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Italian guidelines for the risk classification and management of existing bridges: a case study implementation and evaluation

A Thesis Submitted in Partial Fulfilment of the Requirements
for the Degree of Master of Science (Laurea Magistrale) in

Civil Engineering for the Mitigation of Risk from Natural Hazards

by

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ABSTRACT

Road infrastructure systems are a fundamental component of a society's well-being as they are essential for the transport of people, commercial goods, and the development of industrial and cultural activities. In emergency conditions, these systems become vital as they facilitate immediate post-emergency relief efforts, the transport of medical teams and the sheltering of injured people, as well as repair, rehabilitation, and daily supplies for distressed communities.

Following recent bridge failures in Italy, the Minister of Infrastructure and Transport issued specific guidelines for the classification and management of risk, safety assessment and monitoring of existing bridges. This document illustrates a multi-level approach that allows risk classification, account for different hazards (structural and foundational, seismic, landslides, and hydraulic). On the basis of this risk classification, different options are given to proceed to more detailed safety verifications.

The aim of this study was to apply the guidelines to an existing case study bridge portfolio located in Lombardy region of Northern Italy in collaboration with MSE (Milano Serravalle Engineering S.r.l.). In particular, the bridge portfolio consists of 108 bridges examined and inspected by practicing engineers, who collected information and photographic evidence of the registered defects to complete the first two levels of the approach proposed by the Italian guidelines (Level 0, Level 1). For each structure, the quality of the collected information has been checked and verified in order to evaluate the consistency, coherency of the data and the accuracy of coefficient and parameters assigned by the technical. Based on this analysis, it was possible to highlight advantages and disadvantages of the methodology when scrutinised via a case study application.

Based on this case study application and the subsequent observations made, an innovative approach to overcome some of the observed limitations of the current guidelines, moving from a maintenance philosophy to a performance is discussed.

ACKNOWLEDGEMENTS

Acknowledgements can be provided here (though it is not compulsory to include them).

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1. INTRODUCTION

The serviceability and safety of transportation infrastructure, such as bridges, are of extreme importance for a society since they are essential for the transport of people, commercial goods, and the development of industrial and cultural activities.

The safety of road networks and their components (i.e., bridges, tunnels, and overpasses) are usually taken for granted by users, even though many infrastructures have already exceeded their service life and/or the service conditions have changed during their life span. Important factors such as ageing, increasing traffic volume, and deterioration due to natural phenomena must be considered to assess the performance of each structure and to prioritise retrofit interventions that may be required. A further factor to consider for the prioritisation is the redundancy of the road network: due to high construction costs, there are often only few road alternatives, whose effectiveness in terms of travel time and cost of transportation is not competitive with the major roadway.

In recent years, many bridge collapses have occured in Italy. The most striking case is the one of the Morandi Bridge (14th August 2018); an infrastructure linking A7 and A10 motorways near Genova collapsed causing 43 fatalities and forced more than 560 residents to leave their homes for safety reasons. In addition to the human impact of this disaster, there have also been serious repercussions in terms of urban, motorway and rail traffic, as well as in terms of costs. As already mentioned, however, the list of collapsed bridges in the last decade comprises much more infrastructures: Massa Carrara Bridge (2020), bridge on the motorway section Torino-Savona (2019), bridge on A14 motorway (2017), bridge on the motorway section Milano-Lecco (2016), etc. It is clear that the need to improve the safety conditions of bridges in Italy is of foremost importance.

To this end, the Italian Minister of Infrastructure and Transport issued specific guidelines for the classification and management of risk, safety assessment and monitoring of existing bridges. This document was published on 17/04/2020 and became officially operational following the Ministerial Decree 578 in 17/12/2020; therefore, it is a newly issued tool and guideline to address the issues previously described. The aim of the study presented herein is to test the effectiveness of the guidelines via a case study application to an existing bridge network in Northern Italy, highlight advantages and disadvantages of the methodology, and to propose an alternative to overcome its main criticisms.

The thesis is divided into five chapters. The first chapter provides a brief summary of the main sources used for the development of the study; the second and third chapters consist, respectively, of the presentation of the case study and the application of the Italian guidelines for existing bridges to this case study database. Subsequently, in the fourth chapter, an evaluation of the methodology to discuss strengths and weaknesses is provided and finally, in the fifth chapter, a general summary and conclusion of the thesis are provided.

2. LITERATURE REVIEW

2.1 GUIDELINES FOR THE CLASSIFICATION AND MANAGEMENT OF RISK, SAFETY ASSESSMENT AND MONITORING OF EXISTING BRIDGES

The guidelines published by the Italian Minister of Infrastructure and Transport highlights a procedure to assess the safety level of existing bridges, to avoid inadequate damage states, and to reduce risk to an acceptable level. As suggested by the title itself, the guideline is divided into three parts: risk census and classification, safety verification, and bridge surveillance and monitoring. In particular, the guideline proposes a multi-level approach that, starting from a simple census of the bridges, leads to the classification of the risk associated with each of them, and, consequently, to prioritise the execution of surveillance and monitoring operations, safety verifications (when required by the methodology) and retrofit interventions.

The multi-level approach includes six levels with increasing complexity:

- Level 0 involves the census of all the existing bridges and of the associated characteristics by collecting the available information and documentations;
- Level 1 involves direct visual inspections and survey of the structure and of the geo-morphological and hydraulic characteristics of the area, in order to identify the state of deterioration and the main structural and geometrical characteristics of the bridges, as well as the potential risks associated to landslides and hydrodynamic actions;
- Level 2 allows risk classes to be assigned, denoted in the text as "Classe di Attenzione" (CdA), to each structure of the stock, on the basis of hazard parameters, vulnerability and exposure determined in the previous steps. Depending on this classification, one of the following levels is then carried out;
- Level 3 consists in performing preliminary assessment to understand whether it is necessary to proceed with in-depth analyses through the execution of the accurate verifications included in Level 4;
- Level 4 foresees the execution of accurate assessment according to what is indicated in the current Italian Code (Norme Tecniche per le Costruzioni 2018 – NTC18);

- Level 5 applies to bridges considered to have significant importance within the network. In these cases, it is useful to perform more sophisticated analyses such as those regarding resilience of the specific branch of the road network and/or of the transport system, analysing the transport significance, the interaction between the structure and the road network to which it belongs and the consequences of a possible interruption of the bridge functionality on the socio-economic context.

An overview of the approach is showed in the flowchart reported in Figure *1*.

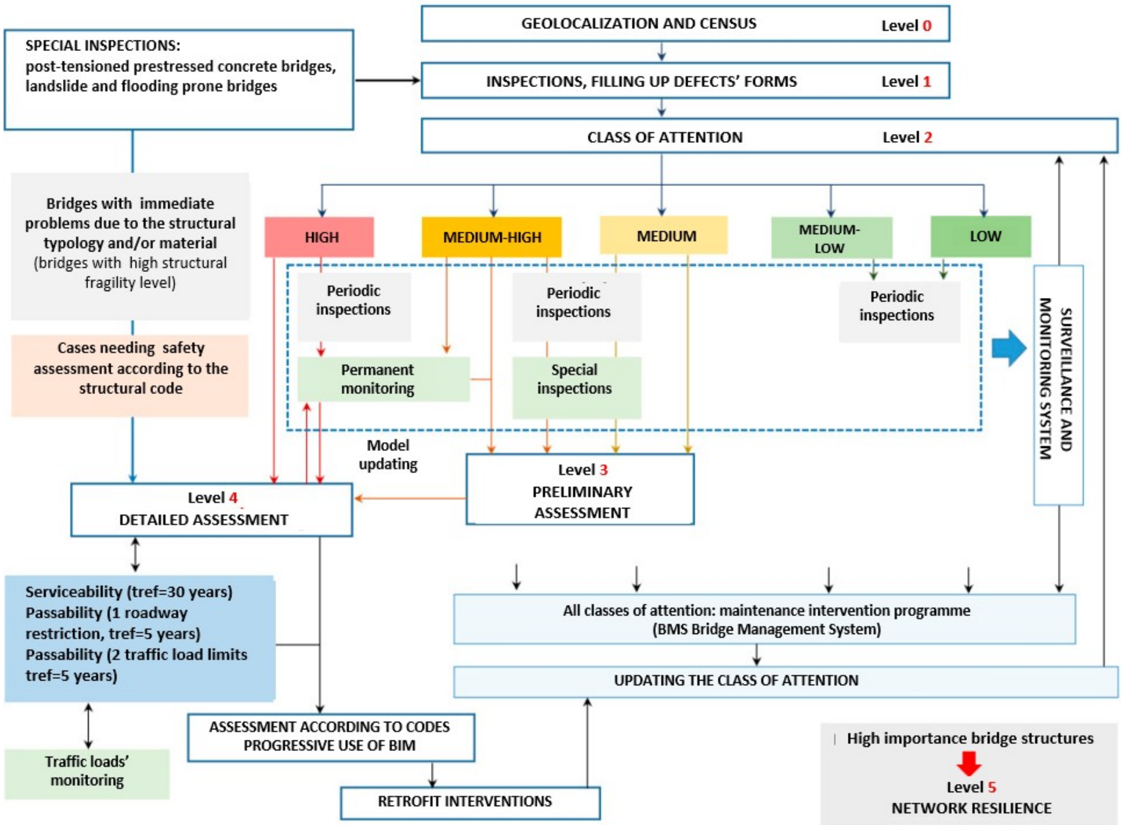

Figure 1 - Multi-level approach and relationships between levels of analysis (Adopted from "The Italian Guidelines on Risk Classification and Management of Bridges: Applications and Remarks on Large Scale Risk Assessments", G. Santarsiero, A. Masi, V. Picciano, A. Digrisolo (2021))

The core of the approach is Level 2, namely the definition of the risk classes. In particular, five classes are defined (high, medium-high, medium, medium-low, and low) and they correspond to specific actions in terms of survey, monitoring, and verifications.

In particular:

- For bridges having ***High risk***, it is appropriate to perform more accurate analysis, both in terms of safety verifications and in terms of investigating geotechnical and/or structural characteristics. Moreover, ordinary periodic inspections and, if necessary, also extraordinary periodic inspections are mandatory together with the installation of periodic or continuous monitoring systems.
- For bridges having ***Medium-High risk***, Level 3 preliminary assessment is envisaged. Also in this case, ordinary periodic inspections and, if necessary, also extraordinary periodic inspections are mandatory together with the installation of periodic or continuous monitoring systems. Regarding safety verifications, the owner and/or manager verifies case-by-case if Level 4 accurate analyses are needed, depending on the typology and state of defects, and considering the results of Level 3 preliminary assessment.

  If the data collected from the monitoring systems highlight evolving phenomena and relevant criticisms not previously caught from initial inspections, Level 4 analyses should be conducted, re-classifying the bridge from a ***medium-high*** to ***high*** class.
- For bridges having ***Medium risk***, Level 3 preliminary assessment is envisaged, and ordinary periodic inspections are mandatory. If rapidly evolving degradation phenomena are detected, it is necessary to perform also extraordinary periodic inspections. Moreover, the owner and/or manager verifies case-by-case if the installation of periodic or continuous monitoring systems is necessary, re-classifying the bridge from medium to medium-high class, and/or if Level 4 analyses should be performed, re-classifying the bridge from medium to high class.
- For bridges having ***Medium-Low and Low risk***, no further analyses are necessary. However, frequent periodic inspections should be carried out.

Regardless of the class, essential maintenance interventions to improve the conditions of the detected issues must be executed. Additionally, considering the results of inspections and monitoring, the risk class, and therefore all the associated measures, must be reassessed. In any case, the re-evaluation of the risk class must be done at least every two years.

The different analysis levels should not be necessarily performed in a sequential manner since there is no need to wait the completion of the activities comprised in one level to proceed with the next one. For example, the census that is performed at Level 0 typically requires a great effort in terms of time, and Level 1 visual inspections may be conducted simultaneously, allowing the owner and/or manager of the considered branch of the road network to prioritise the structures whose conditions are of greatest concern. To this aim, it is also necessary to assess the viability and characteristics of the road network, considering, in particular, parameters such as the *traffic volume*, the presence of *exceptional loads*, the *age* of the road section, and the *state of preservation* of the bridges themselves.

2.1.1 Level 0: Census

Census covered by Level 0 of the approach consists in cataloguing all bridges in the national territory with the objective of identifying the number of structures to be managed and their main characteristics, considering geometrical and structural properties, their location and the branch of the road network to which they belong.

Data collection leads to the creation of a comprehensive database of bridges, which allows their subdivision into macro-classes and the prioritisation of Level 1 visual inspections and surveys. As already mentioned, the process involves the retrieval of technical documentation (i.e., initial design, construction phases, subsequent retrofit intervention design, etc.) and administrative documentation, which enables the reconstruction of the actions and transformations to which the structure has been subjected over the years. In addition, the role that the bridge plays in the transport system should be also investigated, considering the following aspects: traffic volume and type, presence and features of alternative routes (i.e., redundancy).

A fact/information sheet on Level 0 census is provided in Annex A of the guidelines.

2.1.2 Level 1: Visual inspection and survey

Level 1 of the multi-level approach consists of executing visual inspection and surveys on the whole national bridge stock, previously registered in Level 0 census. The aim of this process is either to verify the reliability of the information collected in the first step, and to add further details regarding both the geometrical and structural properties, and the state of preservation of the structures.

Investigations include photographic survey, geometric survey and survey of the main degradation phenomena registered. All the bridge elements must be examined both at the intrados and extrados to have a complete and proper visibility of the whole structure, including enclosed compartments such as deck and piers box sections.

While the first two types of surveys mostly serve to validate information collected in Level 0, the assessment of the state of preservation of the structure is a completely new step. In particular, it consists of identifying degradation phenomena and criticisms affecting the structure and recording them in appropriate reports specifying their presence, intensity, and extent (Annex B of the guidelines). Each defect is assigned with a weight (G), which ranges from 1 to 5: minor defects have a value of 1, very severe defects have a value of 5 and are typically associated with structural deficiencies and deterioration. Moreover, for defects of weight 4 and 5, there is the possibility to sign the box denoted by "PS" to notify the possibility of static problems. In addition, the two parameters k1 and k2are introduced to quantify respectively the extent and the intensity of the defect. Both quantities can assume one of the following values: 0.2, 0.5 or 1. The information to quantify G, k1, and k2 is provided in Annex C of the guidelines, together with definition, cause and possible consequence of each defect. In particular, the defects are divided into macro-categories depending on the material as follows:

- Steel or Metal;
- Masonry;
- Reinforced/Prestressed Concrete;
- Prestressed Concrete (which includes all the specific defects, not verifiable in simply reinforce concrete structures);
- Timber.

Further categories include:

- General defects;
- Supports;
- Embankments and Foundations;
- Joints;
- Additional elements (e.g., guardrails, pavement, etc.).

If a particular defect is not detected in the structure, three possibilities are available:

- The defect is not applicable to the considered structural typology (box "NA");
- The defect cannot be detected through visual inspection (box "NR");
- The defect is not present (box "NP").

Final reports must clearly identify the location of each defect and include both the date of the inspection and the name of the technical operator who conducted it.

The inspection of "critical elements", namely elements particularly subjected to degradation phenomena and whose malfunctioning can severely affect the structural response of the bridge, is of foremost importance. Critical elements differ from one structural typology to another; for example, Gerber seats, prestressing wires, and seating devices are considered critical.

Level 1 inspections also include preliminary considerations regarding hydraulic and landslide risk. Also in this case, proper reports are supplied in Annex B of the guidelines. In particular, indications about the three quantities that identify risk (i.e., hazard, vulnerability, and exposure) must be registered.

Regarding landslide risk, the three main parameters considered to quantify the hazard are: magnitude/mobilizing volume, maximum potential velocity, and activity status. In addition, further secondary parameters are included to adjust the final value assigned to this quantity (5 possible levels of increasing severity), considering the reliability of the evaluation and the presence of retaining structures. Vulnerability is estimated on the basis of the structural and foundation typology and considering the extent of the potential interference surface (an event may affect the whole structure or only a specific part of it).

