.it)

The other two tables were implemented separately from the table *Transitabilità*, in order to ensure that it is possible to add other APT loads or design codes in the future.

Table 40: New table APT loads implemented in the APT-BMS database

Oid	APT_load
1	56t
2	13x5=65t
3	12x6=72t
4	13x6=78t
5	13x7=91t
6	13x8=104t
7	13x9=117t

Table 41: New table Design Code implemented in the APT-BMS database

Oid	Design_code
1	DM_14/01/2008
2	DM_04/05/1990
3	NT_02/08/1980
4	Circ_1962
5	Circ_1952
6	Circ_1945-1946
7	Norm_1933
8	Ante_1933

6.5 Decision tree

In Section 5.4, a general decision tree was defined to guide the APT manager in making the most cost effective decision when selecting the appropriate assessment level and in Section 5.5, the values of α thresholds were found.

The APT decided not to perform any assessment levels if the value of α, after whatever assessment level, is less than 4%. The reason is that even if the verification is negative, the value of α is acceptably small and no further assessment is required. This limit value was obtained by imposing the probability of collapse caused by the APT load on a substandard bridge equal to twice the probability of collapse foreseen by the design code. According to Section 3.6, which shows the risk interpretation of parameter α, the value of the lack of capacity that corresponds to twice the original probability of collapse is equal to equation (97). This equation is obtained from equation (96), whose meaning is explained in Section 3.6.

The APT decided to follow an heuristic method, rather than using the alpha thresholds of Chapter 5. However, the values chosen are partly based on the first example of a decision tree shown in Section 5.4.

$$P_F = 2 \cdot P_{F0} = 2 \cdot 10^{-5} = \Phi(-\beta) = \Phi\left(-\left[\left(1 - \frac{\alpha}{\gamma_{m0} - 1}\right)\beta_0\right]\right) \quad (96)$$

$$\alpha = \left(1 - \frac{-\Phi^{-1}(-P_F)}{\beta_0}\right)(\gamma_{m0} - 1) = \left(1 - \frac{-\Phi^{-1}(-2 \cdot 10^{-5})}{4.26}\right)(2.38 - 1) = 0.04 \quad (97)$$

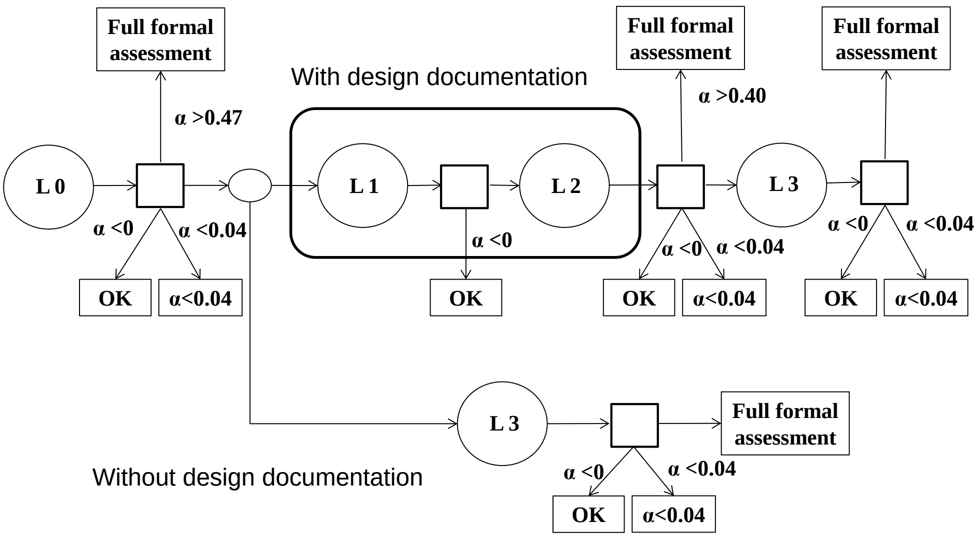


Fig. 61: Final APT decision tree for girder bridges

In the case of arch bridges the final APT decision tree is reported in Fig. 62. In this case the alpha thresholds were also chosen by the APT following an heuristic method.

