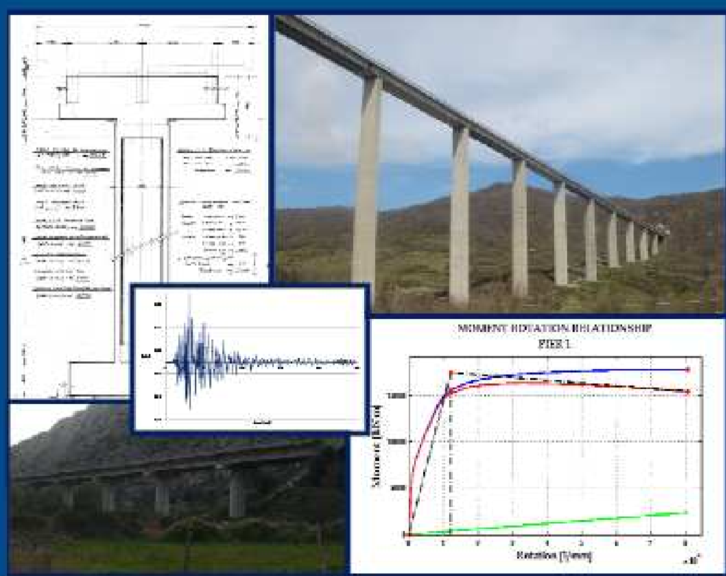
UNIVERSITY OF NAPLES FEDERICO II

PH.D. PROGRAMME IN SEISMIC RISK

XXIV CYCLE

Mariella Mancini

STRUCTURAL PERFORMANCE ASSESSMENT OF EXISTING REINFORCED CONCRETE BRIDGES IN SEISMIC PRONE AREAS



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UNIVERSITY OF NAPLES FEDERICO II

Structural and Geotechnical Dynamics Laboratory

University of Molise

PH.D. PROGRAMME IN SEISMIC RISK

COORDINATOR PROF. ALDO ZOLLO

XXIV CYCLE

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PH.D. THESIS

STRUCTURAL PERFORMANCE ASSESSMENT OF EXISTING BRIDGES IN SEISMIC PRONE AREAS

TUTOR PROF. GIOVANNI FABBROCINO

2011

To my husband

for his brave,

and for his patience.

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I want you all to say that I am sorry for all times I had to tell you: "I call you later".

Mariella

Abstract

For the issue of planning construction areas in order of the prevention and mitigation of seismic risk, in addition to emergency management, the scientific community has for decades studies the identification of tools and methodologies for seismic vulnerability of buildings and infrastructure. In this context it’s very difficult the seismic vulnerability evaluation for existing structures, and especially those distributed.

The complexity of the infrastructural vulnerability analysis is represented by the deep interconnection between different features. Several aspects are connected to this problem, from the correct choose of seismic input, structural and geological knowledge, correct modeling and analysis. The process of assessment of structural security conditions therefore must necessarily be dynamic and iterative, depending on the time scale and spatial analysis in the report. The type and quantity of data needed are dependent on the complexity and heterogeneity of both the geological and geotechnical characteristics of the natural system and of the characteristics of structures. So is very important the need for interdisciplinary approach that can integrate organically different methods of investigation related to the different disciplines involved (geology, geotechnical engineering, structural engineering).

Structural knowledge refers not only to the geometry structural details, and materials, but also to the state of maintenance of the structure. A careful analysis of conservation state of and degradation of the structure, allows two goals. The first is to ensure that they are not workings special phenomena that may compromise the structural safety of the work, and for which the seismic capacity of the structure can be greatly cut down, the second is related to the observation of the conservation status structure, and quantification of degradation/corrosion due to natural structure working.

The attainment of reliable information to study fragility analysis is a complicated and hard work. Still, to obtain hazard and vulnerability structure, it’s need knowledge of site features and geometrical and mechanical structure’s property. The literature on this subject is relatively large but, given the heterogeneity, complexity and scope of works is to be tested is the territory on which they insist, significant progress can be achieved by directing research towards the implementation of strategies and integrated methodologies multi-scale science-engineering.

In this context the evaluation of seismic performance of some really existing viaducts are analyzed.

The result of the work, is the implementation of global Bridge Management System, useful for global assessment at single structure level, and sufficiently detailed for administration of medium-sized populations, such for example a regional scale. The different sections of BMS, allows the collection and cataloguing of background data and knowledge data, and constitute, the base for performance evaluations in structural perspective, linked to the road network exercise, and in seismic perspective, for seismic vulnerability assessment. A guide for the maintenance program and seismic retrofit is provided, useful for by responsible agencies.