Regarding hydraulic risk, to quantify the hazard it is important to estimate the free-board value and eventual waterway section reductions due to erosive phenomena, whereas the presence of particular conditions that may increase the vulnerability should be investigated.

In both cases, exposure depends on the possible consequences that an event may induce when affecting a structure.

The amount of data collected in the first two steps of the multi-level approach enables the identification of such structures whose conditions are so deteriorated that the execution of Level 4 accurate analyses is mandatory. In these cases, Level 2 and 3 of the process are skipped to accelerate interventions. In particular, this happens when:

- Safety verifications according to NTC18 ("Norme Tecniche per le Costruzioni", Ministerial Decree 17/01/2018, GU 20/02/2018, Ministero delle Infrastrutture e dei Trasporti (MIT) (2018)), Chapter 8.3 are needed;
- The structure is characterised by brittle failure modes (e.g., shear failure, span failure due to unseating at movement joints, etc.).

2.1.3 Level 2: Analysis of relevant risks and classification

As previously described, Level 2 of the approach allows the risk class (CdA) of each structure of the stock to be assigned through a simplified estimation of the risk parameters: hazard, vulnerability, and exposure. Also in this case, the ultimate objective is to define a prioritisation scheme to be followed to carry out surveys, safety verification, retrofit interventions, and install monitoring systems in an optimised way.

The Italian guideline distinguishes five risk classes: high, medium-high, medium, medium-low, and low. The final CdA of an individual structure is obtained as a combination of the CdAs associated to the four risk categories considered relevant for bridge structures:

- Structural and foundational risk;
- Seismic risk;
- Landslide risk;
- Hydraulic risk.

The assessment of the different CdAs is based on common rules, although the parameters to be considered vary from case-to-case. In particular, primary and secondary parameters are selected for hazard, vulnerability, and exposure associated to each risk category and the individual CdA is obtained with an approach by classes and logical operators, without any numerical calculation. Primary and secondary parameters are determined on the basis of Level 0 census and Level 1 visual inspections. Depending on the value of primary parameters, the same five classes aforementioned are determined and subsequently modified considering the values of secondary parameters, which are typically divided in two or more classes. Once hazard, vulnerability and exposure classes are determined for a specific risk type (e.g., structural and foundational), they are combined to obtain the final CdA of that risk type (e.g., structural and foundational CdA).

The logic behind the determination of the CdA is summarised in Figure ***2***.

Figure 2 - Logic flow for the CdA determination (Adapted from "Linee guida per la classificazione e gestione del rischio, la valutazione della sicurezza ed il monitoraggio dei ponti esistenti", Ministero delle Infrastrutture e dei Trasporti (MIT) (2020))

2.1.3.1 *Structural and Foundational CdA*

The definition of the structural and foundational CdA involves those parameters affecting the overall response of a structure in its typical operating conditions. Primary and secondary parameters are summarised in Table ***1***.

Table 1 – Primary and secondary parameters for the determination of hazard, vulnerability and exposure factors associated with structural and foundational risk

	Primary Parameters	Secondary Parameters
Hazard	Extent of loads	-
Vulnerability	Defectiveness level Static scheme, span length, material, number of bays	Degradation speed Design code
Exposure	Average daily traffic and average span length	Alternative routes Typology of bypassed entity Transport of dangerous goods

2.1.3.2 *Seismic CdA*

The definition of the seismic CdA involves those parameters affecting the seismic response of a structure. Primary and secondary parameters are summarised in Table ***2***.

Table 2 – Primary and secondary parameters for the determination of hazard, vulnerability and exposure factors associated with seismic risk

	Primary Parameters	Secondary Parameters
Hazard	Peak ground acceleration (PGA) and topographic category	Soil/subsoil category
Vulnerability	Structural scheme, span length, material Defectiveness level	Design criteria
Exposure	Average daily traffic and average span length	Alternative routes Typology of bypassed entity Transport of dangerous goods Bridge strategic nature

2.1.3.3 *Landslide CdA*

The definition of the landslide CdA takes into account specific parameters that indicate the level of involvement of the structure in possible landslide phenomena, both from a spatial and temporal point of view. In this context the term “hazard” is replaced by “susceptibility”. Primary and secondary parameters are summarised in Table ***3***.

Table 3 – Primary and secondary parameters for the determination of susceptibility, vulnerability and exposure factors associated with landslide risk

	Primary Parameters	Secondary Parameters
Susceptibility	Slope instability (magnitude, speed, activity status)	Model uncertainty Mitigation measures
Vulnerability	Structural and foundational typology/stiffness	Interference extent
Exposure	Average daily traffic and span length	Alternative routes Typology of bypassed entity Bridge strategic nature

2.1.3.4 *Hydraulic CdA*

Hydraulic CdA is defined by parameters that account for structural involvement, both from a spatial and temporal point of view. Primary and secondary parameters are summarised in Table *4*.

Table 4 – Primary and secondary parameters for the determination of susceptibility, vulnerability and exposure factors associated with hydraulic risk

	Primary Parameters	Secondary Parameters
Hazard/ Susceptibility	Probability of occurrence and event consistency	Model uncertainty Mitigation measures
Vulnerability	Resilience to the natural event	Typology, magnitude, and frequency Typology and efficiency of mitigation structures
Exposure	Potential damage	Typology of bypassed entity Bridge strategic nature Damage extent

2.1.3.5 *Global CdA*

The global, multi-risk CdA of a bridge is obtained from the partial CdAs once again with an approach by class and logical operators. For clarity, a unified “Landslide and Hydraulic CdA” is introduced as shown in Table *5*.

Table 5 – CdAs combination for the definition of the unified hydraulic and landslide CdA

<table>
<tr><th colspan="2"></th><th colspan="5">Landslide CdA</th></tr>
<tr><th colspan="2"></th><th>High</th><th>Medium-High</th><th>Medium</th><th>Medium-Low</th><th>Low</th></tr>
<tr><th rowspan="5">Hydraulic CdA</th><td>High</td><td colspan="2">High</td><td colspan="2">Medium-High</td><td>Medium</td></tr>
<tr><td>Medium-High</td><td>High</td><td colspan="2">Medium-High</td><td colspan="2">Medium</td></tr>
<tr><td>Medium</td><td colspan="2">Medium-High</td><td colspan="2">Medium</td><td>Medium-Low</td></tr>
<tr><td>Medium-Low</td><td>Medium-High</td><td>Medium</td><td colspan="2">Medium-Low</td><td>Low</td></tr>
<tr><td>Low</td><td colspan="3">Medium</td><td>Medium-Low</td><td>Low</td></tr>
</table>

Lastly, the global CdA is defined as shown in Table ***6***.

Table 6 – CdAs combination for the definition of the global CdA

Structural and Foundational CdA: ***HIGH***

<table>
<tr><th colspan="2"></th><th colspan="5">Landslide and Hydraulic CdA</th></tr>
<tr><th colspan="2"></th><th>High</th><th>Medium-High</th><th>Medium</th><th>Medium-Low</th><th>Low</th></tr>
<tr><th rowspan="5">Seismic CdA</th><td>High</td><td colspan="5">High</td></tr>
<tr><td>Medium-High</td><td colspan="5">High</td></tr>
<tr><td>Medium</td><td colspan="5">High</td></tr>
<tr><td>Medium-Low</td><td colspan="5">High</td></tr>
<tr><td>Low</td><td colspan="5">High</td></tr>
</table>

Structural and Foundational CdA: ***MEDIUM-HIGH***

<table>
<tr><th colspan="2"></th><th colspan="5">Landslide and Hydraulic CdA</th></tr>
<tr><th colspan="2"></th><th>High</th><th>Medium-High</th><th>Medium</th><th>Medium-Low</th><th>Low</th></tr>
<tr><th rowspan="5">Seismic CdA</th><td>High</td><td colspan="2">High</td><td colspan="3">Medium-High</td></tr>
<tr><td>Medium-High</td><td>High</td><td colspan="3">Medium-High</td><td>Medium</td></tr>
<tr><td>Medium</td><td colspan="3">Medium-High</td><td colspan="2">Medium</td></tr>
<tr><td>Medium-Low</td><td colspan="2">Medium-High</td><td colspan="3">Medium</td></tr>
<tr><td>Low</td><td>Medium-High</td><td colspan="4">Medium</td></tr>
</table>

Structural and Foundational CdA: ***MEDIUM***

<table>
<tr><th colspan="2"></th><th colspan="5">Landslide and Hydraulic CdA</th></tr>
<tr><th colspan="2"></th><th>High</th><th>Medium-High</th><th>Medium</th><th>Medium-Low</th><th>Low</th></tr>
<tr><th rowspan="5">Seismic CdA</th><td>High</td><td>High</td><td colspan="2">Medium-High</td><td colspan="2">Medium</td></tr>
<tr><td>Medium-High</td><td colspan="2">Medium-High</td><td colspan="3">Medium</td></tr>
<tr><td>Medium</td><td>Medium-High</td><td colspan="4">Medium</td></tr>
<tr><td>Medium-Low</td><td colspan="4">Medium</td><td>Medium-Low</td></tr>
<tr><td>Low</td><td colspan="3">Medium</td><td colspan="2">Medium-Low</td></tr>
</table>

Structural and Foundational CdA: ***MEDIUM-LOW***

<table>
<tr><td colspan="2" rowspan="2"></td><td colspan="5">Landslide and Hydraulic CdA</td></tr>
<tr><td>High</td><td>Medium-High</td><td>Medium</td><td>Medium-Low</td><td>Low</td></tr>
<tr><td rowspan="5">Seismic CdA</td><td>High</td><td>Medium-High</td><td colspan="4">Medium</td></tr>
<tr><td>Medium-High</td><td colspan="4">Medium</td><td>Medium-Low</td></tr>
<tr><td>Medium</td><td colspan="3">Medium</td><td colspan="2">Medium-Low</td></tr>
<tr><td>Medium-Low</td><td colspan="2">Medium</td><td colspan="3">Medium-Low</td></tr>
<tr><td>Low</td><td>Medium</td><td colspan="4">Medium-Low</td></tr>
</table>

Structural and Foundational CdA: ***LOW***

<table>
<tr><td colspan="2" rowspan="2"></td><td colspan="5">Landslide and Hydraulic CdA</td></tr>
<tr><td>High</td><td>Medium-High</td><td>Medium</td><td>Medium-Low</td><td>Low</td></tr>
<tr><td rowspan="5">Seismic CdA</td><td>High</td><td colspan="3">Medium</td><td colspan="2">Medium-Low</td></tr>
<tr><td>Medium-High</td><td colspan="2">Medium</td><td colspan="3">Medium-Low</td></tr>
<tr><td>Medium</td><td>Medium</td><td colspan="4">Medium-Low</td></tr>
<tr><td>Medium-Low</td><td colspan="4">Medium-Low</td><td>Low</td></tr>
<tr><td>Low</td><td colspan="2">Medium-Low</td><td colspan="3">Low</td></tr>
</table>

2.1.4 Level 3: Bridge preliminary assessment

Level 3 preliminary evaluation aims to assess the quality and typology of the defects recorded during Level 1 visual inspections and to estimate the current performance of the bridge. In first place, for structures with medium-high CdA, it is important to carefully analyse defects and degradation phenomena to determine whether Level 4 verifications are necessary. Moreover, a comparison between the capacities obtained with the code of the time and the current one must be performed. Assuming the bridge under consideration has been design in accordance with the code in force at the time of its construction, the demand on each element of the bridge can be calculated and compared with the one induced by the loads defined in the current code. For what concerns the Italian code history, at least until 1980 two bridge categories were considered: on 1st category bridges the transit of military equipment was permitted, whereas in 2nd category bridges only civil vehicles were considered. In general, the effects induced by military equipment are comparable to those prescribed by the current code (in some cases even larger), whereas it is not the case for civil vehicles.

Considering the results of the preliminary assessment, the owner and/or manager must evaluate case by case whether Level 4 accurate verifications should be performed.

2.1.5 Level 4: Safety Verifications

The accurate Level 4 safety verifications described in the guidelines are in accordance with the current Italian code "Norme Tecniche per le Costruzioni" (NTC18) (Ministerial Decree 17/01/2018, GU 20/02/2018, Ministero delle Infrastrutture e dei Trasporti (MIT) (2018)), in particular with Chapter 8, and with the associated documentation (Circ. 21/01/2019, n.7/CSLLPP, GU 11.02.2019, Ministero delle Infrastrutture e dei Trasporti (MIT) (2019)). The time interval to be considered for the safety verifications depends on the objective of the analysis and is indicated in the guidelines as the reference time (tref). At the end of this period, the analysis should be performed again.

According to NTC18, Chapter 8.3, safety verifications are necessary if even only one of the following conditions arises:

- Clear reduction of the load-bearing and/or deformation capacity of the structure or some of its components due to: significant deterioration of the materials' mechanical characteristics, significant deformations leading to problems at the foundation level, damage induced by environmental actions (earthquakes, wind, snow, and thermal variations), exceptional actions (impacts, fires and explosions) or abnormal operation and use;
- Proven serious errors in design or construction;
- Change of use of the structure or part of it;
- Execution of not explicitly structural retrofit interventions, when they interact with structural elements, reducing their capacity and/or modifying their stiffness;
- Execution of structural retrofit interventions;
- Structure designed in contrast to the code in force at the time of its construction.

Furthermore, the same chapter specifies that safety verifications must allow the determination of whether: the structural use can continue without interventions, the structural use must be modified or interventions to increase the structural safety must be designed.

Based on the requirements of the code and the associated documentation, in the guidelines a bridge can be classified as:

- **Appropriate**, if the safety verifications prescribed by NTC18 and considering the associated loads and factors are satisfied;
- **Operational**, if safety verifications performed following the principles of NTC18 but considering a reduced reference time are satisfied (tref typically assumed to be 30 years);

- **Viable**, if safety verifications performed following the principles of NTC18 but considering a further reduced reference time (tref typically assumed to be 5 years) are satisfied and if interventions such as load limitations or usage restriction are adopted.

To quantify the safety level of an existing structure, the two parameters E and V,i are introduced. The first one represents the ratio between the maximum seismic action that an existing structure can sustain and the one used for the design of a new structure, whereas the second one is obtained as the ratio between the maximum vertical variable load that can be sustained by the portion *i* of the structure and the one used for the design of a new structure. As specified in the circular, the typical parameter used for the definition of E is the ground acceleration agS. In general, both parameters may be lower than 1, which means that the safety checks are not satisfied. If this is the case, after the adoption of specific measures the updated V,i must be grater than 1, while specific reference values are provided for E in NTC18 and the associated documentation.

An indispensable element for the safety assessment, as well as for the selection of retrofit interventions, is the knowledge of the bridge's history. The process is organised in successive levels of detail and includes the following activities:

a. Historical-critical analysis;
b. Original design analysis to understand the design idea and to identify any criticisms linked to design errors or gaps in the simplified approaches provided by the code in force at the time of construction;
c. Survey (i.e., geometric and structural characterisation; construction details, cracks, and instabilities investigation);
d. Geological characterisation of the site to determine the main stratigraphic, lithological, geomorphological, and seismic elements;
e. Surveys aimed at the characterisation of materials and construction details (in situ and laboratory tests);
f. Evaluation of the hydraulic conditions, analysis of possible detrimental phenomena and definition of the efficiency of pre-existing mitigation structures;
g. Evaluation of the geo-morphological conditions, analysis of possible slope instabilities and definition of the efficiency of pre-existing mitigation structures.

For what concerns point (e) of the list, the associated documentation distinguishes three detailing levels for tests:

- Limited tests, which allow the correspondence between the design documentation and the actual structure to be determined;

- Extensive tests, typically performed when construction drawings are not available or when only insufficient and/or incomplete information are provided;
- Exhaustive tests, typically carried out when design documentations are not available.