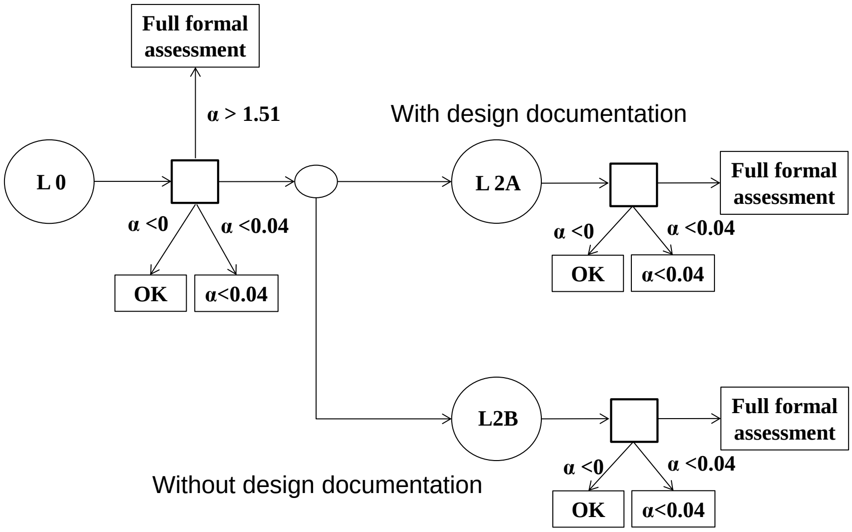


Fig. 62: Final APT decision tree for arch bridges

6.6 Prediction of re-assessment results and costs

The aim of this Section is to predict the results and costs when the decision trees shown in Section 6.5 are applied. The prediction is based on the lack of capacity shown in the results of Level 0 assessment. Table 27 shows the decrease in α from one assessment levels to another.

Fig. 63 outlines the decision-making scheme and shows the prediction of re-assessment results for girder bridges. The results show both strategic and non-strategic bridges. More specifically, after Level 0 analysis, 433 bridges are automatically assessed, including 49 bridges with α<0.04, while 22 others are unlikely to pass any further level of simplified assessment and so must be formally reassessed and possibly strengthened. 139 bridges, out of the remaining 237, have no design documents and must be directly assessed under a Level 3 procedure. The other 98 bridges, with design documents, can undergo the multi-level assessment procedure. After the reassessment, the APT estimates that there will be 72 substandard bridges out of the 692 total bridges. These bridges will be analyzed individually and possibly strengthened to resist higher loads.

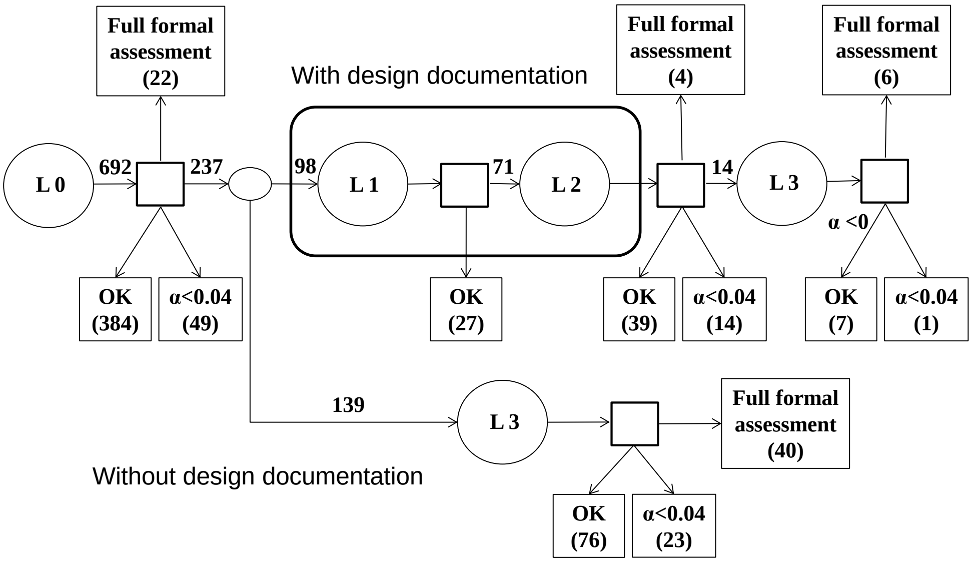


Fig. 63: Anticipated results of re-assessment for girder bridges

Fig. 64 shows the prediction of re-assessment results for arch bridges. As for girder bridges, the results show both the strategic and non-strategic networks.