Keywords: *existing bridges maintenance, seismic vulnerability assessment, Bridge Management Systems, soil structure interaction.*

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Chapter 1

INTRODUCTION

1.1 INTRODUCTION

In Italy, as well as in Europe and in the US, a strong impulse to the construction of modern road infrastructures dates back about 50 years. In the 50s and the 60s due to economic as well as technological constraints the standard solution adopted throughout the Country consisted of simply supported multiple pre-stressed beam decks with standard span length of 30 to 32m. The bearings were almost invariably made of the low neoprene pads and the joints between decks were rather primitive. For the crossings of large valleys the solution was still that of the RC arch-bridge with upper deck. This typology was progressively replaced in favor of segmental cast-in-situ pre-stressed concrete bridges with symmetric cantilever construction and span lengths of the order of 100m. Mainly due to limited predictive control of the long-term creep and shrinkage effects, with the ensuing pre-stress losses the preferred solution was to have a hinged connection at mid-span. During the 70s, while the construction of the highway infrastructure was reaching completion, in the medium to long span range the segmental launching technique replaced the balanced cantilever construction, and the most common typology for short span length (35 to 40m) remain almost unvaried, as simply supported pre-stressed concrete decks. These latter were now made up either of pre-cast pre-tensioned multiple beams, with T or U sections and cast-in place RC slab, or of pre-cast box-section girders having the full width of the deck, constructed off-site and positioned with launching girders. Bearings did not see any significant evolution until the end of the 70s early 80s when the first pot-bearings made it to the market.

1.1.1. *Seismically historical considerations*

In Japan, first seismic design code was introduced in 1925. Because instability of soils was the major causes of damage, attention was paid for seismic design of foundations at the early days, and prevention devices were first developed and they have been implemented since the 1964 Niigata earthquake. In Japan seismic retrofit programs which were initiated in 1971 and repeated at approximately every 5 years, providing unseating prevention devices has been one of the most common practices of the seismic retrofit. The extensive damage in the 1995 Kobe earthquake revealed inadequate ductility capacity of columns and inadequate strength of restrainers. Shear failure and premature shear failure of columns with termination of main reinforcements at mid-heights resulted in the extensive damage. Design seismic force of unseating prevention devices was increased, and detailing of design for cable restrainers and joint protectors were extensively modified in the code [Kawashima (2000)].

In Italy, as in Europe, seismic design considerations for bridges were implemented in this decade, implying that the design of existing bridges and new bridges are quite different. Until the 90’s, no proper seismic design code existed and seismic prescriptions were only nominal, limited essentially to conventional forces (maximum spectral acceleration of 0.07g-0.1g), without any detailing and capacity design indications. The only exception at Italian level to this rule took place after the 1981 Campania Earthquake, which affected a good number of highway bridges in an area close to the epicenter.

For existing buildings and infrastructures, seismically vulnerability studies of has taken in recent years a particular relevance in relation to the earthquake that struck in 2002 and recently Molise and Abruzzo.

Technical investigation has started in order to carry out an extensive testing on several strategic buildings distributed throughout the country and design of interventions to reduce risk, which was subsequently extended to bridges in network infrastructure [Dolce et al. (2007a)].

1.1.2. *Italian road networks*

The Italian road network is distinguished by its extension, and have many bridges structures located throughout the national territory, both in under the ordinary road, both in the highway.

The major impetus to the development of road network was certainly given, as previously presented, similar to what happened to the building structures, aaaaain the mid-fifties, and has been intense development of the highway network until the mid seventies. After the construction period of new highways, started works of maintenance and modernization of the network of roads and the strengthening of some critical part of high way.

In this way, a very large number of structures it is layered over the. With reference to the ANAS's competence structure, we know that they consist of over 8500 units with light greater than six meters, which combine to form a total area of 900 km. The biggest part of this structures have deck concrete, about 80% of the total, and it is estimated that the average development of decks is approximately 80 m, but there are still many examples of stone viaducts.

From a structural problem, the recent update of the Italian seismic classification has show the issue of seismic vulnerability of infrastructural structures. The lack of the connection between components, of resistance hierarchy, obsolete design or details construction, geotechnical works insufficient to resist at the real load acting are common to many bridges structures in the territory.

On the other hand, it should be noted that only recently, with the evolution of the regulatory framework for seismic design, the subject of evaluation and retrofitting of existing buildings was placed in a field unconnected with the works of new construction.

An examination of the last technical literature, which below, shows how the evaluation of existing buildings is well developed by the codes [NTC (2008)]. Instead, vulnerability assessment of bridges were still in specialist areas, with clear reflections on what documents are defined by the local authorities after the advent of OPCM 3274.