Depending on the level of detail obtained through the cognitive process, three knowledge levels of increasing information are distinguished for the definition of the confidence factors (FC): LC1, LC2, LC3. LC1 is achieved when at least historical-critical analysis, geometrical survey, and limited tests for the characterisation of materials and construction details are performed. The associated confidence factor is $FC = 1.35$. LC2 is achieved when at least historically-critical analysis, geometrical survey, and extensive tests for the characterisation of materials and construction details are performed. The associated confidence factor is $FC = 1.2$. Lastly, LC3 is achieved when at least historically-critical analysis, geometrical survey, and exhaustive tests for the characterisation of materials and construction details are performed. The associated confidence factor is $FC = 1$.

At the end of the cognitive process, the actual verification process begins. In particular, the process is divided into the following phases:

- Load evaluation (i.e., permanent loads, traffic loads, seismic loads, etc.);
- Load combinations;
- Definition of the mechanical parameters of the materials and associated factors;
- Numerical modelling;
- Structural analysis and evaluation of the static and dynamic actions;
- Capacity evaluation and static and seismic safety checks.

A detailed explanation regarding the definition of the loads that should be considered and of the associated factors is reported in the guidelines.

An important step to reproduce the real behaviour of an existing structure is to reduce to minimum all the uncertainties that may be introduced in the analysis (e.g., loads uncertainty, material uncertainty, model uncertainty, etc.). For this purpose, static and dynamic tests can be conducted on the structure or on a part of it that can be considered representative of its overall behaviour in order to calibrate and validate the structural model.

In general, only ultimate limit state (ULS) checks are performed, with the exception of Class IV structures, which require also serviceability limit state (SLS) verifications.

For the foundation system, in accordance with NTC18, safety verifications are mandatory only if conditions that may lead to global instability exist or at least one of the following occurs:

- Significant disruptions linked to foundation settlement are registered;
- Overturning and/or sliding may occur;
- Soil liquefaction under the design seismic excitation may occur.

The general procedures to assess the safety level of an existing structure are provided by the Italian code (NTC18) and the associated circular. All the references are clearly reported on the guidelines.

2.1.6 Level 5: Transport significance assessment

Level 5 of the multi-level approach is linked to those bridges considered to have significant importance within the network and is not explicitly investigated through the guidelines.

As mentioned previously, in these cases, it is useful to perform more sophisticated analyses such as those regarding resilience of the specific branch of the road network, analysing the transport significance, the interaction between the structure and the road network to which it belongs and the consequences of a possible interruption of the bridge functionality on the socio-economic context. For this purpose, published transport studies are of foremost importance, together with temporary traffic flow counting sessions that can validate them or highlight the need for updating.

2.1.7 Bridge surveillance and monitoring

The last chapter of the guidelines regards the discussion about surveillance and monitoring systems, which represent the totality of control, inspection and monitoring activities on the structures that the owner and/or manager of the network must carry out in order to ensure the availability, functionality, and maintenance of the safety conditions.

The operational tools comprehend:

- Ordinary periodic inspections;
- Extraordinary inspections;
- Non-destructive and semi-destructive surveys;
- Static load tests and dynamic response measurements;
- Instrumental monitoring;
- Data analysis and interpretation algorithms;
- Models representative of the actual behaviour;
- Condition indices and degradation models;
- Computer databases.

The procedures envisaged for each of the activities listed above are described in detail in the guidelines.

2.2 INFRA-NAT PROJECT

The INFRA-NAT project focused on the evaluation of the seismic vulnerability and risk of the Italian roadway bridge stock, within the framework of an EU Civil Protection sponsored project. The results of the analyses allow retrofit priorities and real-time scenario evaluation to be highlighted for the purpose of emergency rescue operations.

The procedure to evaluate the risk was divided into different steps:

- Creation of a unified database;
- Definition of taxonomies;
- Creation of synthetic finite elements (FE) models consistent with the properties of real bridges;
- Definition of seismic hazard;
- Definition of performance levels and of the associated capacity thresholds;
- Vulnerability evaluation;
- Road network modelling,
- Evaluation of consequences.

2.2.1 Unified database

The purpose of creating a unified database of existing bridges on the national territory was to facilitate the subsequent creation of finite element models, which were necessary to carry out the analyses. In particular, the information necessary to build the FE models comprehend geometry, loads, seismic masses, and nonlinear properties of elements and supports.

2.2.2 Taxonomies definition

Before proceeding with the risk assessment, the bridge inventory was analyzed to infer the most representative bridge types existing in the study area. Due to the lack of base information, it was not possible to produce specific numerical models for every single bridge in the inventory, therefore bridges were grouped into taxonomies based on key structural properties available for many assets in the inventory. Bridges classified under the same taxonomy were assumed to have similar performance under similar seismic excitation intensities.

The definition of the structural characteristics that define the choice of taxonomies were determined based on expert opinion and common practice, considering the available information stored in the database. In particular, for the INFRA-NAT Project, the considered structural characteristics to define the taxonomies are summarized in Table 7:

Table 7 – Structural characteristics to define taxonomies

Material	Number of Spans	Static Scheme	Deck Type	Pier Type
Reinforced concrete (RC)	Variable (1, 2, 3, ...)	Simply Supported Continuous Frame	Beam Plate Box	Wall Single Column Multiple Columns

For the Italian case study, a total of six taxonomy branches were selected. The characteristics of each class are summarised in Table ***8***.

Table 8 - Taxonomy branches

<table>
<tr><th>Material</th><th>Number of Spans</th><th>Static Scheme</th><th>Deck Type</th><th>Pier Type</th></tr>
<tr><td rowspan="6">Reinforced concrete (RC)</td><td rowspan="3">2 to 4</td><td rowspan="2">Simply Supported</td><td rowspan="2">Beam</td><td>Multiple Columns</td></tr>
<tr><td>Wall</td></tr>
<tr><td>Frame</td><td>Plate</td><td>Any</td></tr>
<tr><td rowspan="3">Above 5</td><td rowspan="3">Simply Supported</td><td rowspan="3">Beam</td><td>Multiple Columns</td></tr>
<tr><td>Wall</td></tr>
<tr><td>Single Column</td></tr>
</table>

Once the representative taxonomy branches were defined, the database was mined to parametrise key structural characteristics of each taxonomy. Global parameters, such as cover distance, concrete and steel strength were taken from statistical distributions created from the entire database, while specific variables, such as pier height, span length and reinforcement ratios were taken from assets that match the specific taxonomy. This information was used during the numerical modelling framework to produce synthetic models consistent with the properties of real bridges in the database.

2.2.3 Modelling

Within the framework of the INFRA-NAT Project, an ad-hoc application called BRI.T.N.E.Y. (BRIdge auTomatic Nonlinear analysis-based Earthquake fragilitY) was developed to automatise the procedure. In particular, this application performs queries to read the database information and creates Finite Elements (FE) models for carrying out nonlinear time history analysis (NTHA) with OpenSees. Moreover, the post-processing of the results is also carried out automatically.

The carriageway was defined as a sequence of span segments to have a better discretisation of masses and to allow for section changes, internal releases and change in plan orientation (e.g., curves). BRI.T.N.E.Y. also allows for special cases in which multiple carriageways supported on the same substructure are present.

An example of carriageway modelling is provided in ***Figure 3***.

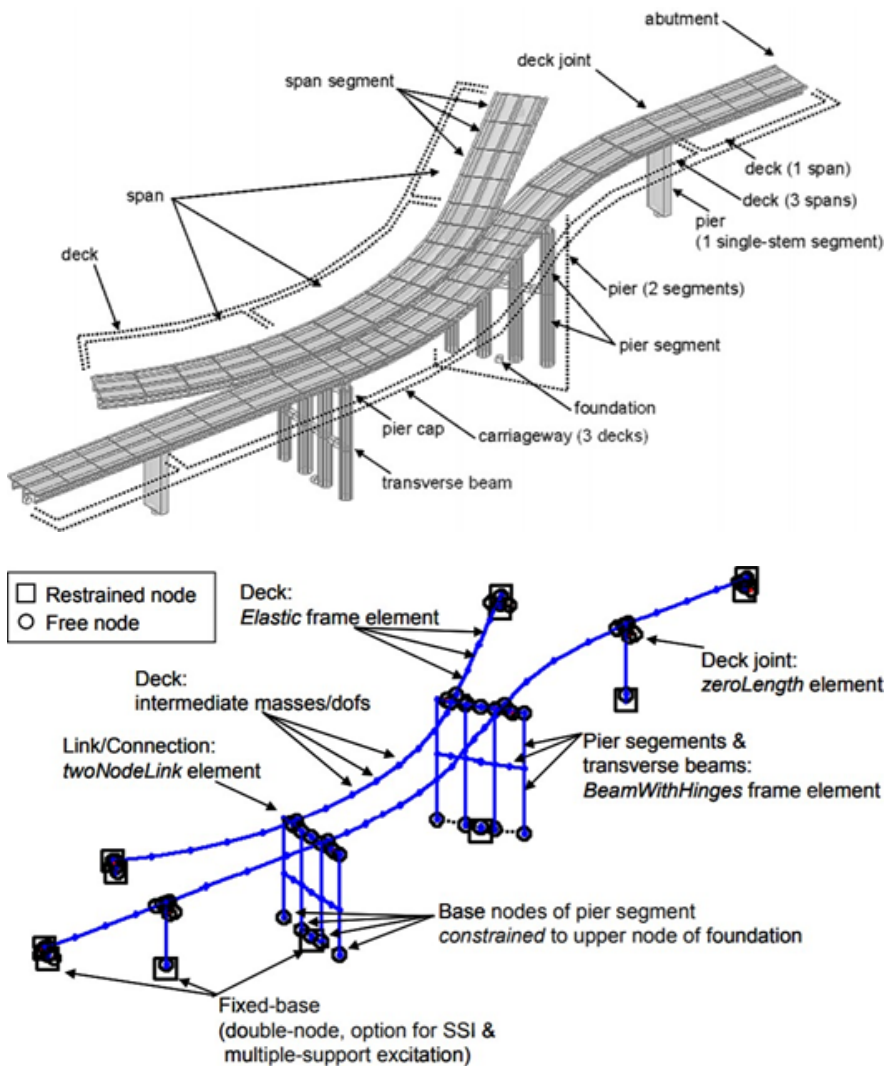

Figure 3 - INFRA-NAT Project example of carriegeway modelling (Adopted from INFRA-NAT Project, "D3.2 - Portfolio of bridge typology numerical models and fragility functions")

The created elements were either frame elements, Elastic for the deck and BeamWithHinges for the pier segments and the transverse beams, respectively, or zeroLength elements for deck connections and twoNodeLink elements for bearing devices within super- to sub-structure connections. Inelasticity was modeled within both frames and zeroLength elements. For this purpose, in the beamWithHinges elements, the cross section was discretised into fibers. RigidLink elements were also used to model connections. Uniaxial constitutive models employed for the fibre section of inelastic elements were the Scott-Kent-Park concrete model and the bilinear steel model.

Connections between super- and sub-structure were specified at the start and end of each span. The geometric quantities used to describe the connection were: the horizontal and vertical distances; the skew angle; the number and spacing of bearing devices. If the number of bearings was zero the connection was assumed to be rigid, otherwise a force-deformation relationship must be specified for the horizontal degrees of freedom of the devices.

Pier caps were modelled as a load and mass for single-stem piers, or explicitly, as in frame piers. Abutment and pier foundations can be modelled within the application as follows. A rigid link connected two nodes, one at the pier or abutment wall base, and the other at the foundation mat base. The inertia associated with the mat was attributed to the second node. The latter node was linked through a ZeroLength element to a coincident node, which was fixed to the external support. The ZeroLength element was used in a sub-structuring approach to model the dynamic impedance of the soil-foundation system (the frequency-dependence of the impedance was approximated through assemblies of springs, dashpots, and fictitious masses). The corresponding stiffness, damping and mass terms were attributed to the degrees of freedom of the ZeroLength element. Ground motion was input to the structure as a motion-proportional force/moment in each degree of freedom of the mat-base node.

Even though the application allows the foundation system to be modelled appropriately, it was neglected because of the lack of information for the 485 considered bridges studied. Besides, this was not considered a major issue for the purpose of the risk assessment of the bridge portfolio, for several reasons. The first reason is that bridge foundations are traditionally designed with a significant conservatism in Italy. Further, systematic failure of inadequate deck-pier connections, as observed in all recent events, limits the forces transmitted to the foundations. Finally, the third reason is that SSI has been shown to be less important than differential input. Hence, the authors felt that, to improve the model of the bridge, priority should be given to the differential input.

2.2.4 Performance levels and associated capacity thresholds

In the INFRA-NAT Project, two performance levels were considered: damage and collapse. The structural elements included in the analysis were piers and bearings that were considered more vulnerable than pier foundations, abutments, and the deck.

Regarding the piers, two failure modes were considered: shear failure, and flexural failure (expressed in terms of chord rotation). The first one is a brittle type of failure, which occurs without the bridge exhibiting any sign of damage before its failure. For this reason, only the collapse limit state was defined for the shear capacity of the piers. On the contrary, for the second failure mode both damage and collapse performance levels were defined, and yield and ultimate chord rotations were computed from the associated curvatures automatically from a bilinear fit of a section moment-curvature analysis.

On the other hand, the failure mode associated with bearing devices is due to the exceedance of the displacement capacity of the device itself. Bearings' failure can lead to the simple unseating of the deck from the bearing or to the full loss of support from the pier head. The first condition detects a damage limit state, while the second a collapse limit state. The displacement capacity of bearings was considered as deterministically known since it is derived from the pier cap and bearing seats geometry.

2.2.5 Vulnerability evaluation

The method adopted for fragility assessment in INFRA-NAT was based on non-linear time history analysis and was carried out with the aforementioned application BRI.T.N.E.Y. The fragility curve was obtained pointwise for nine increasing values of the selected intensity measure (average spectral acceleration), each of them associated with an increasing value of mean return period of exceedance.

At each intensity level, non-linear time-history analysis provided a sample of response corresponding to demand values D in each bridge component, which were compared to the corresponding capacity values C. The obtained sample of the component demand to capacity ratios y = D/C were used to obtain a global structural D/C ratio (Y). If a simple series system scheme is considered (adequate for simple structural systems such as girder bridges), the weakest failure mode determines global failure, and hence the global Y for the j^{th} intensity and k^{th} motion is given in terms of the *m* local D/C ratios by:

$$Y_{jk} = \left(y_{1jk}, \dots, y_{njk}\right) \quad j = 1, \dots, 9; \quad k = 1, \dots, m$$

The *m* values of Y at each intensity level were used to fit a conditional lognormal distribution, by estimating log-mean lnYj and log-standard deviation lnYj, and to determine the probability of exceedance of the unit value of Y that marks the attainment of the performance level D ≥ C. This is the fragility value at the j^{th} intensity level:

$$p(j) = 1 - \phi((ln1 - \mu_{lnY_j})/\sigma_{lnY_j})$$

The vulnerability of the different structural components may increase due to ageing phenomena such as cracking and corrosion. In recent years, in fact, there have been multiple bridge collapses in Europe not caused by the occurrence of seismic activity, but by both the increase of traffic loads and bridge deterioration.

The ageing phenomenon of cracking can affect the shear resistance of a structure, leading to brittle failure of the asset. Therefore, it is of utmost importance to account for it in the fragility assessment analysis. Recently a prioritisation scheme to perform detailed analysis of vulnerable bridges was compiled for the Italian road network (Proestos, Calvi, & Bentz, 2017). The goal of the scheme was to provide tables and charts of maximum crack widths for specific locations that represent the maximum crack widths prior to collapse. A reserve capacity metric is determined for the bridges of the stock, which allows retrofit interventions for bridges with the lowest reserve capacity to be prioritised.