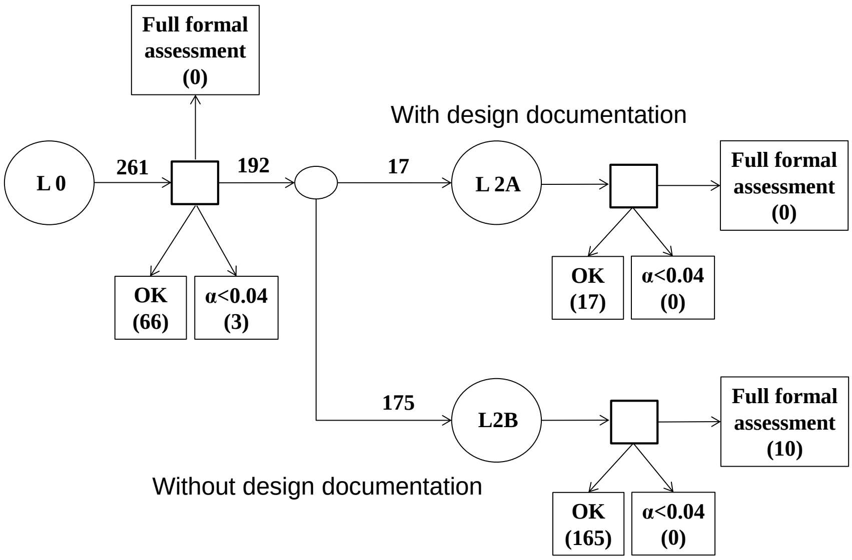


Fig. 64: Anticipated results of re-assessment for arch bridges

After Level 0 analysis, 69 bridges are automatically assessed. Among the remaining 192 bridges, 175 have no design documents; the other 17 bridges, with design documents, can undergo the multi-level assessment procedure. After reassessment, the APT estimates that there will be 10 substandard bridges out of the 261 total.

Table 42 and Table 43 show the total costs for each assessment level for girder and arch bridges respectively (see Chapter 5 for itemized costs). Level 3S indicates the cost of Level 3 whether the design documentation is not available.

Table 42: Total cost of assessment levels for girder bridges

Action	Cost
Level 1	0.08%Cc
Level 1 + Level 2	0.18%Cc
Level 3	0.50%Cc
Level 3S	2.30%Cc

Table 43: Total cost of assessment levels for arch bridges

Action	Cost
Level 2A	0.25%Cc
Level 2B	0.70%Cc

These costs (Table 42 and Table 43) can be applied to estimate the total cost of decision trees. Table 44 and Table 45 show the expected cost obtained by applying decision tree for girder bridges and arch bridges respectively. The total cost for girder bridges is equal to € 2,118,000 while for arch bridges it is € 699,000. It can be noted that there is a high number of bridges without documentation, both for girder (139, which corresponds to 58%) and arch bridges (175, which corresponds to 91%). In addition these numbers are calculated among sub-standard bridges and so the number of bridges without design documentation is higher. The main reason is the policy of decentralization that has relocated state responsibilities to the provinces with a consequent transfer of the design documentation that has not happened yet. For arch bridges, however, the absence of the design documentation is due to the year of the design, which in almost all cases is before the second world war, and in many cases the documentation has been lost.

Table 44: Expected cost applying decision tree for girder bridges

Action	No. bridges	Total (k€)	Total per bridge (k€)
Level 1	27	36	1.33
Level 1 + Level 2	71	105	1.48
Level 3	14	47	3.36
Level 3S	139	1930	13.88
TOTAL		**2118**	

Table 45: Expected cost applying decision tree for arch bridges

Action	No. bridges	Total (k€)	Total per bridge (k€)
Level 2A	17	27	1.59
Level 2B	175	672	3.84
TOTAL		**699**	

Missing design documentation adds extra cost. In fact, the main part of the total cost is due to Level 3S assessment (or Level 2B for arch bridges), which is, for girder bridges, one order of magnitude higher than the cost of assessment levels with design documentation. Although for arch bridges the difference is smaller, the cost per bridge of Level 2B assessment (€ 3,840) is more than double that of Level 2A assessment (€ 1,590). Table 46 and Table 47 show the comparison between costs with and without documentation for girder and arch bridges respectively. These estimated costs were obtained applying the decision tree of Fig. 61 and Fig. 62 supposing that the design documentation of all bridges is available. It can be noted that the saving by owning the design documentation of all bridges would be equal to € 2,097,000 (€ 1,650,000 and € 447,000 for girder and arch bridges respectively). In the case of girder bridges the value of a single design documentation is € 11,800, and for arch bridges is € 2,600. This means that it is worth focusing efforts to look for the design documentation of bridges.

