Assessment of existing bridges: difficulties and challenges following the Italian experience

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Assessment and rehabilitation of existing bridges: difficulties and challenges following the Italian experience

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Abstract

Many Italian bridges were built more than 50 years ago and are subjected to significant phenomena of deterioration that in some cases, especially in presence of hidden structural defects due to design or construction errors, have triggered catastrophic collapses. The current situation would require in-depth analyses to be carried out on all infrastructures to assess the current level of safety, according to in-force national standard, and to define appropriate construction or operational interventions. However, taking account of the large number of infrastructures, the necessity to define the priorities for detailed assessments arises and, this is why the recent Italian 'Guidelines for risk classification and management, safety assessment and structural health monitoring of existing bridges' includes, for the first time in Italy, a stepwise approach with increasing level of detail. In this article the main difficulties and limitations arisen in recent assessments of existing bridges according to these guidelines are discussed. The aim is to highlight either if they can be solved e.g. by improving the guidelines and/or the current codes of practice, or if they should be taken into account when setting the acceptable level of risk and the relevant assessment and rehabilitation requirements.

Keywords

Assessment, Rehabilitation, Existing Structure, Bridge, Risk, Vulnerability, Deterioration, Corrosion, Reinforced Concrete, Prestressing

1 Introduction

Most of existing bridges in Italy were built more than 50 years ago and are mainly reinforced concrete (RC) or prestressed RC structures. According to a first estimate, the total number of Italian bridges is of about 60,000 units. The large number of infrastructures does not allow to perform detailed structural assessment on all bridges, thus a stepwise approach with increasing level of detail is included in the recent Italian 'Guidelines for risk classification and management, safety assessment and structural health monitoring of existing Bridges' [1] (IGB in the following).

In this scenario, ASPI ('AutoStrade Per l'Italia', the major company for the management of Italian Highways) has started a large campaign of assessment and rehabilitation of its bridges (some typical bridges are showed in figure 1). 2,000 bridges with span greater than 10 m are managed by ASPI (see figure 2). Most of them (i.e. 70%) were built in the '60s and '70s and more than 50% was built before 1970 (see figure 3).

Politecnico di Torino is providing support to the activities performed by ASPI, acting as external reviewer for the assessment reports and the rehabilitation projects.

In the following, the main issues that have arisen during these activities are described and commented. The aim is to prompt a paradigm shift in the forthcoming codes of practice to take account a priori of the complexity of the process when setting the acceptable level of risk and the relevant assessment and rehabilitation requirements.

2 The Italian guidelines

2.1 Overview of the guidelines

At the national scale, it is both technically impossible and economically inconvenient to perform detailed investigations of all bridges. The complexity of the investigations, the large number and the variety of these structures make it necessary the adoption of a stepwise approach, from the national scale to the scale of the single bridge. The aim is to select, by rapid inspections, a limited number of bridges that deserve the detailed assessment and keep routine inspections on the other bridges.

Figure 1 Some cases of bridges assessed by ASPI with the external review by Politecnico di Torino.

In this scenario, the stepwise approach given in IGB is based on six different levels of increasing depth and complexity. The first three levels (i.e. 0-2) should be carried out for all bridges and aim to define, for each bridge, a risk indicator called 'attention class' (AC in the following) that combines structural, seismic, geotechnical and hydraulic risks [2].

Level 0 involves the census of the main characteristics of bridges through the collection of available information and documentation.

Level 1 envisages the execution of direct visual inspections and rapid survey aimed to identify the state of deterioration and the main structural and geometric characteristics, as well as potential risk conditions associated with landslides or hydrodynamic actions.