The overall methodology was divided in six phases. Phase 1 of the process consisted of investigating the bridges using street-view or satellite platforms to determine their general characteristics and condition (location, bridge typology, number of lanes and spans, span lengths, bridge width, pier types, beam width, beam height, noticeable corrosion, spalling, damage, deterioration, crack information). Phase 2 involved conducting site inspection to verify data collected in Phase 1 and record missing information. Phase 3 consisted in consolidating the data collected in the previous steps in a standardised way for its use in the subsequent phases. In Phase 4A loading ratios were estimated on various sections of the structure, and, in particular, critical sections for shear and flexure should be considered, together with sections where a significant interaction between them is expected. In phase 4B reinforcement ratios were estimated. In phase 5 a simplified analysis of the beam using a panel element was conducted to calculate the crack widths from the modified compression field theory. Lastly, phase 6 compared the crack width observed in the field and the maximum one that will cause collapse, defining for each asset a residual capacity ratio that can be used to categories the priority of bridges.

The second important ageing phenomenon is corrosion, that causes a reduction of the effective steel quantities in the system (e.g., reinforcement, steel bearings). In particular, the section reduction of reinforcements bars is mainly caused by the ingress of chloride ions from the concrete surface through the concrete cover to the reinforcing steel. The effect of corrosion is time-dependent and the main parameters that should be determined are the corrosion initiation time and the corrosion rate. A fully probabilistic analysis accounting for variation in seismicity and corrosion parameters ws conducted in a recent study (Ghosh & Padgett, 2010), which leads to time-dependent seismic fragility curves (both for reinforcing steel and steel bearings).

2.2.6 Road network

A road network can be evaluated in terms of not only the structural strength of its individual components, but also on the resilience of the system's connectivity and its ability to quickly recover to pre-disaster state. The road network assessment is divided into a three-part process that consists in defining the seismic hazard at the site, combining it with the fragility of its components and determine the damage consequences. The damage in this case can be evaluated in terms of the system disruption (i.e., increase travel time), time and cost of repairs.

A road network can be modelled with different levels of complexity and different components may be considered vulnerable such as bridges, tunnels, and overpasses. The aim of the INFRA-NAT Project was to assess the risk of the Italian road bridge stock, therefore, only bridges were considered in the analysis.

Several configurations have been used in previous studies to model road networks, ranging from the simple definition of roads as links between locations (used to evaluate possible connectivity), to complex models that account for capacity of the infrastructure and the hourly variations in traffic flows causing congestion. Three different levels of increasing complexity can be considered for the modelling of road networks:

- Level 1 considers only the connectivity of the origin-destination points. The road network is only modelled from basic graphic theory defining points of interest as nodes and roads as lines that are linked using a connectivity matrix;
- Level 2 includes consideration of the network capacity to accommodate traffic flows (possibility of congestion) and travel time changes, in addiction to connectivity considerations;
- Level 3 considers also the indirect economic losses after a disaster that changes the travel demand.

In the INFRA-NAT Project, a level 1 modelling was considered in which the main cities of each case study region were defined as origin-destination points and OpenStreetMap information was processed to connect them with the highways present in each of their road networks. Once this was defined, the lowest distance between any two main cities was computed through an algorithm and using information about speed limits, a travel time matrix was derived (considering an uncongested network). Moreover, it was decided not to consider the full road system to limit the number of links (distance between two adjacent nodes) and reduce the computational time of the analysis; in particular, only the primary highway system was included in the study.

2.2.7 Consequence evaluation

After the definition of the road network model, the identification of the vulnerable components within it and the selection of the seismic hazard for the study region, the consequences of the damage scenario on the performance of the network were evaluated. In previous studies, several options have been considered, which range from simple loss of connectivity to considerations about changes in the traffic flows and indirect costs from delays. Some of these options are summarised in this section.

Regarding the connectivity loss, several indicators of consequences may be considered, depending on the level of complexity of the road network model:

- Simple Connectivity Loss (SCL) measures the average reduction in the ability of links to receive flow from sources;
- Weighted Connectivity Loss (WCL) upgrades the previous indicator by weighting the number of sources connected to the specific link in the seismically damaged network and in non-seismic conditions;
- Moving average and moving standard deviation of SCL and WCL considers the evolution of the expected value and the standard deviation of the performance indicators during the simulation.

Another indicator that may be used to assess the consequences is the terminal reliability (TR) which indicates the probability that a path exists between a specific origin-destination pair. To compute TR, during each run, the values 1 and 0 are assigned to each origin-destination path to indicate respectively if the path still exists or otherwise.

To define the network performance as a whole, Shinozuka et al. (2003) introduced the use of an index called driver's delay (DD) that represents the increase in total daily travel time for all travelers. It was defined as the difference between the total daily travel for all network travelers on the damaged network and the one on the original undamaged network.

Coming back to the INFRA-NAT Project, because of the characteristics of the chosen network model (Level 1), the consequence analysis will not be able to account for estimates of indirect costs based on travel delays of users. However, an index was considered to represent the theoretical delay of a single user travelling in an uncongested scenario to each origin-destination node. In particular, a web-based platform was developed for the project. The tools section includes all the functionalities of calculation, research, and selection, while the map section is where the data are shown. Thanks to further extensions of the platform, it is able to compute the optimal route between two nodes based on the shortest geometry, and to account also for barrier elements to exclude disrupted links from the list of available streets. Lastly, an estimation of the travel time was obtained considering the information regarding the speed limits of each road, leading to the definition of a connectivity matrix. Moreover, for each origin-destination pair, dividing the travel time before an event by the one after the event, an interruption coefficient matrix can be computed. In order to evaluate the overall disruption effect of an event on the entire road network, an index can be computed by averaging all individual origin-destination interruption indexes, leading to an overall Event Interruption Index that can be used to compare the relative effect of different earthquake scenarios on the overall network.

2.3 Summary

In Chapter 2, the two main sources used for the development of this study have been reviewed.

The multi-risk, multi-level approach proposed in the Italian guidelines can be further summarised as follows: starting from a cognitive phase, in which the characteristics and the actual conditions of each structure in the considered portfolio are collected (Level 0, Level 1), the methodology allows to assign them a proper CdA (Level 2), depending on hazard, vulnerability, and exposure parameters for each of the considered risks (structural and foundational, seismic, landslide, and hydraulic). After the risk classification, the guidelines propose diversified actions and possible in-depth analyses to assess the actual performance of the bridges (Level 3, Level 4, Level 5).

A more complex procedure has been developed in the European project INFRA-NAT, in which, contrarily to the guidelines' methodology, the modelling of representative structures in the portfolio (taxonomy definition) is mandatory, together with the modelling of the road network as a whole. In particular, finite element models are necessary to perform non-linear time history analyses, applying seismic excitations compatible with the actual hazard register in the considered region. After the definition of specific performance levels, fragility curves that relate the probability of exceeding a specific damage state to a ground shaking intensity are constructed and consequences are evaluated.

The following chapters will evaluate the application of the Italian guidelines to a case study to investigate their feasibility and identify potential issues, while also evaluating the possible integration of recent research for a more comprehensive assessment of road bridge networks.

3. CASE STUDY DESCRIPTION

This section of the study is dedicated to the presentation of the case study developed in collaboration with IUSS Pavia and Milano Serravalle Engineering company (MSE).

As already mentioned, a bridge portfolio composed of 108 structures have been analysed in order to evaluate the consistency and coherency of the data and the accuracy of coefficient and parameters assigned by the technical. The actual application of the Italian guidelines is reported in the next chapter, while a description of the infrastructures managed by MSE company is reported hereafter.

Before proceeding with the description of the infrastructures, a brief characterisation of the complete national bridge stock is reported to highlight its most frequent features and criticisms.

3.1 ITALIAN BRIDGE STOCK

The Italian road network comprises about 17,000 bridges, whose base information are very fragmented due to the large number of managing authorities that store information on their own structures. In fact, the national authority that owns all state roads and bridges (ANAS), in recent years has devolved many state roads to regions and provinces. Moreover, highways have been privatised and split in no less than 26 concessionaires.

Most bridges were built in the 1960s and 1970s and almost exclusively in reinforced or pre-stressed concrete (in particular, more than 90% of the structures are Reinforced Concrete (RC) girder bridges). Therefore, a big part of the set is composed by aged infrastructures, which may have already exceeded their service life.

3.1.1 Typical bridge categories

The Italian bridge stock is composed of different categories of bridges; the main ones are:

- Reinforced concrete arch bridges;
- Reinforced concrete tied-arch bridges;
- Gerber bridges.

Moreover, mixed steel-concrete bridges and reinforced and prestressed concrete isostatic bridges are also present.

Reinforced concrete arch bridges

Even though masonry arch bridges in Italy date back to the Romans, the first reinforced concrete arch bridges were built only in recent times (end of the 19th century), and they started to spread in the 1950s and 1960s, during the construction of the road networks.

Arch bridges are characterised by the deck located completely above the arch. The area between these two elements is called spandrel and may be either solid (closed-spandrel deck arch bridge) or composed of vertical or subvertical elements that support the deck (open-spandrel deck arch bridge). This type of bridge is typically used when there is the necessity to cross a deep gorge or when the construction of a girder bridge is complex.

Figure 4 - Reinforced concrete arch bridge example

In the classical conception, an arch bridge is characterised by a very rigid arch with respect to the deck, leading to the absorption of very high actions by this element (almost all the bending moment and shear). In a sub-category of arch bridges, called Maillart bridges, the arch is replaced by a thin and wide vault. In this case, the inertia associated to the vault is lower than the one associated to the deck which, contrarily to the classical scheme, is very rigid (the flexural stiffness of the deck is generally about 50 times that of the vault). The reversal of the classical stiffness ratio between the elements of the bridge aims to alleviate the arch from bending moments, which are sustained by the deck. Therefore, in Maillart bridges the deck does not limit to transfer the loads to the substructure, but it has also a load-bearing function. However, in both typologies the trust absorbed by the arch is transferred directly to the ground, which should be sufficiently stiff to also resist its horizontal component. The horizontal trust increases as the height of the arch decreases.

The shape of the arch, or vault in case of Maillart bridges, is chosen to be the funicular of all dead loads (arch and superstructure self-weights) such that the element is subjected to a centered normal stress in each section and the ideal operating conditions is achieved. The presence of further accidental loads (e.g., traffic loads) introduces bending moments and shear actions that, as already mentioned, will be sustained mainly by the arch or the deck depending on the relative stiffness of the two members. In Maillart bridges, since the deck is more rigid than the vault, the latter is basically a funicular also when the structure is subjected to accidental loads.

Maillart bridges have both advantages and disadvantages with respect to classical arch bridges. On one side, the introduction of a thin element decreases axial shortening phenomena due to the vault self-weight. Moreover, the overall costs of foundations and formworks decreases, while the bending moments and shear actions are mainly sustained by the superstructure. Lastly the aesthetic qualities of the structural system improve. On the other side, viscous phenomena lead to increasing stresses due to global dead loads acting on the system. Moreover, the effects induced in the structure by thermal variations are more severe than in classical arch bridges since they are concentrated at the deck level, where no compression stresses that can reduce these effects are present.

The arch is a structural element that works mainly because of its shape; therefore, the construction phase is of foremost importance. In general, the construction process can be divided into two main typologies: in situ construction and off-site construction and subsequent in situ assembly. In general, for arch bridges, the first construction technique is adopted. Temporary elements are needed to allow the construction of an arch, leading to an increase in the costs of realisation with respect to other structural types. Typically, the construction of arches is achieved using a rib, above which blocks (if masonry is used) or formworks (if concrete is used) are positioned. The rib can be re-used if multiple arches are present in the same bridge. The deck is constructed only after the realisation of the arch and of the piles. A decrease in the costs is registered for Maillart bridges also concerning the temporary structures.

Reinforced concrete tied-arch bridges

The first reinforced concrete tied-arch bridges were built in Italy in the 1920s and 1930s, in the postwar period, during the reconstruction. At first, this structural type didn't spread due to the high costs of construction, which however were reduced later thanks to new building techniques.

Contrarily to the previous case, in tied-arch bridges the arch is located completely above the deck. This structural system is typically used when the clearance height below the superstructure is limited (e.g., presence of vessel or highway traffic) or when the mechanical characteristics of the soil are not adequate to sustain horizontal trusts. Also in this case, vertical or subvertical elements are used to connect the deck to the arch.

Figure 5 - Reinforced concrete tied-arch bridge example

In this structural type the arch and the deck form a combined system in which both elements have a load-bearing function. The direct consequence of this collaborating system is a reduction of the elements' dimensions and, therefore, of the self-weight of the whole structure. The vertical component of the arch' trust is transferred to the substructure, while the horizontal one is absorbed by the deck. In particular, the arch works in compression, while the deck is subjected to a tension force. To achieve this collaboration between the elements, the static scheme must be isostatic; otherwise, the horizontal component of the arch' trust would be absorbed by the abutments and the deck would be unloaded.

Tied-arch bridges can be divided into different categories depending on both material and geometrical characteristics, considering:

- Arches configuration (central or at the edges);
- Vertical elements material (reinforced or prestressed concrete, steel);
- Vertical elements configuration (vertical, inclined, radial);
- Brace system configuration (spacing, section properties).

Most tied-arch bridges in Italy are composed of two arches located at the sides of the deck, connected by an orthogonal bracing system, and with reinforced concrete ties. The most common arch height-span length ratio is lower than 0.35 and the sections of both the arch and the bracing system is typically rectangular. The ties spacing varies from 2 to 4 metres, depending on the length of bridge and the loading conditions. Vertical ties generally have a rectangular cross-section with different length ratios between the edges, which typically have values around 2.

As in the case of arch bridges, initially the stiffness of the arch was greater than the one of the deck, but subsequently the stiffness ratio was reversed so that the arch was subjected almost exclusively to compression stresses. For what concerns the construction phase, also in this case typically it is carried out in situ with the use of a rib. Contrarily to the previous typology of bridges, in tied-arch systems the arch is constructed only after the construction of the deck, on which the rib is positioned. Once the arch is completed, the ties are manufactured.

Gerber bridges

Figure 6 - Gerber bridge example

The origins of Gerber bridges date back to the 19th century, when engineers understood that a continuous bridge distributes its loads to the supports, decreasing the beam actions. In this way it was possible to increase the span length and construct longer bridges. Initially, the material that was mainly associated with this structural scheme was steel, but later reinforced and prestressed concrete were also used. In particular, in Italy, prestressed concrete Gerber bridges are the most common system within this category, as they are particularly suitable for the in-situ assembly of precast elements.

Gerber bridges belong to the category of articulated bridges. In this structural type a continuous deck is subdivided into several sections separated from each other by hinges that can be located either at the ends or inside the beam (in this case parts of the beam are cantilevered from the supports). Typically, the insertion of these disconnections results in a completely isostatic system, simplifying the design process. However, in some cases flexural continuity between the deck and the piers is registered close to the connections between the superstructure and the substructure. In particular, the most common configurations are reported in Figure 7.

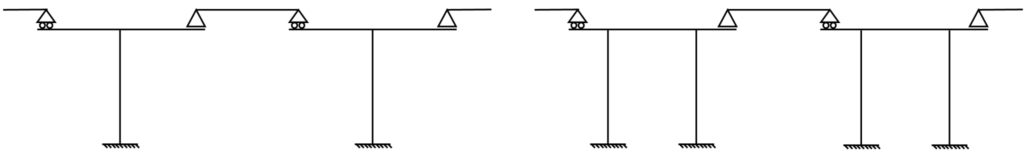

Figure 7 – Typical gerber bridge configuration

In general, Gerber bridges have both advantages and disadvantages. As already mentioned, the possibility of having a completely isostatic structure simplifies substantially the design process, also eliminating the effects of thermal variations that under this condition don't induce additional stresses in the members. Moreover, it is possible to increase the overall length of the bridge, without resorting to more complex and costly structural schemes. On the other hand, the presence of multiple beam joints decreases the drivers' comfort and requires continuous maintenance. In addition, excessively high bending moments may be registered at the mid-section of each span. Lastly, one of the main issues is related to the lack of redundancy of the load path, which may lead to the collapse of the system.