If we consider this area features, we can examine seismic vulnerability issues of strategic infrastructure and the need to proceed by different levels of detail, in accordance with the provisions of OPCM 3274 and subsequent amendments. Emerges a methodological approach useful to optimizing geological and geotechnical investigations to identify the most critical areas in earthquake event and the structural consequence.

1.2 BACKGROUND OF ANALYSIS

For the issue of planning construction areas in order of the prevention and mitigation of seismic risk, in addition to emergency management, the scientific community has for decades studies the identification of tools and methodologies for seismic vulnerability of buildings and infrastructure estimating. In this context it's very difficult the seismic vulnerability evaluation for existing structures, and especially those distributed.

The complexity of the infrastructural vulnerability analysis is represented by the deep interconnection between different features. Several aspects are connected to this problem, from the correct choose of seismic input, structural and geological knowledge, correct modeling and analysis.

The process of assessment of structural security conditions therefore must necessarily be dynamic and iterative, depending on the time scale and spatial analysis in the report. The type and quantity of data needed are dependent on the complexity and heterogeneity of both the geological and geotechnical characteristics of the natural system and of the characteristics of structures [Fabbrocino et al. (2009)].

So is very important the need for interdisciplinary approach that can integrate organically different methods of investigation related to the different disciplines involved (geology, geotechnical engineering, structural engineering).

In this specific context that places the contribution who want investigate some aspects necessary to the implementation and testing techniques to comparative analysis of seismic performance for existing structures in the national road networks. This process, well documented and coded in many guidelines drafted by the regional commissions of experts requires calibration in the case of road works of art, which is for them impact required financial and technical resources much higher than the buildings.

For the structural knowledge, problem in connected not only to the geometry structural details, and materials, but also to the state of maintenance of the structure.

The attainment of reliable information to study fragility analysis is a complicated and hard work. Still, to obtain hazard and vulnerability structure, it's need knowledge of site features and geometrical and mechanical structure's property. Most of the existing bridges was projected and constructed between the fifties and eighties, concomitant whit development of roads and highways network. In most cases don't exist original documentations, but the records of such works consist only on what remained in the Historical Archives of the State or Local Authorities. This material is often not complete, and consists essentially of design, with few references to static or seismic calculations.

The literature on this subject is relatively large [FIB (2007), Pinto and Mancini, (2008) and their related references, Priestley et al. (1996)] but, given the heterogeneity, complexity and scope of works is to be tested is the territory on which they insist, significant progress can be achieved by directing research towards the implementation of strategies and integrated methodologies multi-scale science-engineering.

In this context include the evaluation of seismic performance of some viaducts on the Region of Molise. It represents one of the outcomes of a large activity aimed at the seismic vulnerability evaluation of a number of bridges belonging to a relevant road network in central Italy [Di Carluccio et al. (2009), Fabbrocino et al. (2009)].

The stock analyzed consists of 27 viaducts, located on the main network of Molise Region (§ 2.1).

The sample is very heterogeneous, both in structural and geological issue, and in seismically point of view. A seismic level, there are situations of high seismicity, for viaducts closer to the Apennines, and the area of the Matese hills, areas of medium seismicity, for viaducts along the river valleys Trigno and Biferno, and areas of low seismicity, for viaduct who insist along the Adriatic sea. At geological level, the stock sites is just as varied. There are situations of substrates [Fabbrocino S. et al (2010)], whit different mechanical properties, who justify the presence of different kind of foundation. In only one case, rotational movements and flows along the slope, where the bridge was built, were observed. However, the Molise territory seems to be characterized by different inclination slopes, but all of these versants has gently slopes. No liquefaction phenomenon was observed. The sample is substantially varied with regard to the structural aspects. By using the model of analysis defined for this work, (§ 5.2), 244 piers were studied, whit 13 different cross sections shapes, linked to 3 types of foundation. All soil class are present [NTC (2008)]. In order to show the extreme variability of the case studies analyzed, and the impossibility of accurate description of all viaducts, we report a classification of structures according to the main structural features.

1.2.1. *Simply supported viaducts whit short hollow rectangular piers and shallow foundations*

For this family of viaducts, structural type is beams simply supported viaducts. Beams are simply supported on cast-in place R.C. piercap. The decks are reinforced concrete on pre-cast slab. The viaducts, consist of longitudinal pre-cast I beams whit symmetric or asymmetric section with asymmetric bulbs. In each bays are cast-in place RC diaphragms, 2 at the head and other in the span, placed at a constant distance. Piers are single columns whit hollow rectangular cross section. Piers height are variable by 3-4 meters until 7-8 meters. All bearings are in neoprene pads. Piercaps are devoid of seismically restraint or joint protectors. Foundations are homogeneous, consisting in shallow foundations.