Table 46: Comparison between costs with and without documentation for girder bridges

Girder bridges	
Cost without documentation	2118 k€
Cost with documentation	468 k€
Saving	1650 k€
Number of bridges without documentation	139
Value of documentation	11.8 k€

Table 47: Comparison between costs with and without documentation for arch bridges

Arch bridges	
Cost without documentation	699 k€
Cost with documentation	252 k€
Saving	447 k€
Number of bridges without documentation	175
Value of documentation	2.6 k€

Table 48 illustrates the total costs obtained by adding the costs for arch and girder bridges. As in Section 5.5, these costs also consider the reconstruction cost of the bridge when, after the re-assessment, the verification is negative.

Table 48: Total costs

	Arch bridges	Girder bridges	Total
Re-assessment cost	0.699 M€	2.118 M€	2.817 M€
Reconstruction cost	8.610 M€	61.990 M€	70.6 M€
TOTAL	9.309 M€	64.108 M€	73.417 M€

6.7 Application to 20 APT bridges

In this Section the APT decision trees defined in Section 6.5 are applied to 20 APT bridges, using the same assessment procedure discussed in Chapter 3 and showed in detail in the two case studies *Fiume Adige* and *Rio Cavallo*. The aim is to verify the bridges with the APT loads defined in the strategic objectives of Section 6.3. Therefore, as explained in Section 6.5, Level 2 assessment is applied when the lack of capacity after Level 1 assessment is more than 4%.

Table 49 and Table 50 list, for strategic and non-strategic bridges respectively, the APT substandard bridges selected for the assessments and show the lack of capacity after Level 0 assessment.

Table 49: 8 strategic bridges selected for assessments

Bridge	Identification	Year	Typology	α
rio Secco	SS12 km 364.745	1950	Precast reinforced concrete slab	75%
Rio delle Pile	SP 79 km 12.00	1984	Precast reinforced concrete slab	30%
Rocchetta	SS43 km 23.400 Dx	1950	Concrete arch	27%
rio Ala	SS12 km 337.442	1937	Concrete arch	16%
Fiume Adige	SP 59 km 0.255	1978	Precast reinforced concrete girder	15%
Rio delle Seghe	SP 71 km 32.705	1969	Precast reinforced concrete girder	12%
rio Silla	SP 104 km 1.150	1982	Precast reinforced concrete slab	8%
fiume Sarca	SS 240 km 16.5	1961	Reinforced concrete girder	8%

Table 50: 12 non-strategic bridges selected for assessments

Bridge	Identification	Year	Typology	α
Fiume Adige	SP 21 km 2.895	1943	Reinforced concrete girder	84%
Torrente Maso	SP 65 km 12.89	1966	Reinforced concrete girder	33%
torrente Arione - Aldeno	SP 25 km 0.525	1971	Precast reinforced concrete girder	12%
rio Piazzina	SP 31 km 35.191	1971	Precast reinforced concrete girder	12%
rio delle Stue	SP 31 km 31.63	1972	Precast reinforced concrete girder	11%
rio Val della Setta a Bondone	SP 69 km 9.19	1969	Reinforced concrete girder	11%
rio Lenzi	SP 135 km 11.920	1976	Precast reinforced concrete girder	10%
rio Redebus	SP 8 km 13.580	1973	Precast reinforced concrete girder	10%
Torrente Grigno	SP 78 km 13.310	1972	Precast reinforced concrete girder	10%

rio Loner	SP 89 km 20.420	1977	Precast reinforced concrete slab	10%
torrente Arione-Covelo	SP 25 km 3.49	1975	Precast reinforced concrete girder	10%
Echernachel	SP 8 km 14.320	1973	Precast reinforced concrete girder	10%

Table 51 and Table 52 list the results of assessment for strategic and non-strategic bridges respectively.

In a number of sets of design documentation the verification of piers and abutments was not reported. In these cases it is possible to compare the design and the APT load. This assessment represents an advanced Level 0 as, in contrast with the normal Level 0, the permanent loads are known exactly.

Table 51: Results for strategic bridges

Bridge	**Year**	**Typology**	**α**	**α_new**	**Assessment Level**
rio Secco	1950	Precast reinforced concrete slab	75%	3%	Level 1
Rio delle Pile	1984	Precast reinforced concrete slab	30%	0%	Level 1
Rocchetta	1950	Concrete arch	27%	10%	Level 2
rio Ala	1937	Concrete arch	16%	12%	Level 2
Rio delle Seghe	1969	Precast reinforced concrete girder	12%	0%	Level 2
Fiume Adige	1978	Precast reinforced concrete girder	15%	1%	Level 1
rio Silla	1982	Precast reinforced concrete slab	8%	5%	Level 2
fiume Sarca	1961	Reinforced concrete girder	8%	8%	Level 2