For what concerns the construction process, Heinrich Gerber developed the cantilever and suspended technique, which does not include the use of formworks, but of precast elements that are assembled in-situ. In particular, it allows the construction of the system from both directions starting from adjacent piers (cantilevered beams), and the completion of the span with a suspended beam of limited length that will be connected to the extremities of the cantilevers. Moreover, if multiple spans are present in the bridge, to maintain equilibrium of the cantilevered beams usually these elements are constructed simultaneously on both sides of a pier.

3.1.2 Common criticisms

Degradation phenomena are associated to environmental effects and can be increased by incorrect design of construction details, low quality of materials and inadequate maintenance. In particular, environmental effects can be divided into:

- Physical actions (freeze-thaw cycles, water infiltration, thermal variations, accidental damage caused by impacts);
- Chemical actions (carbonation, corrosion, salt action, alkali-silicate reactions, sulphate attack);
- Biologic decay (biological colonisation).

The majority of concrete bridges in Italy presents the problem of concrete spalling, which may lead to severe deterioration of reinforcing steel. The process that causes this phenomenon begins with the penetration of chlorides, water, and oxygen into the concrete, until they reach the reinforcing steel, which oxidate. The oxidation causes an increase in the volume of the bars which induces tensile forces in the concrete, and the subsequent cracking and expulsion of the cover.

Among physical actions, freeze-thaw cycles are the most common cause of deterioration, particularly when highly porous materials are used. As aforementioned, the increase in volume leads to the development of tensile stresses in the concrete and, therefore, to the formation of micro-cracks and the possibility of cover expulsion.

Another cause of deterioration is the inadequacy of the water disposal system or the absence of a waterproofing layer. Lastly, viscous phenomena such as concrete shrinkage and steel relaxation may cause structural damage if not appropriately included in the design phases.

3.2 INFRASTRUCTURES MANAGED BY MSE

The infrastructures managed by MSE company belongs to the Milano-Serravalle motorway section located in Lombardy region of Northern Italy. In particular, A7, A51, A52, A53, A54 motorways and Milan's external ring road.

The total number of structures included in the project is 108 and three macro-categories can be distinguished as:

- Reinforced concrete and/or prestressed concrete bridges;
- Steel bridges;
- Mixed reinforced concrete and/or prestressed concrete and steel bridges.

Specifically, the percentage of infrastructures belonging to each category is respectively 43%, 1% and 56% of the considered sample.

The global area of intervention of the company is shown in Figure *8*.

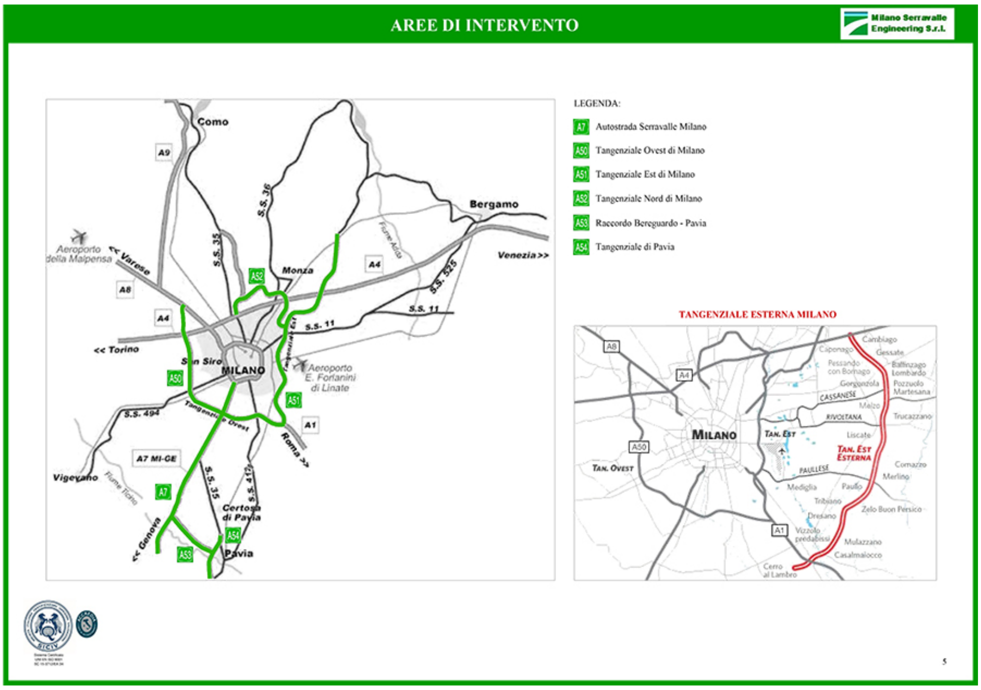

Figure 8 - Area of intervention of MSE Company (Adopted from www.msengineering.it)

Most of the structures in the sample belonging to the category of reinforced concrete and/or prestressed concrete bridges are characterised by the static scheme "simply supported beam". This type of solution is well-suited to the design requirements, since it is mainly found in structures with particularly small spans. The typical cross-sections of longitudinal and transversal beams are rectangular.

The mixed reinforced concrete and/or prestressed concrete and steel bridges in the sample typically present the static scheme "continuous beam on multiple supports" with three spans. In particular, the central span is typically much longer than the external ones. Considering the substructure, most of the bridges belonging to this category is characterised by two groups of piers, each with a number of elements varying between 2 and 5. No information is available concerning the foundations. The deck consists of a steel skeleton and a reinforced concrete and/or prestressed concrete slab; in particular, the skeleton consists of double-T principal beams, solid and trussed transverse beams and a bracing system, the purpose of which is to further strengthen the structure and avoid excessive deformation of the steel sections.

In Figure ***9*** and Figure ***10*** an example for both categories of bridges are reported.

Figure 9 - Example of R.C. bridge of the MSE portfolio

Figure 10 - Example of mixed R.C. and steel bridge of the MSE portfolio

3.3 Summary

In the first section of Chapter 3, the main characteristics of the Italian national bridge stock have been investigated, together with the most common criticisms registered for these typologies.

Three main categories of bridges can be distinguished: reinforced concrete arch bridges, reinforced concrete tied-arch bridges, and Gerber bridges.

Regarding the criticisms, the majority of concrete bridges in Italy present the problem of concrete spalling, which may lead to severe deterioration of reinforcing steel via corrosion. If highly porous materials are used, freeze-thaw cycles are the most common cause of deterioration, leading to micro-cracks formation and the possibility of cover expulsion. Other causes of deterioration are the inadequacy of the water disposal system or the absence of a waterproofing layer and viscous phenomena such as concrete shrinkage and steel relaxation, which may cause structural damage if not appropriately included in the design phases.

The second section of the chapter concentrates on the description of the infrastructures managed by Milano Serravalle Engineering S.r.l. company (MSE). Its area of intervention is shown in Figure 8, and three categories can be distinguished: reinforced concrete and/or prestressed concrete bridges, steel bridges, and mixed reinforced concrete and/or prestressed concrete and steel bridges (even though, steel bridges represent only 1% of the whole portfolio).

Reinforced concrete and/or prestressed concrete bridges are characterized by the static scheme "simply supported beam", whereas mixed reinforced concrete and/or prestressed concrete and steel bridges typically present the static scheme "continuous beam on multiple supports" with three spans.

4. APPLICATION OF GUIDELINES

In this chapter, the methodology described in the Italian guidelines is applied to the bridge stock managed by MSE. The first two levels of the approach, namely the census of the structures and the visual inspections and surveys, were already carried out by MSE technical staff. However, all the data available have been checked comparing all the photographs to the values assigned by the technical to the parameters G, k_1 and k_2, in accordance with Annex C of the guidelines. In this way, it was possible to highlight inconsistencies with the annex and between the same defects registered in different structures by different technicians. Hereafter, the defects detected in the structures of the stock will be illustrated. In particular, both the defects associated with the construction material and the general ones will be considered.

4.1 DETECTED DEFECTS

4.1.1 Defects associated with steel or metal

Welding defects

Welding defects are located along the weld length. They are particularly dangerous as they reduce the strength and toughness of the welded joint and over time, they can cause the weld fracture. The origin of these phenomena can be traced back to errors or problems arising during the welding process itself, such as the choice of unsuitable materials and/or techniques, unfavourable welding conditions, and poor welding skills. For this reason, this defect is assigned a G value of 4. An example is provided in Figure ***11***.

Figure 11 - Welding defect

Peeling paint

Peeling paint consists in the detachment of the protective paint from the steel elements, with the resulting in metal exposure. The defect is mainly due to:

- Poor execution due to the use of unsuitable products and incomplete or incorrect layer application;
- Ageing of the paint layer;
- Chemical attack (e.g., by chlorides);
- Damage caused by vehicle impacts;

The occurrence of this phenomenon is favoured in presence of humidity and aggressive agents, such as those found in marine or industrial environments. If not promptly and properly repaired, it can lead to corrosion and oxidation of structural elements. This defect is assigned a G value of 2. An example is provided in Figure ***12***.

Figure 12 - Peeling paint

Bolting defects

Bolting defects include the incorrect conformation of the nail head and the absence of one of the following elements: the head, the nut, or the entire nail. They originate from:

- Incorrect conformation and dimensioning of the nailed joint;
- Incorrect nail manufacturing;
- Excessive stress.

Bolting defects occur more easily where localised corrosion or fatigue phenomena are present. They are assigned a G value of 5 since such defects can lead to loss of effectiveness of the connection and consequent failure of the entire structure. Moreover, sometimes they are associated to web/flange deformation of the connected elements. An example is provided in Figure ***13***.

Figure 13 - Bolting defect

Web/Flange deformation

This defect is located in the web and/or in the flanges of open sections (T, double T, L, C, etc.) and causes the loss of shape of the steel profiles. The defect is generally caused by landslides or the impact of vehicles, vessels or other material transported by the water flow. Further causes are:

- Deformations caused by incorrect construction tolerances;
- Fire;
- Advanced stages of corrosion;
- Unforeseen concentrated loads or local instability phenomena.

The G value assigned to this defect is 3. An example is provided in Figure ***14***.

Figure 14 - Flange deformation

Joint damage

This defect comprises damages located both in the connections (i.e., welds, bolts) and in the connected structural elements. Joint damage is frequently caused by:

- Incorrect conformation and sizing of the joint;
- Unforeseen loads or loads higher than the ones considered in the design phase;
- Fatigue and corrosion phenomena.

It is particularly dangerous for the stability of the structure, and it is assigned a G value of 5. An example is provided inFigure 15.

Figure 15 - Joint damage (Adopted from Annex C of the Italian guidelines)

Corrosion

Corrosion is the result of material coming into contact with water and humidity. This defect has several causes:

- Deterioration of the metal protection layer;
- Presence of humidity or water stagnation;
- Poor maintenance;
- Presence of stray currents or saline solutions (e.g., marine environment, antifreeze solutions, industrial environment).

As the process evolves, the section progressively reduces until the metal is perforated. It is assigned a G value of 4. An example is provided in Figure ***16***.

Figure 16 – Corrosion

Oxidation

An earlier stage of the corrosion phenomenon, oxidation is caused by the reaction between the iron contained in the steel and the atmospheric oxygen. Depending on the stage of evolution of the phenomenon, it appears as:

- Homogeneous surface oxidation;
- Swelling of the external surface;
- Spot corrosion.

The causes of this phenomenon are the same illustrated in the previous defect. It is assigned a G value of 2. An example is provided in Figure ***17***.

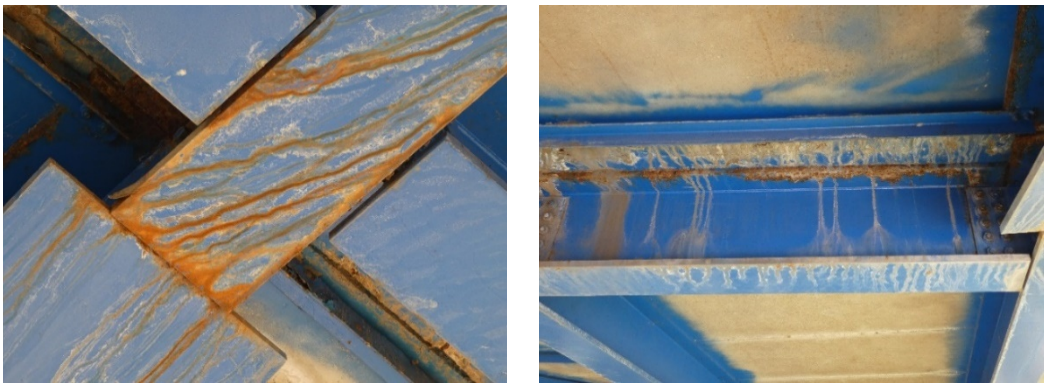

Figure 17 – Oxidation

4.1.2 Defects associated with reinforced and prestressed concrete

Passive moisture stains

The defect is visible as areas of different coloring from the intact material appear. In particular, these areas are traces of calcium released onto the surface by moisture penetrated through the concrete. Moisture and rainwater penetration through the material is favoured in the presence of:

- Highly porous materials;
- Lack of or deficiencies in the waterproofing system;
- Absent, inadequate or damaged water conveyance systems;
- Imperfect sealing of joints;
- Missing or damaged flashings.

The progression of this phenomenon could lead to the initiation of concrete deterioration phenomena. The stains are typically whitish in color and the defect is assigned a G value of 1. An example is provided in Figure ***18***.

Figure 18 - Passive moisture stains

Active moisture stains

The defect is visible as areas of different coloring from the intact material appear. In particular, these areas are traces of calcium released onto the surface by moisture penetrated through the concrete. Contrarily to passive moisture stains, active moisture is linked to ongoing water infiltration phenomena, and it is characterized by dark stains due to continuous contact with water and humidity. The same factors reported in the previous section also facilitate the formation of active moisture stains, but in this case the progression of the phenomenon can lead also to concrete spalling. The defect is assigned a G value of 3. An example is provided in Figure ***19***.

Figure 19 - Active moisture stains

Deteriorated concrete

Deterioration of concrete is due to percolation of surface water and occurs mainly on the vertical or inclined surfaces of the elements. The defect refers both to erosion of the surface layer of material due to the frequent passage of water, and to swelling of the concrete surface, spalling, loss of cohesion, etc. Deteriorated concrete represents the evolution of other defects such as moisture stains (both passive and active) and drainage marks (which will be discussed in the section "General defects") and, as it progresses, it can cause a reduction of the concrete resisting section. This defect is assigned a G value of 3. An example is provided in Figure ***20***.

Figure 20 - Deteriorated concrete

Gravel nests

This defect compromises the surface continuity of concrete resulting in non-homogeneous areas due to the exposure of concrete aggregates. It may affect extensive areas of the structural elements or be localised. Generally, the causes of this phenomenon can be traced back to problems during construction: gravel nests are frequent in the case of poorly made concretes, with inadequate grain size and/or mix, in the case of poorly made casts, with insufficient vibratory, or poorly sealed formworks. In the vicinity of gravel nests a considerable increase in the permeability of the concrete is registered due to the presence of voids between aggregates, which facilitates the penetration of aggressive agents. They are, therefore, often associated with other degradation phenomena, such as oxidation and/or corrosion of the longitudinal and transversal reinforcement. The G value assigned to the defect is 2. An example is provided in Figure ***21***.

Figure 21 - Gravel nests

Concrete spalling

This defect refers to the absence of portions of the concrete covering layer of the longitudinal and transverse reinforcements of the elements, with consequent exposure of the latter to oxidising and corrosive agents.Concrete spalling is due to chemical and physical phenomena, such as the deterioration of the concrete caused by the action of water and humidity, and carbonation, favoured by the presence of porous concretes in aggressive environments, but also to errors during execution, such as very limited thicknesses of concrete, or accidental causes, such as vehicles impacts. Generally, this phenomenon occurs where concrete appears deteriorated or at gravel nest's location. It is assigned a G value of 2. An example is provided in Figure ***22***.