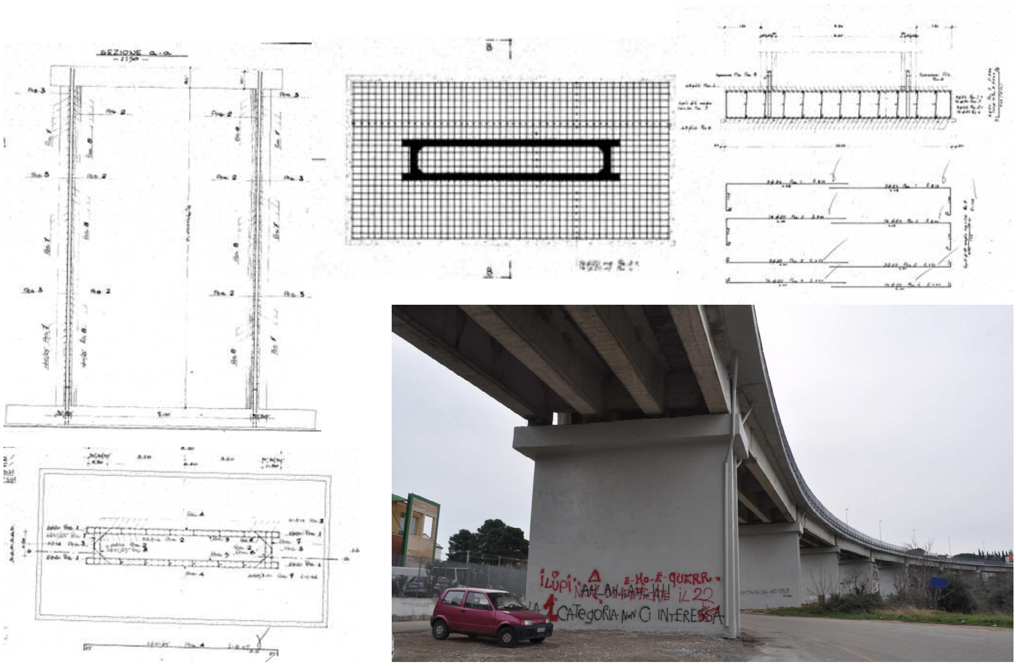


Fig. 1.1 Example of viaduct whit short hollow rectangular piers and shallow foundations

1.2.2. *Simply supported viaducts whit hollow circular piers and on piles foundations*

For this family of viaducts, structural type is beams simply supported viaducts. Beams are simply supported on cast-in place R.C. piercap. The decks are reinforced concrete on pre-cast slab. The viaducts, consist of three longitudinal pre-cast I beams whit asymmetric current section with asymmetric bulbs. In each of the bays are 4 RC diaphragms, 2 at the head and 2 in the span, placed at a constant distance. Piers are single columns whit hollow circular cross section. Piers height are variable by 3-4 meters until 15-16 meters.

All bearings are in neoprene pads. Piercaps are devoid of seismically restraint or joint protectors. Foundations are homogeneous, consisting in on piles foundations.

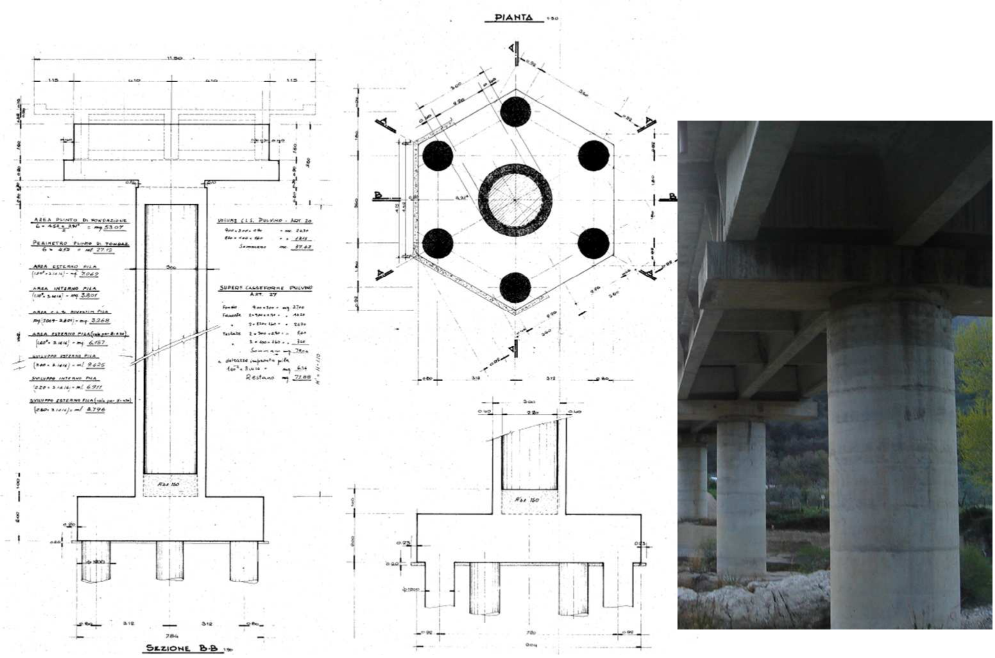


Fig. 1.2 Example of viaduct whit hollow circular piers and on piles foundations

1.2.3. *Simply supported viaducts whit polygonal hollow or full piers and on piles foundations*