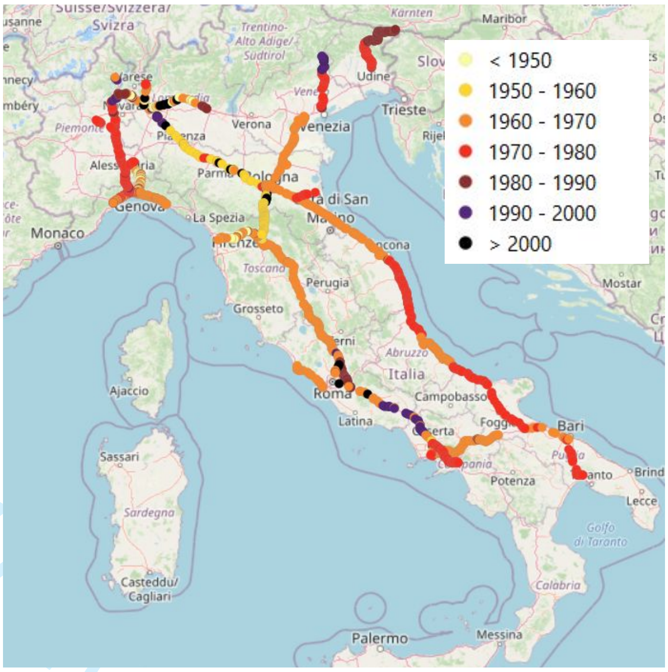

Figure 2 Map of Italy with the indication of bridges (span > 10 m) managed by ASPI and the relevant date of construction.

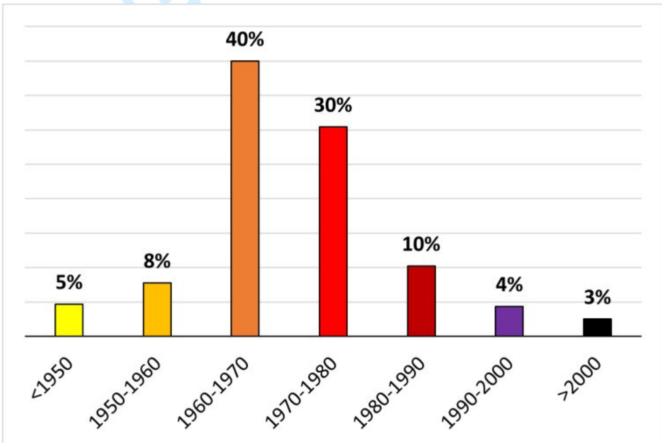

Figure 3 Bridges (span > 10 m) managed by ASPI: percentage of bridges vs. period of construction.

Based on the information gained in Levels 0 and 1, in Level 2 an attention class is associated to each bridge. To this aim five attention classes are defined in IGB (i.e. low, medium-low, medium, medium-high, high).

For all attention classes, routine inspections are recommended whilst for medium-high and high attention classes the bridge management system should include structural health monitoring.

Level 3 should be carried out in case of medium or medium-high attention class. This level includes a preliminary assessment of the bridge that should be performed by the comparison of the traffic load models given in the current code of practice [3] (ITC in the following) with those included in the code in force at the time of construction. The aim of level 3 is to understand if a detailed assessment according to ITC (i.e. Level 4) is needed. Level 4 applies also in case of high attention class.

Level 5 should be carried out only for bridges of significant importance within the road network and requires a specific study for the resilience of bridge network which is not covered by the current version of IGB.

In addition to what above mentioned, according to IGB, in case of post-tensioned RC bridges and for those placed where there is evidence or knowledge of landslides, flooding or erosional phenomena, detailed inspection (called 'special inspection') should be performed to evaluate if there is the need to perform directly Level 4 assessment without completing Level 1.

2.2 Some issues of the guidelines

As above mentioned, the main goal of the IGB stepwise approach is the selection of a limited number of bridges which need a detailed assessment that includes detailed surveys, tests, numerical analyses and verifications. This aims to optimise time and economical resources.

IGB includes a specific risk rating based on performance indicators. On the basis of this rating, at the end of Level 2 it is possible to assign the AC that defines if either preliminary (i.e. level 3) or detailed assessment (i.e. level 4) is needed for the bridge.