Table 52: Results for non-strategic bridges

Bridge	Year	Typology	α	α_new	Assessment Level
Torrente Maso	1966	Reinforced concrete girder	33%	1%	Level 1
Fiume Adige	1943	Reinforced concrete girder	84%	47%	Level 2
torrente Arione - Aldeno	1971	Precast reinforced concrete girder	12%	5%	Level 1
rio Piazzina	1971	Precast reinforced concrete girder	12%	0%	Level 1
rio delle Stue	1972	Precast reinforced concrete girder	11%	0%	Level 1
rio Val della Setta a Bondone	1969	Reinforced concrete girder	11%	0%	Level 1
rio Lenzi	1976	Precast reinforced concrete girder	10%	0%	Level 1
rio Redebus	1973	Precast reinforced concrete girder	10%	0%	Level 1
Torrente Grigno	1972	Precast reinforced concrete girder	10%	2%	Level 1
rio Loner	1977	Precast reinforced concrete slab	10%	0%	Level 2
torrente Arione-Covelo	1975	Precast reinforced concrete girder	10%	0%	Level 1
Echernachel	1973	Precast reinforced concrete girder	10%	2%	Level 1

The results obtained show that 19/20 (95%) bridges have decreased and only one bridge has maintained the lack of capacity estimated after Level 0 analysis. The mean decrease in α was equal to 12% (16% for strategic network and 9% for non-strategic network). For 7 bridges (Fig. 65), Level 2 assessment was necessary and possible, while for 2 bridges it was not possible to complete Level 1 or Level 2 assessment because of the lack of the design documentation of piers or abutments. For these elements only an advanced Level 0 has been done. In the case of the non-strategic network, 7/12 bridges (58%) are safe, and 10/12 have a lack of capacity less than 4%. For the strategic network, 2/8 bridges (25%) are formally verified and 4/8 (50%) have a lack of capacity that is less than 4%.

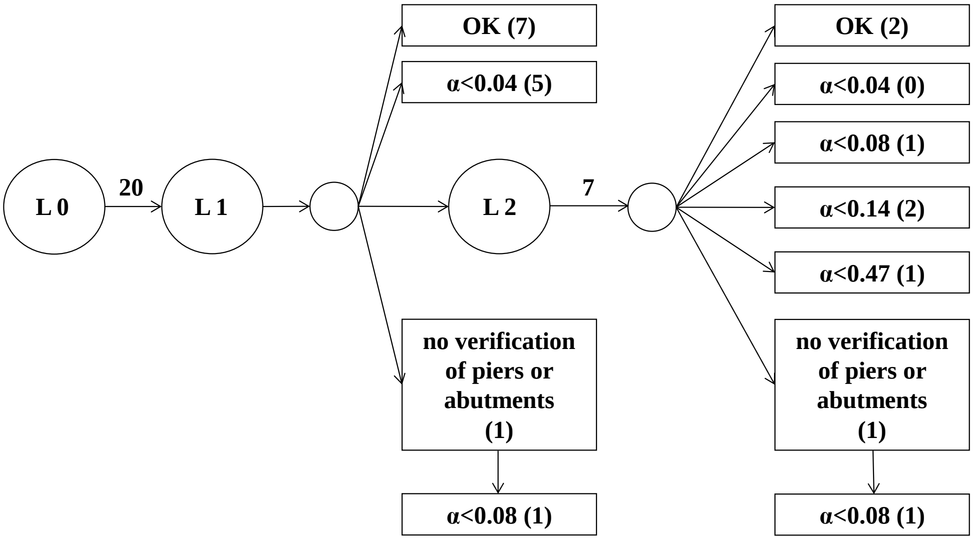


Fig. 65: Results of the application of the decision tree to 20 APT bridges

The results show that:

- with the exception of particular cases, the strategic objectives in the case of the non-strategic network are satisfied;
- in the case of the strategic network the verification of strategic objectives is more difficult and Level 2 assessments are often required;
- in some cases Level 1 and Level 2 assessments are not very useful due to the lack of the design documentation of piers or abutments, which in many cases are the most unfavorable elements.

It is interesting to look at the decrease in the lack of capacity as a function of the design year of the bridge (Fig. 66).

Fig. 66: Decrease in the lack of capacity as a function of the design year of the bridge

As can be noticed, the variability of the decrease in the lack of capacity is higher for bridges designed before the 1960s. For bridges designed from 1960 onwards, the mean decrease in the lack of capacity is 10%.