For this family of viaducts, structural type is beams simply supported viaducts. Beams are simply supported on cast-in place R.C. piercap. The decks are reinforced concrete on pre-cast slab, consists of longitudinal pre-cast I beams whit symmetric or asymmetric current section, or multi-cell box girders. In each of the bays are pre-cast RC diaphragms, 2 at the head and in the span, placed at a constant distance. Piers are single columns whit hollow or full rectangular of polygonal cross section. Piers height are variable by 3-4 meters until 25-30 meters. All bearings are in neoprene pads. Pier cap is devoid of seismically restraint or joint protectors. However, for some viaduct, in the transverse direction, the presence of one only diaphragm at the head of the beams, or of restraint, ensures the impossibility of loss of supports. Foundations are homogeneous, consisting in on piles foundations.

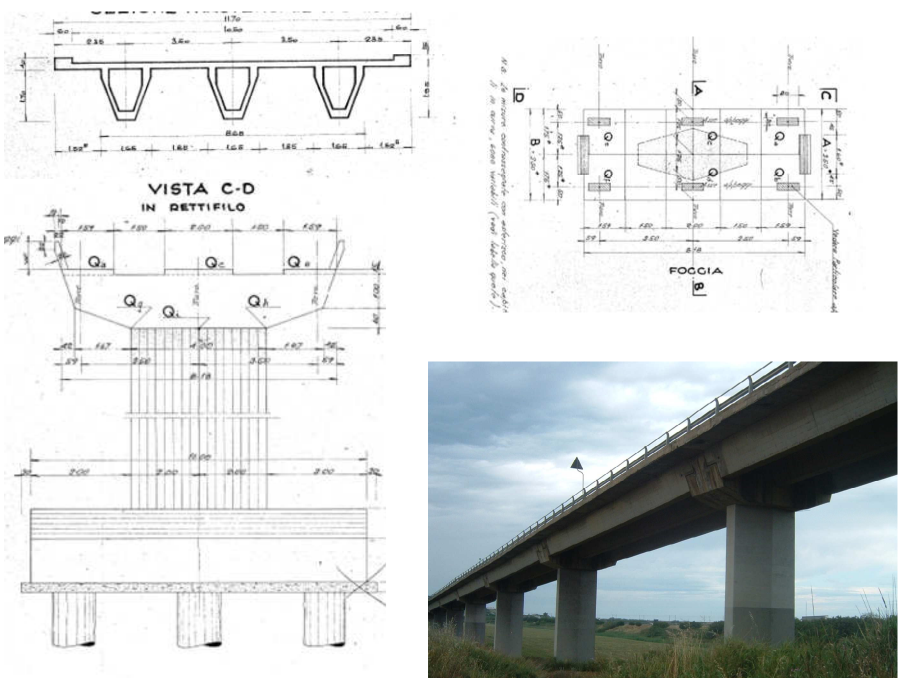
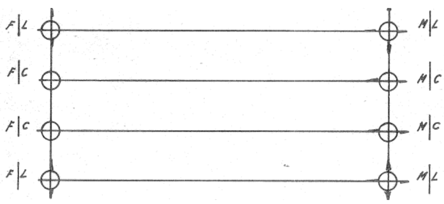


Fig. 1.3 Example of simply supported viaduct whit polygonal hollow or full piers and on piles foundations

1.2.4. *Simply supported viaduct whit rectangular hollow bi-cellular piers and on caissons foundations*

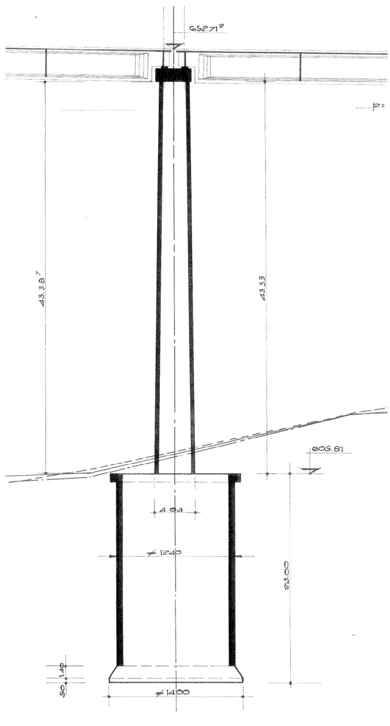
The structural type is beams simply supported viaducts. Beams are simply supported on R.C. piercap. The decks in reinforced concrete cast-in place. The viaduct, 533 m long and 12,50 m width, consists of three longitudinal pre-cast I beams whit asymmetric current section with asymmetric bulbs. In each of the 9 bays are 6 RC diaphragms, two at the head and two in the span, placed at a constant distance.