The analyses and application of the process included in IGB has highlighted the following issues:

- IGB are oriented towards a classification based on hazard (structural, hydrogeological, seismic), exposure and damage (defectiveness) rather than on the structural performance.
- The definition of AC is based on a deterministic approach without taking into account the semi-probabilistic format included in ITC.
- It is possible that low AC is given to a bridge with very high static vulnerability (e.g. due to the inadequacy of the original design). This situation is unacceptable from an engineering point of view and is the consequence of the low contribution given by the structural performance to the evaluation of AC.
- Maintenance interventions that regard only the element surface (e.g. generalised restoration of concrete cover) lead to an improvement in the appearance of the bridge and therefore could lead to the reduction of the AC, even though the structural performance of the bridge has not substantially changed. This is because the rating of IGB is mainly based on visual inspections.
- Some damages, that in general have minor influence on the increase of vulnerability, have high rating and, thus, would lead to the highest ACs.
- There are some parameters (e.g. geographical location, traffic volume) which have great influence on the classification result and tend to significantly affect the value of AC. Moreover, there are some parameters which are present in more than one risk factor and, thus, significantly influence the final result. For instance, the traffic volume is used to calculate both the hazard and the exposure.
- The longer the bridge span the higher the vulnerability score is. This is not always correct since the application of ITC to the detailed assessment of bridges has shown that in continuous girder bridges shorter spans are more vulnerable.
- Landslide risk rating does not account for the position of the bridge on a landslide. This is worth mentioning since usually differential settlements are higher on the borders of a landslide than on its body thus higher damages are expected on bridges located on the border of a landslide. This imply that in IGB there is an overestimation of landslide risk scoring for bridges located within landslide body.
- Landslide vulnerability scoring is calculated by using the same parameters as for seismic vulnerability scoring. Since the performance of a bridge when subjected to a landslide is completely different from that when subjected to an earthquake, this scoring seems to be unreliable.
- Landslide vulnerability scoring does not account for the depth of the foundation with reference to the depth of the landslide sliding surface. This leads to an overestimation of the scoring if the foundations are deeper than the sliding surface of the landslide.

2.3 Possible improvements to the guidelines

Based on the above-mentioned issues the following improvements should be made to the guidelines:

- The number of ACs should be increased by introducing 'sub-classes'. The primary objective of the bridge rating is to provide indications on the priority of detailed assessment and of the relevant rehabilitation works. If, downstream of the rating, most bridges fall into the highest classes, it is not possible to make priority choices if not based on other parameters. Although IGB provides for the presence of other indicators, the use of sub-classes could be a more direct tool for the owner/concessionaire to define the relevant priorities. For example, the operational guidelines used by ASPI already includes an additional parameter to make a classification of those bridges which according to IGB need Level 4 assessment. This classification is useful to define the relevant priorities.
- Vulnerability rating should also be based on the performance of the bridge and on the rapid evaluation of structural robustness. With reference to the latter, for bridges it is usually very simple to highlight the structural hierarchy and the possible chain of progressive collapse, so that the importance (rating) of defects should also be linked to the overall robustness of the bridge system. This would allow the prioritisation of actions based on quantitative

elements to be evaluated directly, avoiding some of the issues above outlined.

- The same parameter should be included only in a single risk factor.
- Landslide risk rating should be revised accounting for the position of the bridge on the landslide.
- Landslide vulnerability rating should be revised accounting for position of the bridge on the landslide, speed of the landslide, static configuration of the bridge, depth of the foundation, depth of the landslide sliding surface.
- The target reliability level of an existing bridge should be lower than that of a new one taking into account that e.g. the cost of safety measures is higher. This would reduce the number of bridges that need rehabilitation.

3 Lack of original design document

When the original design and construction records are not available or if some random tests performed on the bridge reveal that the original design documents are not reliable there is the need to reconstruct the geometry (including structural details) of structural elements.

This is a key issue especially for RC bridges since it is quite impossible to reconstruct the entire reinforcement layout. To this aim both IGB and the commentary to ITC [4] suggest performing the so-called 'simulated project' (SP in the following) referring to codes and good practice of the construction period. The aim of SP is to reconstruct the reinforcement layout in order to limit destructive tests. The reinforcement layout obtained by SP should be checked by using non-destructive tests and slight destructive tests (i.e. removal of concrete cover). However, SP has the following major limitations:

- Tests to check the reinforcement layout can be made only for reinforcement placed near the edge of the structural element. Thus, verification of SP validity is limited to this kind of reinforcement.
- In Italy the code of practice in force up to 1971 gave very few rules for the design of reinforcement layout and no guidelines were available. Thus, the design of reinforcement layout was very subjective and based on the knowledge and experience of the practitioner. In this scenario, especially for bridges designed before 1971, it is quite impossible to reconstruct the original reinforcement layout since many different options are available to the assessor.