Figure 22 - Concrete spalling

Rusted and/or corroded reinforcement

Reinforcement steel rusts and/or corrodes mainly due to the absence or deficiency of a suitable concrete cover. The absence of the cover layer allows external aggressive agents to come into contact with steel. If the concrete is carbonated and the concrete cover intact, this phenomenon is not visible but develops below the concrete cover. Carbonation means calcium carbonate formation with the consequent reduction in the PH of concrete and the formation of an iron oxide layer around reinforcement bars, which facilitates their oxidation. The latter causes an increase in the steel volume and subsequent cracking, first, and then expulsion of concrete cover. The defect is assigned a G value of 5 because in the worst cases, it can cause a reduction in the reinforcement cross-section. An example is provided in Figure 23.


Figure 23 - Corroded reinforcement

Minor cracks

The defect refers to the presence of small cracks irregularly distributed on the surface of the elements and clearly visible in areas particularly subjected to the effects of humidity. In general, the causes of this phenomenon are linked to shrinkage in the case of inadequately mixed concrete and, in particular, when an excessive amount of water is used in the mix. Moreover, minor cracks can also occur in the case of inadequately cured casts or when the superficial reinforcement is insufficient. It is assigned a G value of 1. An example is provided in Figure ***24***.


Figure 24 - Minor cracks

Cracks

Annex C of the guidelines distinguishes five categories of cracks: vertical, horizontal, diagonal, longitudinal, and transversal cracks. The presence of cracks facilitates the infiltration of water and aggressive agents through the material, which could lead to other defects such as moisture stains and deteriorated concrete.

Vertical, horizontal, and diagonal cracks result, in general, from abnormal stresses leading to concrete failure in the weakest sections of the elements. When vertical cracks occur in abutments and piers, they can be caused by concrete shrinkage effects: in this case they occur at regular intervals with limited width. Otherwise, if the cracks have a significant and not constant width along their path and are isolated, they are caused by foundation settlements or differential soil thrusts. In general, if the defect occurs in structural elements it is potentially dangerous. Regarding horizontal cracks, they are favoured by inadequate reinforcement linking successive casts, large spacing between bars, or even in the case of different quality between successive casts. Lastly, in the case of diagonal cracks affecting elements such as abutments and piers, their causes are, once again, foundation settlements or differential soil thrusts (same cracks characteristics as in the case of vertical cracks). Generally, diagonal cracks in beams and pier caps are found at maximum shear location or where reinforcement bars change their curvature. In these cases, they have structural origin and are caused by excessive stresses and/or reinforcement deficiencies. Similar considerations can be made if the cracks occur on horizontal surfaces, such as slabs.

Longitudinal and transversal cracks, generally, result from errors during design and construction phases. In fact, their presence is often caused by a lack of transversal reinforcement or large spacing between bars, if not by poor quality concrete. Other more specific causes may be concrete shrinkage or foundation settlements.

Vertical, horizontal, and longitudinal cracks are assigned a G value of 2, whereas diagonal and transversal cracks are assigned a G value of 5. An example is provided in Figure *25*.

Figure 25 – Cracks

Deteriorated reparations

This defect comprises concrete localized interventions such as void filling, concrete cover reconstruction, and gravel nests reparation. Deteriorated reparations include several defects: cracks, moisture phenomena and detachment of new elements from the old material. This phenomenon can be caused by several factors, including unsuitable repair materials, incorrect design or execution of retrofit intervention, presence of aggressive external agents (e.g., freeze/thaw cycles, carbonation, etc.), and failure in preventing deterioration phenomena to affect the structure. The defect is assigned a G value of 1. An example is provided in Figure ***26***.

Figure 26 - Deteriorated reparations

Crush lesions

The term "crush lesions" refers to cracks generally located at the supports position, typically inclined by 45°. They are caused by excessive compression and the phenomenon is often associated with the detachment of wedges of material. Excessive compression may result from design errors (e.g., incorrect element sizing) or construction errors (e.g., poor materials, lack of reinforcement). The presence of cracks facilitates the infiltration of water and aggressive agents through the material, which may lead to moisture stains, deteriorated concrete, and reinforcement oxidation/corrosion. The defect is assigned a G value of 4. An example is provided in Figure 27.

Figure 27 - Crush lesions

Cracks at the stirrups position

The defect is characterised by regularly distributed cracks reproducing the stirrups arrangement in structural elements. The presence of cracks at the stirrups position is due to oxidation of the reinforcement surface layer, the increase in volume of which causes concrete cover to crack. This phenomenon is favoured in the case of small thickness of the concrete cover and very porous concrete in aggressive environments, where air and moisture penetrate more easily. A further cause could be the effects of concrete shrinkage. It The development of the phenomenon could cause the complete detachment of the concrete cover and the consequent exposure of the transverse reinforcement which, exposed to atmospheric agents, could be subject to rusting and/or corrosion. The defect is assigned a G value of 2. An example is provided in Figure ***28***.

Figure 28 - Cracks at the stirrup position

Rusted and/or corroded stirrups

This defect refers to the exposure of the transverse reinforcements to air and water in the external environment and their consequent rusting/corrosion. The causes of the defect are concrete spalling and cracks at the stirrup position, which allows external agents to come into contact with steel. It is assigned a G value of 3. An example is provided in Figure ***29***.

Figure 29 - Rusted and/or corroded stirrups

Ruptured stirrups

The defect refers to the complete failure of the transverse reinforcement of the structural elements. Continuous exposure of stirrups to external agents following concrete spalling can lead to rusting and, in the most serious cases, to a reduction in the bars' cross-section up to complete failure. The defect is assigned a G value of 4. An example is provided in Figure ***30***.

Figure 30 - Broken stirrups

4.1.3 Defects associated with supports

Deformed baseplate

The defect relates to the loss of shape and flatness of the steel baseplate of the supports. The origin of the defect can be traced back to the assembly process, if the defect is due to surface irregularities or incorrect positioning of the equipment, or during service life, if it is due to abnormal or unexpected movements and in the case of severe deterioration of the plate itself. It is assigned a G value of 2. An example is provided in Figure ***31***.

Figure 31 - Deformed baseplate

Rusting of supports

The defect consists in the rusting of the steel components of the support. Depending on the progress of the phenomenon, it can manifest in various forms, ranging from the mere perforation of the surface coating layer to the reduction of the element' section. Rusting is triggered if the steel parts of the devices are not adequately protected (e.g., lack of paint, paint deterioration). The phenomenon is intensified in a humid environment. Other causes include stray currents or aggressive chlorides from de-icing salts, marine environment, etc. It is assigned a G value of 2. An example is provided in Figure ***32***.

Figure 32 - Bearing device rusting

Locking

The defect refers to incorrect, or other than intended, functioning of the supports. The occurrence of this phenomenon is related to the preservation characteristics of the devices. It is more likely in the case of deteriorated or aged materials or if the devices are poorly positioned. Other possible causes are the presence of debris, which blocks the movement and foundation movements. It is assigned a G value of 4. An example is provided inFigure *33*.

Figure 33 - Locking

Incorrect presetting

The defect refers to incorrect, or other than intended, functioning of the supports due to incorrect presetting. The origin of the defect is to be found in the installation phase, as it is due either to incorrect original positioning or to positioning without taking into account the long-term phenomena that may affect their operation (e.g., temperature variations, concrete viscosity, etc.). It is assigned a G value of 4. An example is provided in Figure *34*.

Figure 34 - Incorrect presetting

Presence of debris

This defect refers to the presence of material deposits (e.g., earth, mud, wooden planks, bitumen, etc.) in the vicinity of the supports that affects its normal functioning. Debris generally results from unremoved manufacturing residues or weathered materials deposited on the horizontal surfaces of abutments and pier caps. It is assigned a G value of 2. An example is provided in Figure 35.

Figure 35 - Presence of debris

Crushing

Annex C of the guidelines distinguishes between two categories of crushing, depending on the typology of bearing device considered: lead plate supports, and neoprene supports. In the first case, the defect may manifest as plate deformation or as a change in its position due to, for example, sliding. Crushing is caused by inadequate slab size, insufficient adherence to the supports during deck movements, or by incorrect original positioning. In the case of neoprene supports, the causes of the defect are:

- Presence of greater vertical forces than those foreseen;
- Incorrect dimensioning of the device or manufacturing defects.

The defect is favoured by ageing and the consequent loss of elasticity of the material. Both defects are assigned a G value of 4.

Neoprene/Teflon deterioration

This defect comprises all the degradation phenomena typical of the material which reduce the supporting device functionality (e.g., cracks, leakage, detachment, crushing, ageing, etc.). If neoprene supports are considered, the phenomenon is related to the age of the devices and their degraded conditions due to usage. The defect occurrence is accentuated by installation errors and by aggressive agents from the external environment (e.g., ultraviolet rays). On the other hand, if teflon devices are considered, the defect can be caused by dust intrusion, unintended loads, poorly executed assembly or the use of materials with unsuitable characteristics and qualities. The defect is assigned a G value of 3. An example is provided in Figure *36*.

Figure 36 - Neoprene/Teflon deterioration

Excessive lateral deformation of neoprene

The phenomenon relates to excessive sliding of the bearings in the horizontal plane, along the transverse or longitudinal axis of the road surface. The causes of such sliding may be:

- Presence of horizontal thrusts or internal friction between neoprene and unforeseen plates;
- Incorrect sizing of the device or manufacturing defects.

The defect is favoured in the case of ageing and consequent loss of elasticity of the material. It is assigned a G value of 4. An example is provided in Figure *37*.

Figure 37 - Excessive lateral deformation of neoprene

4.1.4 General defects

Drainage marks

This defect is visible as areas of different coloring from the intact material appear. In particular, they are caused by the repeated passage of rainwater over the surface of the elements. The presence of drainage marks is due to the lack or inadequacy of the rainwater conveyance system, as well as deficiencies in the waterproofing system. Similar consequences are associated with non properly manufactured or maintained joints and problems related to other construction details, such as the absence of drip traps. It is assigned a G value of 3. An example is provided in Figure *38*, on the left.

Water stagnation

The defect occurs when a significant quantity of water accumulates in specific areas of structural elements. This phenomenon can induce degradation phenomena and corrosion of the elements. Water stagnation occurs where there are irregularities on the surfaces and in the case of poor maintenance of the water drainage system and lack or insufficient waterproofing of the slab. The phenomenon is accentuated in the presence of particularly porous and permeable concrete. Annex C distinguishes the case in which water stagnation verifies inside box sections. If the latter is not the case, the defect is assigned a G value of 2, otherwise it is assigned a G value of 4. An example is provided in Figure *38*, on the right.

Figure 38 – Left: Drainage marks; Right: Water stagnation.

Impact damage

This phenomenon refers to the damage caused by the impact of vehicles, vessels or materials transported by watercourses with structural elements.

Such damages comprise breaking of the edges, detachment of the concrete cover, and reinforcement cut or deformation. The impact of elements of various kinds with the structure can be caused by the passage of vehicles that do not comply with the clearance height under the bridge, or by traffic accidents. The defect is assigned a G value of 4. An example is provided in Figure ***39***.

Figure 39 - Impact damage

Characteristic cracks at the support location

The defect refers to crack formation at the support location due to the bearing device functioning. Cracks can occur on both the supporting element and the supporting device. Cracks form as a result of tensile stresses arising from the failure in sliding of the device, which in turn can be caused by material deterioration, blockage or incorrect dimensioning. The defect is assigned a G value of 3. An example is provided in Figure ***40***.

Figure 40 - Characteristic cracks at the support location (Adopted from Annex C of the Italian guidelines)

4.1.5 Defects associated with expansion joints

Pavement-expansion joint gap

The defect refers to the difference in height between the extrados of the pavement and the joint. The height difference between the elements can be caused by errors during installation, if the joint is not correctly inserted or if pavement is too much compacted. However, if this is not the case, this defect may indicate support settlements and foundation movements. It is assigned a G value of 1. An example is provided in Figure ***41***, on the left..

Gap plug detachment

This defect is typical of joints where the continuity element is represented by a layer of poured asphalt. It occurs when cracks between the gap plug and the pavement appear, and it can be caused by:

- Incorrect choice of joint typology;
- Choice of materials with insufficient elasticity;
- Incorrect manufacturing of the joint;
- Deterioration of the materials.

Sometimes the presence of cracks can be indicative of foundation movements. The defect it is assigned a G value of 1. An example is provided in Figure ***41***, on te right.


Figure 41 – Left: Pavement-expansion joint gap; Right: Gap plug detachment (Adopted from Annex C of the Italian guidelines)

Screed damage

Screeds are the supporting elements of the joint continuity components, namely the elements that form the connection between the latter and the pavement slab. The defect must be reported if there are transverse cracks on either one or both screeds. Cracking is mainly due to repeated impacts caused by vehicles driving over the joint. It is favoured in the case of unsuitable materials or poor installation. The first detachments may also be caused by shrinkage phenomena that naturally occur in mortars. Lastly, they are amplified by impact between passing vehicles. The defect is assigned a G value of 2. An example is provided in Figure ***42***, on the left.

Gap plug deformation

Defect characterised by irregularities on the gap plug surface. The same causes reported in the previous defect are also valid in this case. The defect is assigned a G value of 1. An example is provided in Figure ***42***, on the right.

Figure 42 – Left: damaged screeds; Right: gap plug deformation (Adopted from Annex C of the Italian guidelines)

Deformation/breakage of continuity elements

This defect refers to the presence of surface irregularities or, in more serious cases, cracks, breaks or missing portions on the continuity elements of the joints. Deformation and/or breakage of continuity elements is due to material deterioration or incorrect installation. Sometimes it may be indicative of foundation movements. The defect is assigned a G value of 2.

An example is provided in Figure 43.

Figure 43 - Deformation/breakage of continuity elements (Adopted from Annex C of the Italian guidelines)

Permeable or absent flashing

The flashing is a retention element that prevents the passage of water across the joint. The defect occurs when it is absent, or its functioning is inadequate. The main cause of damage to flashings is material consumption, which, by losing their elasticity, becomes damaged and breaks. The defect is therefore favored in the case of poor-quality materials and if thickness is low. Other causes may be abnormal movement of the joints or incorrect installation. The defect is assigned a G value of 2. An example is provided in Figure ***44***

Figure 44 - Permeable or absent flashing (Adopted from Annex C of the Italian guidelines)

4.2 CRITICISMS

At the end of the defect sheets revision, several problems, common to the considered bridge portfolio, were identified.

One of the most critical problems encountered in almost all the structures reviewed concerns the location of the bridge elements to which the sheets refer. Often the sheets refer to elements (e.g., piers) on the “right” or “left”, but it is not clear whether this indication refers to the structure prospect or not. Although this issue is of minor importance in structures with low to medium attention classes (CdA), the unambiguous location of the elements becomes particularly important for structures with medium-high or high CdA, since in these conditions it is crucial, in order to improve the functionality and safety of the bridge, to carry out accurate Level 4 analyses and to model the structure properly. In addition, the same sheet often refers to groups of elements (e.g., groups of piers, supports, etc.). In these cases, it is not always possible to identify through photographic evidence which element of the group has certain defects. Again, errors may be introduced at the modelling stage.

A second critical point registered in the revision of the defect sheets concerns the identification of the photographic evidence relating to the specific elements and/or defects: only in 30% of the cases the photographs to be considered for a given defect are indicated in the "Notes" section of the sheets. In the absence of such an indication, it is particularly difficult to distinguish, for example, a photograph referring to one pier rather than another.

The additional problems that emerged from the review of the sheets are less frequent than those reported above. In particular, these can be summarized as follows:

- Inaccuracies in the definition of parameter G;
- Inaccuracies in the definition of parameters k_1 and k_2;
- Absence of photographic evidence and/or sufficient information to review the sheets.

Precise indications for the definition of parameters G, k_1 and k_2 are provided in Annex C published in conjunction with the Guidelines. In particular, the value of parameter G is uniform for each defect, while k_1 and k_2 are, in some cases, defined and in others variable (the technician must assign a value between 0.2, 0.5 and 1 depending on the severity of the defect). In the latter case, "inaccuracies" means both the assignment of an incorrect value in the case of a uniform solution, and the assignment of a value which differs from the severity of the defect supported by the photographs in the case of a variable solution.