It is important to point out that in four cases the decrease in the lack of capacity is very large (>30%). In the case of *Rio Secco* bridge (Fig. 67), $\Delta\alpha$ from Level 0 to Level 1 assessment is equal to $\Delta\alpha$=72%. The reason is that the Italian Army Authority (Ispettorato Arma del Genio) imposed design loads (military load) greater than those foreseen by the design code (civil load). The *Rio Secco* bridge was built immediately after the Second World War (1950), and the Italian Army Authority wanted to preserve its strategic networks.

Fig. 67: Rio Secco bridge (www.bms.provincia.tn.it)

As can be noticed in Fig. 67, two signposts placed by NATO at the beginning of the bridge indicate the maximum loads that can cross the bridge in the case of free travel and bridge crossed in the center of the roadway. If there are NATO signposts, we can infer that the design loads are military. In conclusion, in these cases the design load could be higher than that foreseen by the design code and therefore we suggest that those particular cases should be analyzed in depth.

In the other three cases (*Torrente Maso* bridge, *Rio delle Pile* bridge, *Fiume Adige* bridge), the reason for the large decrease in the lack of capacity is that there was some incorrect data within the APT-BMS database. The inventory inspector had classified the bridges as second category, which means the design loads are lower than those effectively used, although they should have been classified as first category.

This demonstrates that the automatic program used to perform Level 0 assessment cannot recognize the rough mistakes contained in the APT-BMS database. This could become a significant problem if the bridge was classified as first category but it was in reality a second category. The solution to this problem could be investigated in future work.

The assessment results have been implemented in the web application of the APT-BMS (www.bms.provincia.tn.it). Fig. 60 shows the summary of the results in the web application (in the *inventory* section) of *rio delle Seghe* bridge for each APT load, for both free travel and bridge crossed in the center of the carriageway.

This table is generated automatically by the web application, summarizing the results inserted in the *inspection* section by the evaluator. In fact, he or she completes a more detailed table that contains the results of all the verifications performed for each transit condition, assessment level, structural element, and limit state. In this manner the APT-BMS manager can quickly check the reason for any lack of capacity summarized in the *inventory* section. In the APT-BMS application can also be downloaded the assessment report of the evaluator.

It is important to highlight that even though the procedure requires only the strategic objectives to be verified, the other APT loads should also be analyzed. This means that in the same assessment level, the evaluator must analyze each APT load, while the decision to apply the subsequent assessment level depends only on the result of the APT load chosen in the strategic objectives. This is because the cost of the assessment level is independent of the number of loads to verify. The APT could authorize, in the case of positive verification, the crossing of overweight loads higher than those defined in the strategic objectives.

6.8 Bayesian updating and thresholds of α optimization

In Section 6.7, 20 APT bridges are analyzed and the decrease in the lack of capacity from one assessment level to another is obtained. In this Section I use these data to update the distribution of $(\Delta\alpha)_{(i)}$ (see equation (86)) in order to optimize the alpha thresholds (Section 6.8) to minimize the expected cost. Only the results concerning girder bridges are used to update the values, as only the thresholds for girder bridges are shown in the following. The methodology has been already described in Section 5.6.

Table 53 shows the mean and the standard deviation of $(\Delta\alpha)_{(i)}$ for each assessment level after the evaluations.

Table 53: Mean and standard deviation of $(\Delta\alpha)_{(i)}$ for each assessment level after the evaluations

Assessment Level	$\boldsymbol{\Delta\alpha_{mean}\,(\Delta\alpha_i)}$	$\boldsymbol{\sigma(\Delta\alpha_i)}$
Level 1	0.16	0.17
Level 2	0.045	0.059

Applying the Bayes updating (87) the following results are obtained (Fig. 68 and Table 54).