Following the above-mentioned limitations, in case of lack of a reliable original design document it is utopian to assume that a detailed assessment can be performed since reliable information on the geometry is available only for the sections where detailed tests have been made. Thus, it should be accepted that in these cases the assessment should be considered as preliminary since its final result is based on numerical verifications performed only in some specific sections.

4 Knowledge levels and confidence factors

According to ITC, numerical verifications of an existing structure are performed by using the partial factor format and are based on the identification of the relevant Knowledge Level (KL). ITC and its Commentary [4] define three KLs (i.e. KL1, KL2, KL3). The factors determining the appropriate KL are geometry, structural details, materials. The KL achieved determines the values of the Confidence Factor (CF) to be used in the numerical verifications. CFs equal to 1.35, 1.20, 1.00 are associated to confidence factors KL1, KL2, KL3, respectively. The evaluation of CF is needed to determine the assessment value of material strength f_d to be used in the numerical verifications according to the following equation (IGB):

$$f_d = min\left(\frac{f_m}{CF \cdot \gamma_M}; \frac{f_k}{CF}\right) \qquad (1)$$

where:

- f_m and f_k are the mean and the characteristic value of material strength, respectively;
- γ_M is the material partial factor.

There are two main issues related to this approach.

The first is that neither ITC nor its commentary [4] nor IGB include specific rules to evaluate the appropriate KL for bridges (e.g. number of tests). There are some guidelines developed by some Italian organizations that are used by some clients. The review performed by Politecnico di Torino to some assessments has found, in many cases, great subjectiveness in the choice of the appropriate knowledge level. Moreover, some assessors assumed the greatest CF to compensate the lack of tests to determine material characteristics. This is, for example, the case of post-tensioned tendons. Since it is difficult to determine their characteristics and the relevant prestressing action, some assessors usually assume the values included in the original project documents, applying the greatest CF (i.e. 1.35) and without making any test. This is based on the (erroneous) conviction that CF=1.35 is able to conservatively cover the lack of validation by testing.

The second issue is that the values of CF given by the commentary to ITC [4] are taken from the current part 3 of Eurocode 8 [5]; thus, these values are not reliability-based and, according to the Eurocode approach, should be used only for the assessment to seismic actions.

It is the authors' opinion that to avoid the above-mentioned issues, this approach should be revised to be consistent with that to be included in the 2nd generation of the Eurocodes. This new approach will imply the need to update the basic variables and to adjust the partial factors on the basis of both prior information (if available) and of the tests performed on the existing structure.

5 Materials or products not included in ITC

In some existing bridges there are materials or products for which neither ITC nor its commentary [4] include specific rules (e.g. resistance models).

For example, it is very common to find plain bars in RC bridges built before the '70s. ITC and its commentary [4] don't include rules to evaluate the anchorage/lap capacity of these bars thus, in absence of prescriptive rules, it is very common that the assessors don't perform the relevant verifications. This could lead to an overestimation of the element capacity taking into account that in Italy the first rules on anchorage are present in the technical standards starting from the '70s and that the anchorage length usually assumed in the past by the designer is very low if compared with the most recent development in practice and research.

In these cases, to overcome the lack of specific rules in ITC, the most recent formulations included e.g. the 2nd generation of the Eurocodes should be used or specific tests should be performed.

6 Use of classical bending and shear verifications for D-regions

Schlaich and co-authors [6] made distinction between B-regions ('B' stands for beam or Bernoulli) and D-regions ('D' stands for discontinuity, disturbance, detail) in RC structures. In B-regions the Bernoulli's hypothesis of plane strain distribution is assumed valid. Their internal state of stress is easily derived from the sectional forces (bending and torsional moments, shear and axial forces). In D-regions the strain distribution is significantly nonlinear; these regions may be due to static discontinuities (e.g. point loads caused by supports or anchorage zones) or to geometrical discontinuities (e.g. frame corners or openings in members) or combination of both (e.g. corbel with point load at a column). In D-region the classical methods used for bending and shear verifications of beams cannot be adopted and different methods based e.g. on Strut-and-Tie Model (STM in the following) should be used [7].