Finally, considering the last critical point listed, it should be noted that in many cases it is difficult to verify the values of the parameters if the photographs of the defects are not accompanied by reference scales that facilitate the determination of their dimensions (e.g., cracks width).

The criticisms reported above are specific for Level 1 of the guidelines, however a more general consideration can be made: the process of collecting Level 0 information and integrating them with Level 1 visual inspections is very laborious and time consuming, since it requires the retrieval of origin-design documentation and the analysis of hundreds photographic evidence for each structure in the portfolio. Moreover, the next step of the multi-level approach described in the MIT document requires the definition of hazard, vulnerability, and exposure parameters for each single bridge structure in the portfolio, which renders the procedure even more time-consuming. The procedure can be considerably shortened by the definition of taxonomies, as illustrated in Chapter 2, section 2.2.2. In MSE case study, instead of analyzing 108 bridges, with the introduction of taxonomies, just two principal structural typologies can be distinguished: reinforced concrete and/or prestressed concrete bridges and mixed reinforced concrete and/or prestressed concrete and steel bridges. Therefore, the effectiveness of the guidelines can be highly increased by the integration of recent research approaches, such as the INFRA-NAT Project.

Another consideration concerning the effectiveness of the Italian guideline is that, since the bridges belonging to the same structural typology present more or less the same conditions, the possibility that, at the end of Level 2 of the methodology, most of the structures are classified with the same CdA is very high when the bridge portfolio considered include structures located in a restrict region, that present the same hazard. This means that the creation of a prioritasation scheme that facilitates decision-making processes and fund allocation becomes very difficult. To overcome this limitation, further consideration on the road network as a whole should be included. If parameters such as the redundancy of the road network, the bridge importance, the travel time, and the traffic flow indicators are added, even though the general conditions of the bridges are very similar, it is still possible to identify which structures should be given priority. Some considerations are made in this regard in the MIT guidelines, but no specific procedural guidance is provided to enable its practical implementation to overcome these difficulties.

4.3 SUMMARY

The first section of Chapter 4 of the thesis provides a brief description of the characteristics and causes of each defect detected during the Level 1 validation phase of the procedure described in the guidelines. In particular, all the defect sheets compiled by Milano-Serravalle Engineering S.r.l. (MSE) technical have been checked in accordance with Annex C of the guidelines (108 bridges have been analysed).

The defects registered in the bridge portfolio fall under the following categories:

- Defects associated with steel or metal;
- Defects associated with reinforced and prestressed concrete;
- Defects associated with supports;
- General defects;

- Defects associated with expansion joints.

The validation made it possible to highlight some criticisms specific for Level 1 of the procedure:

- Element localisation;
- Inaccuracies in the definition of parameter G;
- Inaccuracies in the definition of parameters k_1 and k_2;;
- Absence of photographic evidence and/or sufficient information to review the sheets.

Moreover, more general considerations on the effectiveness of the guidelines are reported:

- The methodology proposed by MIT is laborious and time-consuming since it requires the analysis case-by-case of all the structures in the portfolio. This criticism can be overcome by the introduction of taxonomies, that, in MSE case study, reduce the number of structures to analyse from 108 to only 2.
- The similar conditions of the structures belonging to the same bridge typology and the similar hazard to which they are subjected if a restrict region is considered don't allow the creation of a prioritisation scheme that facilitates decision-making processes and fund allocation. This limitation can be overcome by integrating in the procedure further parameters that allow to consider the road network as a whole.

In the following chapter, possible improvement of the overall methodology will be discussed, based on the general criticisms highlighted from the guidelines' application.

5. POSSIBLE IMPROVEMENTS

In this chapter, an alternative procedure to improve the effectiveness of the guidelines is illustrated. In particular, it allows to move from a maintenance philosophy to a performance-based philosophy, taking into account further factors (e.g., the redundancy of the road network, the bridge importance, the travel time, etc.) and performing complex analyses that facilitate the creation of a prioritisation scheme and, therefore, decision-making processes. In this way, the criticisms illustrated in Section 4.2 of the previous chapter can be overcome.

Let's consider the simple road network reported in Figure 45.

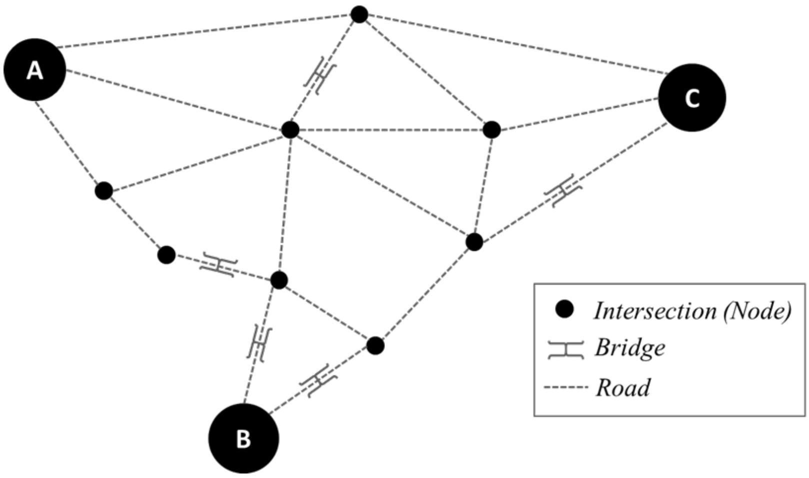

Figure 45 - Example of a road network connecting three points of interest (Adopted from INFRA-NAT Project, "D4.3 - Decision framework for effective resource managment for increased resilience")

A road network can be assessed not only in terms of the structural strength of its individual components, but also in terms of the resilience of the system's connectivity and its ability to recover quickly to a pre-disaster or pre-damage state. In the network shown in Figure 45, where three points of interest (A, B and C) are interconnected by a road system, it is intuitive to conclude that location B is more likely to be severely affected by bridge failure than locations A and C which are more redundantly connected to the network. Therefore, the fragility of the bridges connecting location B becomes more important and critical, and stakeholders (i.e., concessionaires, owners, municipalities, provinces, etc.) should be interested in reducing these bridges vulnerability by implementing modernisation and improvement measures.

To account for this effect and the possible disruption of an infrastructure network due to damage to a bridge (for example, following a seismic event), the overall

To account for this effect and the possible disruption of an infrastructure network due to damage to a bridge (for example, following a seismic event), the overall methodologies developed in the literature follow the procedure outlined in Figure *46*.

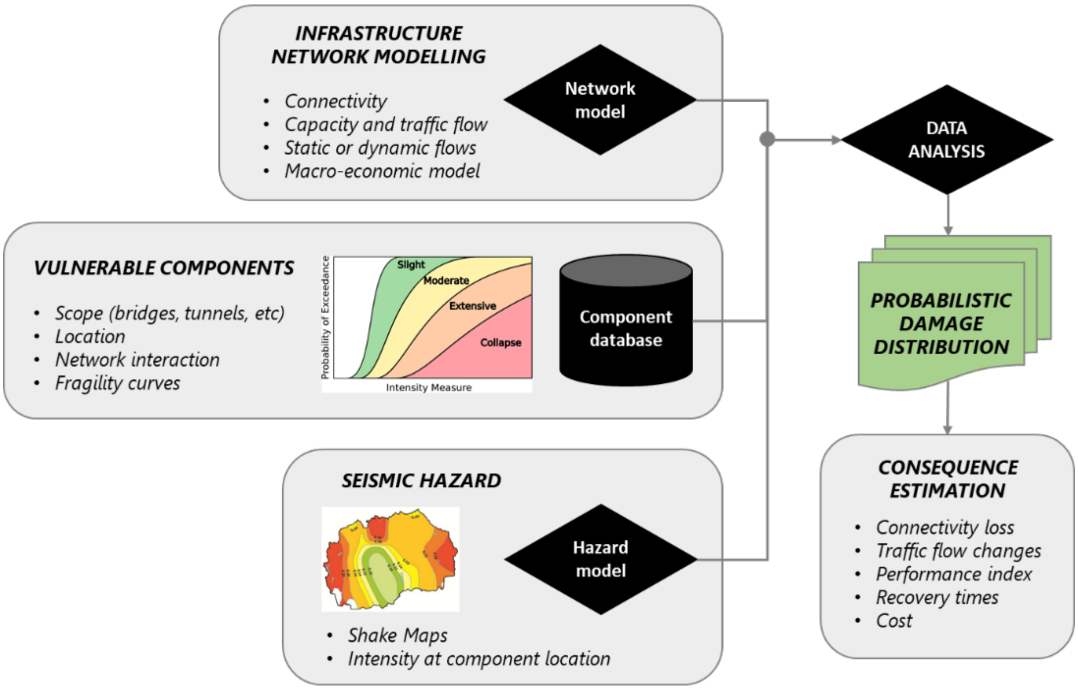

Figure 46 - General methodology for assessing the impact of seismic events on an infrastructure network (Adopted from INFRA-NAT Project, "D4.3 - Decision framework for effective resource managment for increased resilience")

As shown, three inputs are necessary to proceed with the actual analysis: a network model, a hazard model, and a vulnerable component database. The inputs are successively combined to obtain a probabilistic damage distribution that allows consequences in terms of connectivity loss (increased travel time), traffic flow changes, time and cost of repairs to be evaluated.

The different steps of the methodology will be discussed in the following sections considering seismic hazard. However, the approach can be extended to other types of hazards, such as those reported in the guidelines (i.e., structural and foundational hazard, hydraulic hazard, and landslide hazard).

5.1 INFRASTRUCTURE NETWORK MODELLING

Various configurations have been used in the literature to model road networks, ranging from the simple definition of roads as links between locations (used to assess possible connectivity at local level), to complex models that take into account infrastructure capacity and hourly variations in traffic flows that can influence road congestion phenomena. In particular, three different levels of increasing complexity can be considered for the modelling of road networks:

- Level 1 considers only the connectivity of the origin-destination points. The road network is only modelled from basic graphic theory defining points of interest as nodes and roads as lines that are linked using a connectivity matrix;
- Level 2 includes consideration of the network capacity to accommodate traffic flows (possibility of congestion) and travel time changes, in addiction to connectivity considerations;
- Level 3 considers also the indirect economic losses after a disaster that changes the travel demand.

5.2 HAZARD DEFINITION

The second input necessary to proceed with the analysis is the hazard model, which is used to obtain intensity predictions at each bridge location and consequently a probabilistic distribution of the damage.

5.3 VULNERABLE COMPONENT DATABASE

Once a network model has been chosen and implemented, seismically vulnerable components in the system can be identified. The components that are considered vulnerable are those that, if damaged, can cause an impact in the infrastructure service provided. In the case of a road network, these elements may also include tunnels and retaining structures, as well as buildings whose collapse debris may obstruct roads, however, bridges are generally considered the most seismically vulnerable components. Therefore, the methodology described hereafter will concentrate on these infrastructures.

5.3.1 Bridge database and taxonomies definition

In order to assess the effect of earthquakes on the previously defined road network model, it is necessary to create a bridge database and identify taxonomies that well represent the real structural behaviour of the bridge typologies included in the model.

Regarding the bridge database creation, the information that should be included can be divided in different categories:

- **Actual bridge geometry**, that can be deduced from the original construction drawings and from documents relating to any changes introduced as a result of structural interventions after the construction. In the absence of such documentation, a complete survey of the geometry and sample investigations of the foundations must be carried out.
- **Construction details**, namely arrangement and amount of reinforcement. This information can be deduced from the original construction drawings and from documents relating to any changes introduced as a result of structural interventions after the construction. In the absence of such documentation, it is necessary to carry out a number of tests to determine the amount of reinforcement present in a sufficient number of sections to construct a structural model suitable for the analyses and the subsequent verifications.
- **Mechanical properties of materials**: in addition to the initial design specifications, the information must be the result of experimental test carried out at or after structural testing. In the absence of the above documentation, samples must be taken for laboratory testing of concrete. Non-destructive tests carried out on a larger scale are a useful complement but cannot be used as a substitute for destructive tests. For steel, in the absence of adequate experimental data, the characteristics of the material prescribed at the design stage may be considered as a reference, after limited sample testing of its actual use. The above prescriptions refer to the deck support structures (i.e., piers, abutments). Regarding the decks, whatever their type, it is sufficient to verify their good state of preservation, even without experimental measurements, if not considered necessary by the designer.
- **Geotechnical characterisation**, namely soil stratigraphy and mechanical parameters definition.

Levels 0 and 1 of the procedure described in the Italian "Guidelines for the classification and management of risk, safety assessment and monitoring of existing bridges" are a perfect example of how to proceed in the cognitive phase.

On the other hand, taxonomies are defined considering the construction material, the number of spans, the static scheme, the deck type, and the pier type.

Once taxonomies are selected, Finite Elements (FE) models can be constructed to simulate the real structural behaviour of each category when subjected to a seismic event.

5.3.2 Structural models

The structural models must be able to describe all the significant degrees of freedom characterising the dynamic response of the bridges and faithfully reproduce the characteristics of inertia and stiffness of the structures, and the constraints of the decks. The stiffness of the elements must take into account their level of cracking following a seismic event. The deck elements (i.e., beams, transversal beams, slabs), which generally remain in the linear elastic range with limited cracking, may be given the characteristics of fully reacting sections. For piers, which in most cases exceed the yield strength, the effective secant stiffness may be used.

For the global analysis of the structure, the piers and abutments are generally considered to be fixed at the base. The effects of soil-foundation-structure interaction must be considered when the following three conditions occur simultaneously:

- Bridge use class III or IV;
- Soil category D or worse;
- Medium-High seismicity.

5.3.3 Performance levels definition

At this stage, a probabilistic approach is generally used to account for vulnerability through the use of fragility curves, which are functions that relate the probability of exceeding a specific damage state to a ground shaking intensity. To this aim, it is necessary to define the performance levels of interest.

In accordance with the philosophy of safety underlying the national and international standards in force (e.g., Norme Tecniche per le Costruzioni (NTC) 2018, Eurocodes), the infrastructures must be provided with a level of anti-seismic protection differentiated according to their importance and, therefore, to the consequences of their possible damage after an event.

Safety (level of protection) is determined by the association of an expected performance (limit state) with a level of seismic intensity characterized by an assigned probability of exceedance PVR in an assigned period of time (reference life VR). According to NTC 2018, the reference life is obtained by multiplying the nominal life VN of the structure, a function of the "type of construction", by a coefficient CU which is a function of the "class of use": VR=CUVN. The values of VN and CU are given in NTC 2018. The maximum acceptable probability of exceedance in the reference life is given in the codes as a function of the considered limit state. NTC 2018 defines four limit states: two operational (SLS) and two ultimate (ULS). For existing structures, it is generally permissible to verify only the ultimate limit states, namely the life-sustaining limit state or the collapse limit state, defined as follows:

- Life-sustaining limit state: “Following the earthquake, the construction undergoes breakage and collapse of the non-structural components and systems, and significant damage to the structural components, which is associated with a significant loss of rigidity in relation to horizontal actions; however, the construction retains some resistance and rigidity for vertical actions and a margin of safety against collapse for horizontal seismic actions".
- Collapse limit state: “Following the earthquake, the building suffers serious cracks and collapses of non-structural components and systems, and very serious damage to structural components; the building still retains a safety margin for vertical actions and a very small safety margin against collapse due to horizontal actions".

Strategic structures (i.e., bridges in use classes III and IV) represent an exception for which it is necessary to verify that complete usability is ensured following an intense seismic event. The limit state associated with the maintenance of usability is the damage limit state, to which the code associates the maximum value PVR=63 %. This limit state is defined as follows:

- Damage limit state: “Following the earthquake, the building as a whole, including structural and non-structural elements and equipment relevant to its function, is damaged in such a way that it does not endanger users and does not significantly compromise its capacity for resistance and stiffness with respect to vertical and horizontal actions, and remains immediately usable even if part of the equipment is not used”.