Piers are single columns whit hollow rectangular bi-cellular cross section. Piers height are variable by 11-12 meters until 45-50 meters. All bearings are mechanical, fixed and movable at the two ends of beams. The static scheme is uniform. The supports consist of fixed hole constraints for the central beams and without hole for the external beams in one end, and mobile multi-direction for external beams and mobile one-way for central beams in the other end. It is assumed that mobile bearings are in neoprene pads, and fixed bearings are steel hinges. Pier cap is devoid of seismically restraint or joint protectors. Foundations are homogeneous, consisting in caissons foundations.



CARATTERISTICHE APPOGGI					
APPOGGIO TIPO	CARICO VERTICALE t/appoggio	TAGLIO LONGITUDINALE t/appoggio	TAGLIO TRASVERSALE t/appoggio	ESCURSIONI LONG. mm	ESCURSIONI TRASVERSALE mm
F	280	23	23-APPOGGI CENTRALI ~ 0 - APPOGGI LATERALI	0	±0 - APPOGGI CENTRALI ±2- APPOGGI LATERALI
M/50	280	~0 (ATTRITO)		±50	
M/80	280	~0 (ATTRITO)		±80	
M/130	280	~0 (ATTRITO)		±130	
M/250	280	~0 (ATTRITO)		± 250	

Fig. 1.4 Bearings characteristics



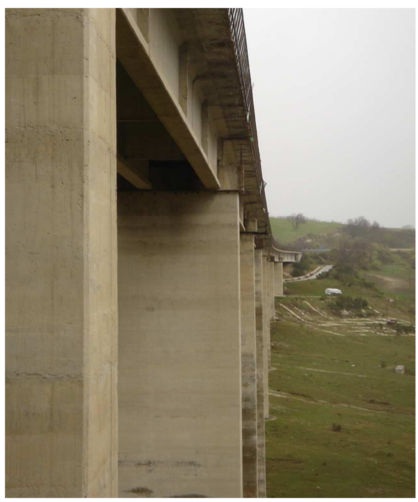


Fig. 1.5 Simply supported viaduct whit rectangular hollow bi-cellular piers and on caissons foundations

1.2.5. *On frame viaducts whit rectangular hollow bi-cellular piers, and on vary types on foundations*

The structural type for this viaducts is not homogeneous. Some spans is simply supported, other are on frame. In one frame spans, beams are rigid connected by slabs, forming a multi-cell box girder for a length of cantilever of 7,00 meters. The decks are reinforced concrete on pre-cast slab.

Beams are pre-cast I beams whit asymmetric current section. In each bays are diaphragms, two at the head and other in the spans, placed at a constant distance.