In RC bridges there are many parts of the structure that are D-regions taking into account either their geometry or their loads (e.g. column footing, pile cap, anchorage zone of prestressing tendons, deck diaphragm, pier cap).

In many cases analysed by Politecnico di Torino, classical shear and bending verifications have been applied also to D-regions. This is because the use of different approaches based e.g. on the STM is not widespread in Italy both in practice and in university courses.

7 Assessment of prestressing action for post-tensioned bridges

Most of Italian RC bridges built before the '80s are post-tensioned.

In the analysed cases, even if specific investigations have been performed to evaluate the geometry of tendons (see e.g. figures 4 and 5), in very few of them tests to assess the relevant material characteristics and the prestressing action have been made. Moreover, tests to investigate tendon corrosion are often limited to very few parts of the bridge (figures 5 and 6).

As highlighted in paragraph 4, there is a common habit to assume the values included in the original design document applying the greatest value of CF, indicated in the commentary to ITC [4], to compensate the lack of knowledge on material characteristics and prestressing action. However, as above-mentioned, this approach is extremely dangerous since the use of CFs is not reliable in absence of tests made to check the validity of information present in the original design documents.

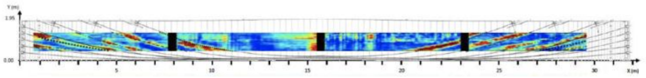

Figure 4 Example of X-ray tomography to assess tendon layout.

Figure 5 Removal of concrete cover to assess tendon layout and relevant deterioration.

Figure 6 Video-endoscopy to check tendon deterioration.

Besides, extensive tests to evaluate tendon corrosion are needed. In fact, it is not possible to extend to the entire element the results of tests performed in very few parts of the element mainly for the following reasons:

- Sections where severe corrosion to tendons is present are difficult to foresee since corrosion mostly occur in sections where internal grouting of tendon ducts is incomplete and moist air, water and contaminants can enter the ducting system.
- When corrosion is relevant to chloride attack it is

very localised (i.e. pitting corrosion) and often without signs on the element surface.

It is worth adding that due to the high prestress on tendons, pitting corrosion process may be accelerated and brittle rupture might occur earlier than expected.

Taking into account what above mentioned, FABRE (i.e. Italian Research Consortium for the assessment and monitoring of bridges, viaduct and other structures) has developed two documents on special inspections on post-tensioned RC bridges according to IGB. These documents give all details to perform non-destructive and destructive tests to evaluate geometry, material characteristics, prestressing action and deterioration of post-tensioned cables by using an approach that is perfectly consistent with IGB. They include a reliability-based approach to define the residual area of corroded tendons accounting for the number of inspected sections, the number of detected defect and the value of CF to be applied according to the commentary to ITC [4]. The documents by FABRE are currently under validation by using specific case studies and, when validated, will represent a useful tool to assess post-tensioned tendons. Moreover, these documents include also a first step devoted to the analysis of available documentation that aims at a preliminary evaluation of tendon corrosion risk. This could be very useful to define the priorities to develop the special inspections requested by IGB.

8 Evaluation of concrete strength characteristic value from in-situ tests and relevant partial factor

IGB gives the following equation to evaluate the characteristic value f_k of a material strength, based on the number *n* of test results:

$$f_k = exp(\overline{\mu}_{0.16} - 1.64\overline{\sigma}) \qquad (2)$$

where:

- $\overline{\mu}_{0.16} = \overline{\mu} - \frac{\overline{\sigma}}{\sqrt{n}}$ (3)
- $\overline{\mu} = \frac{1}{n}\sum_{1=1}^{n} ln(x_i)$ (4)
- $\overline{\sigma} = \sqrt{\frac{1}{n-1}\sum_{i=1}^{n}[ln(x_i) - \overline{\mu}]^2}$ (5)
- x_i is the i-th test result.

Equation (2) assumes that the material strength is log-normally distributed. This equation is general and does not account for the peculiarities of each material when assessing its strength by testing.