5.4 STRUCTURAL ANALYSIS

The safety verifications involve a structural analysis, either linear or non-linear, and subsequent *ad hoc* checks of deformability and strength in all the critical parts of the structure. The use of this procedure therefore requires the availability of the values of all the geometric and mechanical quantities that allow verification on the considered bridge. In most cases the deck is not significantly involved in the seismic response of the structure. It follows that the cognitive investigations are to be addressed mainly to the substructures (i.e., piers and abutments), as well as obviously to the systems of constraint and interconnection between the structural elements (i.e., supports, joints, etc.) which very often are the most vulnerable part of the system.

5.5 CONSEQUENCE EVALUATION

After the structural analysis has been carried out, different methodologies can be used to determine the consequences of each damage scenario on the performance of the road network, ranging from a simple assessment of the loss of connectivity to taking into account changes in traffic flows and indirect costs caused by possible delays.

To this aim, a procedure proposed in the INFRA-NAT Project is discussed hereafter as a potential further development of the guidelines. It consists of a web-based platform composed of two main sections: a section containing "tools" and a section on maps. The first section includes all the calculation, search and selection functionalities; while the map section is where the data is displayed graphically. The latter data should then be stored as tables in a dedicated database capable of handling geographical information. In order to perform a post-event route calculation, the data are uploaded to the database where they can be selected and displayed in the map section of the web platform.

A further extension of the database could include "routing" functionality and contain information about the definition of the shortest route (Dijkstra, 1959). Given a starting point and an end point, the Dijkstra algorithm can calculate the optimal route based on the shortest geometric length of an OSM graph, as shown in Figure *47*, on the left, for an example between points A and B. In addition, the tool should allow the inclusion of break elements that exclude the connection for certain routes and thus indicate a list of available alternative paths. This can be used to represent, for example, collapsed bridges after a seismic scenario has been performed and to use the Dijkstra algorithm to find an alternative route as shown in Figure *47*, on the right.

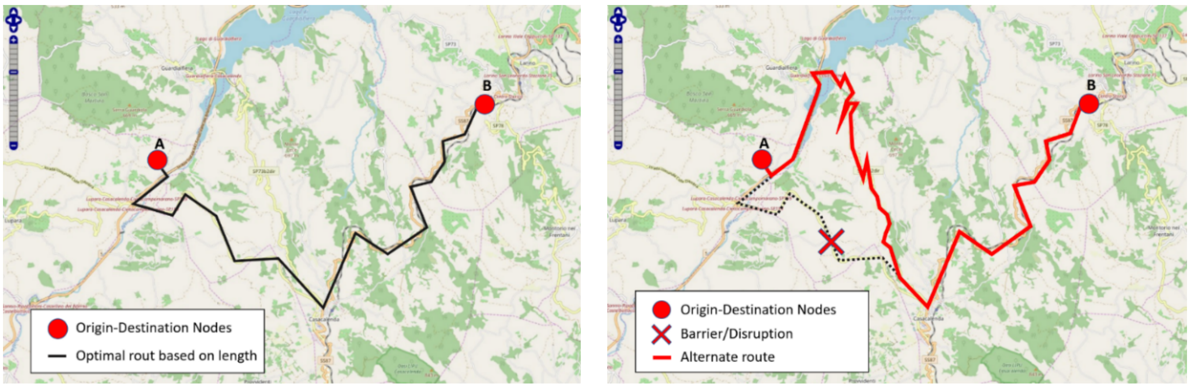

Figure 47 - Left: Example of optimal routes between two points based on the shortest geometrical length; Right: Example of calculation of the alternative route between two points after the bridge collapse (Adopted from INFRA-NAT Project, "D4.3 - Decision framework for effective resource managment for increased resilience")

Moreover, it would also be useful to have traffic and speed limit information for each road. This information combined with the length of the analysed element can be used to derive an estimate of the travel time an individual user would incur while driving on the uncongested road network. By exploiting the routing algorithm in combination with the speed limit information, it is possible to calculate the travel time, thus allowing the construction of a connectivity matrix in which each index reflects the travel time required to pass between the various points in the system, as shown in the network model in ***Figure 48***.

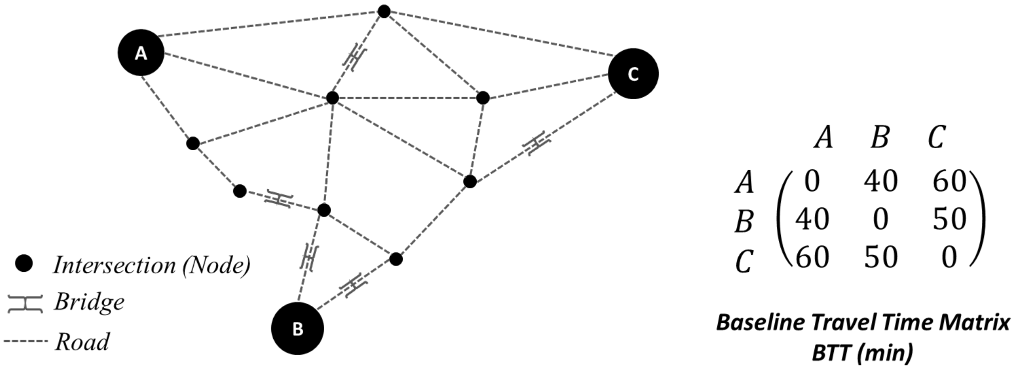

Figure 48 - Example of network model (as in Figure 45) with travel time matrix in minutes (Adopted from INFRA-NAT Project, "D4.3 - Decision framework for effective resource managment for increased resilience")

Including a seismic event to simulate high damage or collapse of some infrastructures in the model, the platform can identify such bridges as obstacles and exclude the road link from the available list, calculating alternative routes in order to avoid the disruption. This procedure can be used to recalculate the updated travel time matrix as shown in Figure ***49***.

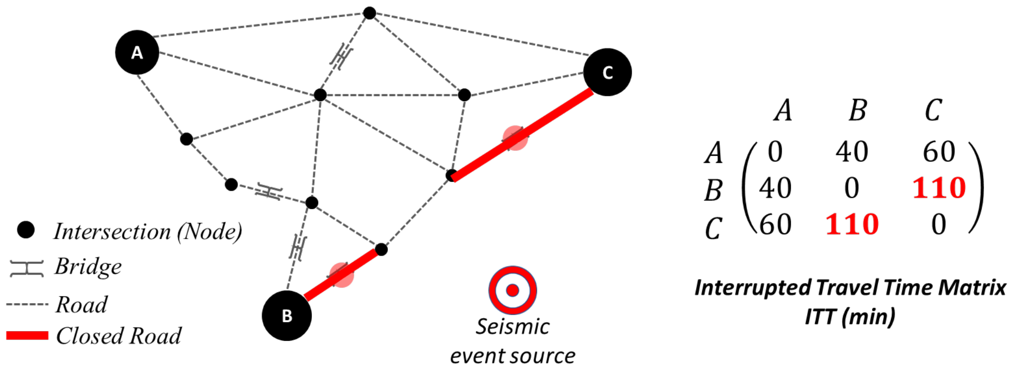

Figure 49 - Example of seismic event simulation (with collapses and obstacles) and recalculation of the travel time matrix in minutes (Adopted from INFRA-NAT Project, "D4.3 - Decision framework for effective resource managment for increased resilience")

Based on the previous example, an index can be calculated for each 'origin-destination' route by dividing each component of the 'Baseline Travel Time (BTT)' matrix by its equivalent in the 'Interrupt Travel Time (ITT)' matrix calculated after the scenario, leading to an 'Interruption Coefficient (IC)' matrix. Furthermore, in order to assess the impact of disruption on each of the 'origin-destination' routes in the constructed network model, an average value of all the non-diagonal terms in the same row of the matrix can be taken to calculate a representative index of such disruption (Origin-Destination Interruption Index). This would allow an understanding of whether certain cities or communities are more likely to be affected by the seismic event. Lastly, in order to assess the overall disruptive effect of the seismic event on the whole road network, it is possible to construct a suitable index by averaging all the individual Origin-Destination Interruption Indexes. This would produce an overall disruption index (Event Interruption Index) that can be used to compare the relative effect of different seismic scenarios on the network as a whole. An example of the indexes discussed above is reported in Figure *50*.

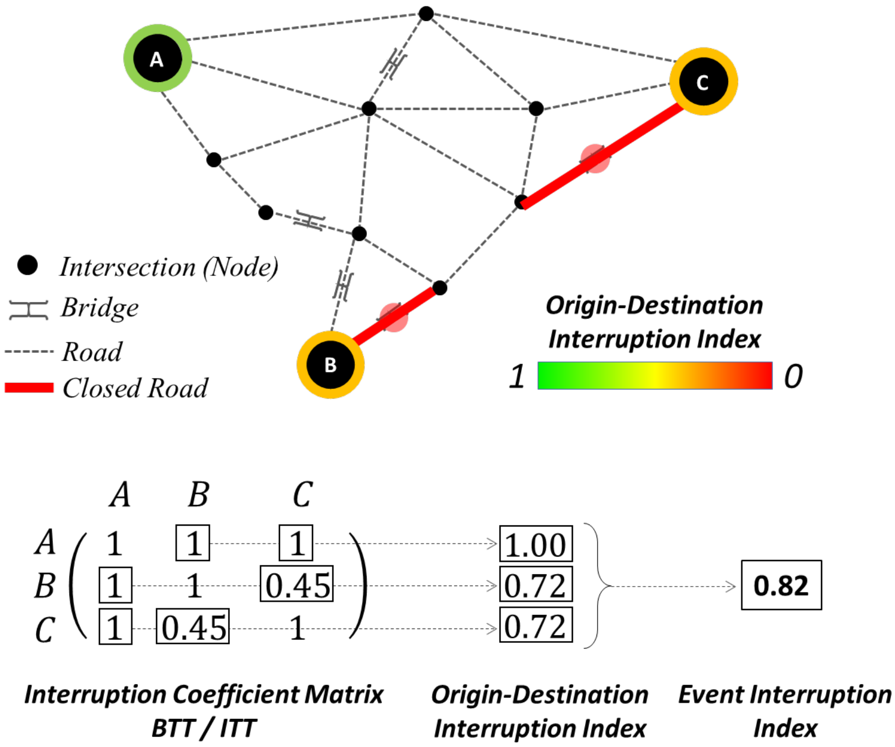

Figure 50 - Example of matrices and indexes: Interruption Coefficient Matrix, Origin-Destination Interruption Index, Event Interruption Index (Adopted from INFRA-NAT Project, "D4.3 - Decision framework for effective resource managment for increased resilience")

In the example shown in Figure 50, it is quite evident that the city represented by node A is not affected by the seismic event since no increase in travel time is caused by the collapsed bridges, leading to an Origin-Destination Interruption Index equal to unity, while the cities represented by nodes B and C are both equally affected since the travel time between them has increased considerably.

5.6 SUMMARY

In Chapter 6, a possible improvement of the Italian guidelines for the risk classification and management of existing bridges is provided, based on the results of previously developed studies discussed in Chapter 2.

To overcome the two main criticisms highlighted with the guidelines' application (time-consuming procedure, difficulty in the creation of a real prioritisation scheme), a more complex analysis is illustrated. In particular, it requires three inputs: a network model, a hazard model, and a vulnerable component database.

The road network can be modelled considering various configuration, ranging from the simple definition of roads as links between locations, to complex models that take into account infrastructure capacity and hourly variations in traffic flows that can influence road congestion phenomena.

The vulnerable component database requires the retrieval of information regarding the actual bridge geometry, construction details, mechanical properties of materials, and geotechnical characterisation. Level 0 and Level 1 of the Italian guidelines are a perfect example of how to proceed in the cognitive phase. Once the information has been collected, taxonomies are defined to shorten the procedure and finite elements models can be constructed to simulate the real structural behaviour under seismic excitation.

After the definition of the performance levels, the inputs are combined though a structural analysis (that can be both linear and non-linear) to obtain a probabilistic damage distribution that allows the consequences in terms of connectivity loss (increased travel time), traffic flow changes, time and cost of repairs to be evaluated.

6. SUMMARY AND CONCLUSIONS

As part of this thesis on the case study implementation and evaluation of the recent "Guidelines for the classification and management of risk, safety assessment and monitoring of existing bridges" in Italy, Chapter 2 consists in a brief description of the two main sources used to develop the thesis: the Italian guidelines Italian "Guidelines for the classification and management of risk, safety assessment and monitoring of existing bridges" and INFRA-NAT Project. The aim of both documents is to create a prioritisation scheme that allows stakeolders to facilitate decision-making processes and fund allocation for the retrofit of existing bridges, which appears to be an importante issue in Italy, after the several bridge collapses that verified during the last decade. The Italian guidelines propose a multi-level approach that, starting from a cognitive phase of the structures in the considered portfolio, allows to assign them a proper CdA based on hazard, vulnerability, and exposure parameters. Depending on the risk class, diversified actions are provided: from a simple preliminary assessment to in-depth analyses. Even though the goal of INFRA-NAT Project is the same of the guidelines, the methodology proposed is completely different: in this study, in fact, the national bridge portfolio is divided into taxonomies, and bridges are considered not as single structures, but as part of a complex road network. Considering a hazard model and proper finite elements models (based on the defined taxonomies), non-linear time-history analyses are performed to obtain a probabilistic damage distribution, on the basis of which consequences are evaluated.

Chapter 3 of the thesis consists of the case study presentation. First of all, the main characteristics of the Italian national bridge stock have been investigated, together with the most common defects affecting the structures. Three main categories of bridges can be distinguished: reinforced concrete arch bridges, reinforced concrete tied-arch bridges, and Gerber bridges. The second section of the chapter concentrates on the description of the infrastructures managed by Milano Serravalle Engineering S.r.l. company (MSE). Also in this case, three categories can be distinguished: reinforced concrete and/or prestressed concrete bridges, steel bridges, and mixed reinforced concrete and/or prestressed concrete and steel bridges (even though, steel bridges represent only 1% of the whole portfolio).

In Chapter 4, Level 1 of the guidelines have been applied to MSE bridge portfolio leading to the identification of several criticisms: element localisation, inaccuracies in the definition of parameters G, k_1, and k_2, and absence of photographic evidence and/or sufficient information to review the sheets. Moreover, also general considerations on the effectiveness of the guidelines are highlighted: the methodology proposed by MIT is laborious and time consuming, and the creation of a prioritisation scheme is not an easy task because of the similar conditions of the structures belonging to the same bridge typology and the similar hazard to which they are subjected when a restrict region is considered, such as in the case study. Both criticisms can be overcome integrating to the procedure recent research developments.

In Chapter 5, a possible improvement of the Italian guidelines for the risk classification and management of existing bridges was discussed, based on the results of previously developed studies. A more complex methodology is illustrated, in which a network model, a hazard model, and a vulnerable component database are combined performing structural analyses, that allows to obtain a probabilistic damage distribution and consequences to be evaluated.

In conclusion, it can be stated that the Italian “Guidelines for the classification and management of risk, safety assessment and monitoring of existing bridges” are a very useful tool to analyse the conditions of the national bridge stock, even though some criticisms can be highlighted.

Almost all of the specific criticisms on Level 1 of the procedure are strictly linked to the subjective judgement of technical, who may interpret the same defect as more or less serious. As a consequence, they can be overcome providing a more accurate description of the defects and their characteristics.

On the other hand, the general criticisms highlighted in Chapter 4 can be overcome respectively by introducing taxonomies and considering the structures as part of a more complex road network. Taxonomies, in fact, allows to reduce the number of structures to be analysed (in the MSE case, from 108 to only 2), whereas the introduction of a road network model leads to consideration of further parameters (e.g., bridge importance, redundancy of the road network, travel time, etc.) that facilitate the creation of a prioritisation scheme, even if the general conditions and the hazard to which the structures are subjected are similar.

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