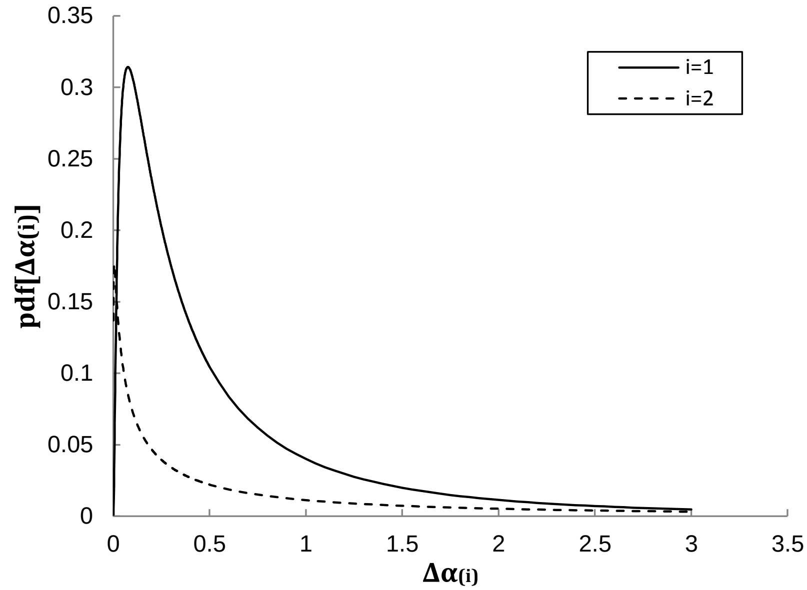


Fig. 68: Updated distributions of $(\Delta\alpha)_{(i)}$

Table 54: Updated distributions of $(\Delta\alpha)_{(i)}$

Assessment Level	$\Delta\alpha_{mean}$ $(\Delta\alpha_i)$	$\sigma_{ln(\Delta\alpha)i}$
Level 1	0.076	1.27
Level 2	0.005	2.26

As can be noticed, the updated value of the mean in the case of Level 1 assessment is close to that *a priori* (which was equal to 0.08), while for Level 2 assessment it is less, (0.04). In contrast, the updated values of standard deviation $\sigma_{ln(\Delta\alpha)i}$ are both larger than those *a priori*. This means that the variability of $\Delta\alpha_i$ is large and consequently the probability of obtaining a large decrease is also high. The shape of the lognormal distribution is wider for values greater than the mean value. In conclusion, the updated mean values are smaller than those *a priori* (in particular for Level 2 assessment), but the standard deviations are larger and so we can expect a larger decrease in the lack of capacity when an assessment level is performed.

Applying the same methodology and costs already shown in Section 5.5, the results shown in Table 55 are obtained.

Table 55: Results of optimization of alpha thresholds (decision tree for girder bridges with documentation)

Alpha thresholds	Bernoulli's loss function	Probability of failure as an outcome
$\alpha_{a,(0)}$	0.15	0.00
$\alpha_{n,(0)}$	0.55	0.28
$\alpha_{a,(1)}$	0.20	0.16
$\alpha_{a,(2)}$	0.29	0.19
$\alpha_{n,(2)}$	0.55	0.28
$\alpha_{a,(3)}$	0.52	0.25

The results are quite different using Bernoulli's loss function or the probability of failure as an outcome. The reason is that the loss functions are very different from each other. As can be noticed, in both cases the thresholds $\alpha_{a,(i)}$ increase during the transition to the following assessment level, while $\alpha_{a,(i)}$ is constant.

Fig. 69 shows the decision tree for girder bridges with documentation and the results of optimization of alpha thresholds using Bernoulli's loss function.

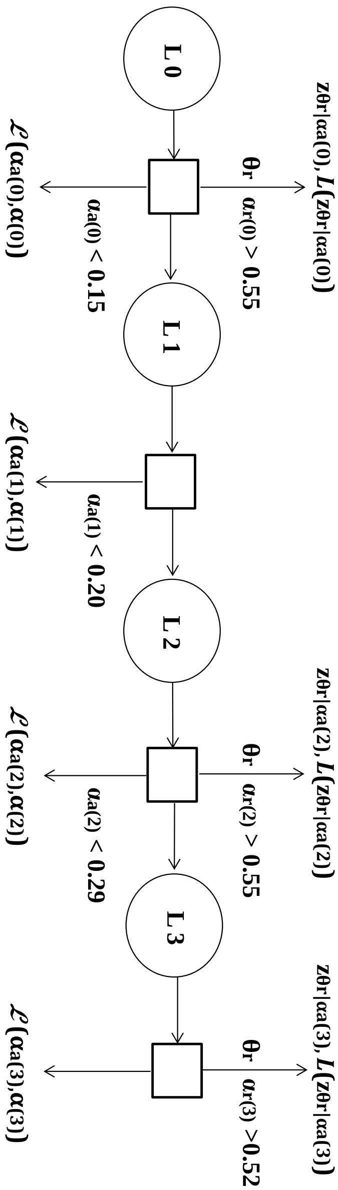


Fig. 69: Results of optimization of alpha thresholds using Bernoulli's loss function (decision tree for girder bridges with documentation)