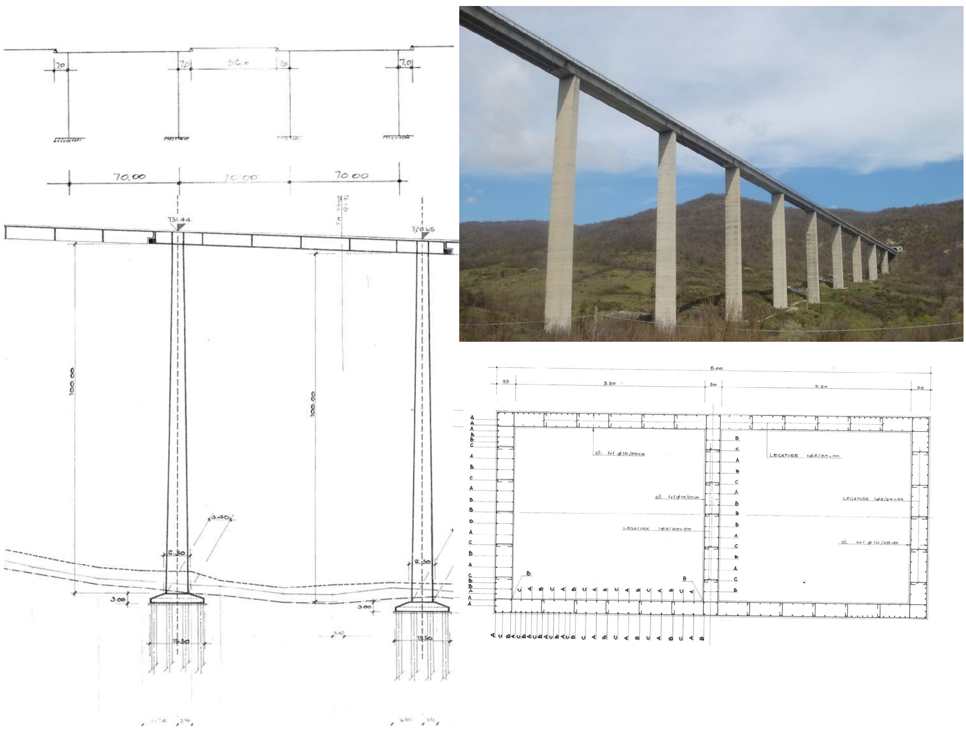


Fig. 1.6 On frame viaduct whit rectangular hollow bi-cellular piers, and vary types on foundations

Piers are single columns whit hollow rectangular bi-cellular cross section. Piers height are variable by 8-10 meters until 100 meters. All bearings are mechanical, fixed and movable at the two ends of beams. The supports consist of fixed hole constraints for the central beams and without hole for the external beams in one end, and mobile multi-direction for external beams and mobile one-way for central beams in the other end. It is assumed that mobile bearings are in neoprene pads, and fixed bearings are steel hinges. Pier cap is devoid of seismically restraint or joint protector. Foundations are not homogeneous, consisting in shallow, on piles and caissons foundations, vary by the soil characteristics and the height of the piers.

1.3 OBJECTIVES AND OUTLINE

A theme of studies in this context could be proposition to implement seismic bridge vulnerability through simplified mechanical models, take into account the effective performance level of the structure.

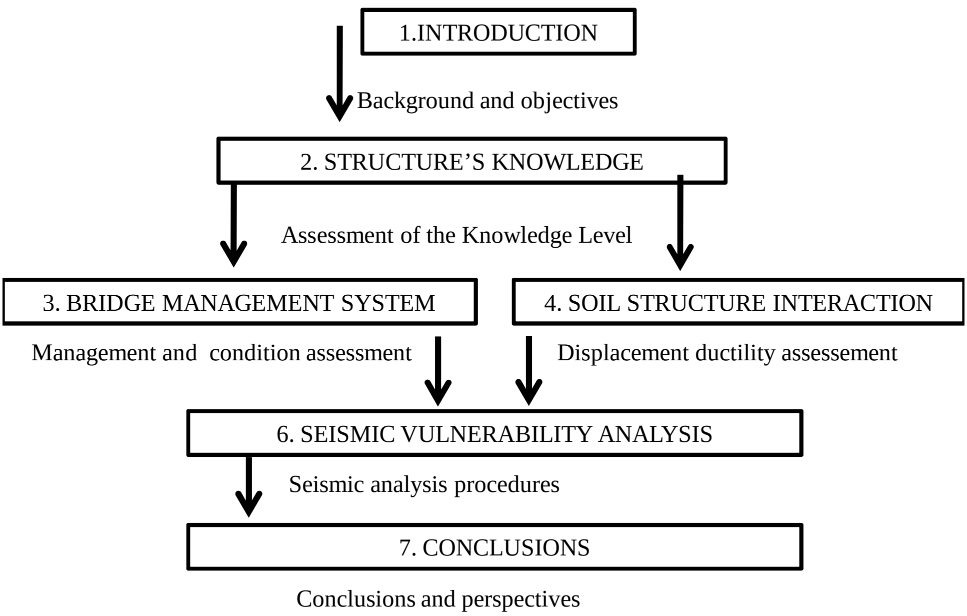


Fig. 1.7 Thesis outline

In fact, the availability of advanced models and complex mathematical models, could be invalidated of since basic input parameters are not readily available.

The aim is the definition of a rational Bridge Management System, able to support the structural maintenance of bridges at regional scale and provide criteria for the prioritization of interventions once vulnerability classification. The knowledge of different kinds of degree, for each structural element, could be estimated non only by subjective judgments, but quantify in objective manner. Models and analysis are integrated part of the seismic bridge assessment, providing the necessary tools to quantify seismic demands and capacities. To select the most appropriate model and type of analysis, is required: the knowledge of the site and of the structure (§ 2), the condition of deterioration the structure, to perform real capacities (§ 3), the seismic bridge design and assessment process (§ 5). In bridge response modelling and analysis still needs understanding, soil-structure interaction (§ 4), for best displacement ductility evaluation.