In the analysed cases, the assessors, to evaluate concrete strength, have used equation (2) by taking directly as x_i the value of resistance obtained from compressive test on cylindric cores extracted from the structural element. This habit is not correct mainly for the following reasons:

- Formulations included in ITC for concrete refer to f_{ck} (i.e. compressive strength of concrete determined from samples of concrete taken in accordance with EN 12350-1:2019 [8] made into cylinder or cube specimens and cured in accordance with EN 12390-2:2019 [9]) whilst what calculated by practitioners is the in-situ compressive strength of concrete $f_{ck,is}$. Differently from f_{ck}, $f_{ck,is}$ accounts for:
  - (i) damage sustained during core extraction;
  - (ii) effect of curing;
  - (iii) effect of consolidation of fresh concrete (settlement and bleeding) leading to a significant geometric variability within the structure;
  - (iv) anisotropy (e.g. difference between vertical and horizontal strength resulting from bleeding).
- If the cores are extracted from regions not necessarily representing the conditions of whole structural member nor the region governing for the verification, the effect of the position of the cores, within the structural member, should be taken into account when evaluating f_{ck}.

In this scenario a conversion factor $k_{\mu fc}$ is needed to calculate f_{ck} from $f_{ck,is}$:

$$f_{ck} = \frac{f_{ck,is}}{k_{\mu fc}} \qquad (6)$$

Annex I of the last draft of the 2nd generation Eurocode 2 [10] includes the values of $k_{\mu fc}$ (varying from 0.85 to 0.95) which depend on the region where cores are extracted and account for the above-mentioned effects. Taking account of these values, it appears that the habit of Italian assessors leads to an underestimation of f_{ck} from 5% to 18% depending on the case. The fact that this is on the safe side is not sufficient to ignore this issue. In fact, the underestimation of concrete strength could imply strengthening works and/or traffic limitations that will be not needed by applying the conversion factor to get f_{ck} from $f_{ck,is}$.

Another issue to be mentioned is the one relevant to the definition of partial factor for concrete strength.

For concrete, ITC uses the same partial factor value at ultimate limit state (i.e. γ_c = 1.5) given by both the current [7] and the 2nd generation Eurocode 2 [10]. This value is based on the assumption that the coefficient of variation of concrete strength is equal to 0.10, as clarified in Annex A of [10].

It is very common that for old RC structures, this coefficient of variation is significantly greater than 0.10 (in some cases even greater than 0.25) because in the past very few procedures were used to control the quality of concrete. In these cases, keeping γ_c = 1.5 will imply a significant reduction in the relevant reliability level and, thus, in the relevant safety level. In this scenario the relevant adjustment of concrete partial factor should be recommended e.g. by using the procedure given in Annex A of the 2nd generation Eurocode 2 [10].

9 Underestimation of thermal action

Even if ITC gives the values of both annual minimum and annual maximum shade air temperature depending on the zone, neither ITC nor its commentary [4] gives specific rules to evaluate, from these values, the relevant changes to be used when designing or assessing a bridge structure.

This is why many assessors are used to adopting, for thermal change, either the values given for buildings or those frequently used in the past. The problem is that these values are usually very low (in general equal to ± 15° C) if compared to those obtained by using the map of annual minimum and annual maximum shade air temperature, and the procedure given e.g. in section 6 of the current part 1-5 of Eurocode 1 [11]. This is worth noting since the significant underestimation of thermal action could lead the assessor to be unaware of the need to carry out specific interventions e.g. on expansion joints and bearings.

10 Conclusive remarks

This article presents the issues that have arisen in recent assessments of Italian existing bridges performed by ASPI with Politecnico di Torino acting as external reviewer. These activities were performed according to the current Italian code of practice (ITC) and to the recent 'Guidelines for risk classification and management, safety assessment and structural health monitoring of existing Bridges' (IGB).

In the first part the stepwise process of IGB is briefly described highlighting the relevant issues resulting from the application of this document to real case studies. In the second, the most recurrent problems detected during the assessments are discussed.

The aim of the paper has been to propose possible improvements to both IGB and ITC to overcome these issues and, if not possible, to highlight that the complexity of the process should be taken into account when setting the acceptable level of risk and the relevant assessment and rehabilitation requirements.

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