6.9 Conclusions

In this Chapter I apply the decisional model to the APT bridge stock after having described the strategic objectives of the APT. The mid-term APT objective is to assess in the next 10 years: the strategic road network for 72 ton free travel vehicles, and 104 ton restricted travel vehicles; and the non-strategic network for 56 ton free travel vehicles, and 72 ton restricted travel vehicles. The final decision tree adopted by the APT has been defined for reassessing existing bridges using a multi-level verification procedure. Based on Level 0 analysis, of the 953 bridges in the stock, 502 are automatically acceptable, while 429 need further re-assessment before being formally acceptable. To re-assess substandard bridges, the APT has launched a re-assessment program, expecting to find about 82 substandard bridges that need retrofit strengthening. The prediction of costs shows that, with the current design documentation available, the total re-assessment cost is equal to € 2,817,000. If the design documentation of all bridges were available the total re-assessment cost would be equal to € 720,000 with a total saving equal to € 2,097,000. Hence it is very important to endeavor to locate for the design documentation of bridges.

Finally, this Chapter has also described the optimization of the thresholds of the lack of capacity based on the results of 20 APT bridge case studies. The results show that as the number of assessment increases, the thresholds α can be updated in order to optimize the choices of the decision maker. In comparison to the *a priori* distribution, the updated probability distribution function shows that the updated mean values are smaller than those *a priori* (in particular for Level 2 assessment), but the standard deviations are larger.

7 CONCLUSIONS AND FURTHER WORK

7.1 Conclusions

In this thesis I illustrated how the problem of overweight traffic management can guide transportation agencies, focusing on the legal issues entailed by overweight/oversize load permits. More specifically, practical criteria have defined for issuing overweight load permits, based on the definition of a number of reference load models and on two travel conditions (free and restricted traffic). It has been demonstrated that in the case of free road traffic, the transit of the overweight load on the bridge depends only on the maximum span and on the year of bridge design. Furthermore it is completely independent of the mechanical conditions of the bridge. Moreover it has been obtained that when of a road is closed to free traffic and the bridge is crossed in the center of the roadway, transit of the overweight load on the bridge also depends on the transversal load distribution.

A protocol has been defined for reassessing existing bridges using a multi-level verification procedure and, tested with two case studies. The results show that the decrease in terms of the lack of capacity of the bridge can be great. In fact, for both cases, it was equal to 33% in the case of a 9x13t APT load (free travel condition).

I applied utility theory to optimize the expected cost in order to propose a process to guide the APT in making most cost effective decision when selecting the appropriate level of verification. The choices of the decision maker depend on the lack of capacity of the bridge obtained from Level 0 analyses. Different decision trees have been defined for arch and girder bridges with and without design documentation. The separation of decision trees is necessary due to the different costs of the assessment levels with or without design documentation and the different assessment methodologies for arch and girder bridges. In addition, a

methodology has been proposed to optimize these thresholds of the lack of capacity α, along with a method to update these parameters when a number of case studies are available. These methodologies have been applied to the APT bridge stock in accordance with the strategic objectives to assess/retrofit the APT bridge stock. Based on the decision tree proposed and Level 0 analysis 502 of the 953 bridges are automatically acceptable, while 451 need further re-assessment. It is worth emphasizing that the APT is protected against legal liability because the APT authorizes the transit of an overweight load that the bridge designer would have allowed. The prediction of costs shows that, with the current design documentation available, the total cost for reassessment of the APT bridge stock is equal to € 2,817,000 (Table 56). If the maximum expected utility principle was applied to determine the thresholds of the lack of capacity of the decision trees instead of using an heuristic approach, the total saving would be equal to 44,300,000 €, which corresponds to a 60% less.
Moreover, a methodology has been proposed for the optimization of the thresholds of the lack of capacity based on a number of bridge case studies.

Table 56: Comparison between costs with and without documentation

Cost without documentation	2817 k€
Cost with documentation	720 k€
Saving	2097 k€
Number of bridges without documentation	314
Value of documentation	6.7 k€

7.2 Further work

The methodology proposed in this thesis has been applied to the APT stock using information contained in the APT-BMS database. It was necessary to implement the Courbon coefficient to obtain the transversal load distribution required to calculate the lack of capacity in the case of bridge crossed in the center of the carriageway. This was due to the absence of all the geometrical and mechanical details of the deck in the APT-BMS database necessary to calculate the Massonnet-Guyon-Bares coefficients. However, as it is planned to enter the data of the Massonnet-Guyon-Bares coefficients for each bridge in the next two years, it is recommended to re-assess Level 0 analyses using these more accurate coefficients. Although the overall results should remain similar, the lack of capacity of a number of particular cases could change.

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