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Chapter 2

STRUCTURE'S KNOWLEDGE

2.1 INTRODUCTION

The infrastructure stock under investigation is made by 27 bridges located on the four main roads of the Molise region (Fig. 2.1). They play a primary role both for post-earthquake emergency management and daily traffic. They are: SS650 (*Strada Statale di Fondo Valle del Trigno*), SS647 (*Strada Statale Fondo valle del Biferno*) and Branch A, SS87 (*Strada Statale Sannitica*), SS16 (*Strada Statale Adriatica*).

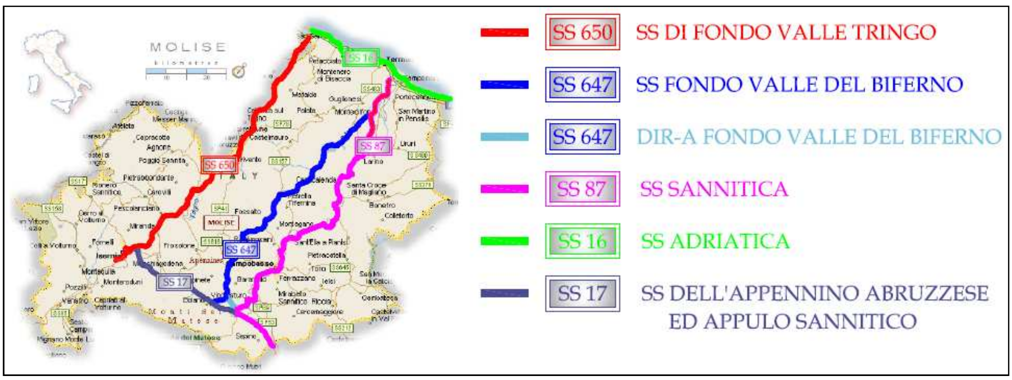


Fig. 2.1 Geographical framework of the main network in Molise Region

SS650 is 75.650 km long, with the first 43.350 km in Molise and the remaining 32.300 km in Abruzzo. It links the internal lands with the Adriatic coast. SS647 is 75.300 km long and also links the internal lands and the Adriatic coast. SS87 is 115.333 km long and it is parallel to SS647. They join right outside Termoli.

SS16 is 35.277 km long, goes across Termoli and links with the Termoli's urban highway. Among the 27 bridges under investigation, 15 are located along SS650, 4 along SS16, 4 along SS87 and 4 along SS647.

2.2 ASSESSMENT OF THE CURRENT STATE OF STRUCTURES AND SOURCES OF INFORMATION

The safety assessment of bridges is carried out through linear and non-linear analyses. Local assessment is also carried out, including evaluation of deformability and strength. Thus, information about geometry of the structures and mechanical parameters of the materials are fundamental for the assessment. Geology, mechanic and seismic classification of the sites are also relevant. Thus, as a first task, all available data and information about each structure have been collected. They can be classified as [Fabbrocino et al.(2009)]:

- Administrative information;
- Hazard information;
- Structural information;
- Geologic information.

In the first group there are data related to the position of the structure, information about design and construction, and daily traffic volumes. In the second group information about seismic hazard of the site where the structure is located is collected. In the third group information and data about geometry and structural schemes are collected. In the fourth group, all information and data about geology of the area where the structure is located are collected [Di Carluccio et. al. (2009)].

2.2.1. *Administrative information*

Administrative information and data for identification and localization of the bridges are collected. In particular, the information about the time of design, construction and maintenance interventions are fundamental for the structural assessment of the existing bridges.

Lack of information and difficulties in carrying out accurate in-situ structural investigations can be overcome by simulated design according to the codes adopted at time of design and construction of the analyzed bridge. In particular, the average daily traffic volume is relevant for management of the infrastructure, for both the planning of maintenance interventions and the evaluation of the consequences of a seismic event on the network.

2.2.2. *Geological and geomorphological information*

The acceleration, velocity and displacement time histories that will excite the bridge foundation will then be filtered by the local soil before attacking the bridge foundation system. An appropriate characterization of the local soil therefore has two objectives:

- to provide parameters required for the definition of the ground motion (filtering effect);
- to provide parameters required to model the soil-foundation interaction (soil stiffness) and to avoid soil failure (soil strength);

The area is characterized by a very complex and heterogeneous geology and morphology. Many bridges of the most important highways of the Region were built in sites with a high hydro-geological hazard. Moreover the seismic hazard of the Region is spanning from moderate to high.

The geological investigation has been carried out according to a number of steps. In the first step documents in the ANAS archive and other administrative offices have been analyzed. Detailed cartography has been collected and integrated with information coming from field investigations and in-situ geological survey. Such an information is useful also to quantify the stability and inherent vulnerability of the area. Geological, geomorphologic, idro-geological and geotechnical information and data able to provide an information about the expected risk are collected.

Geolithological and geomorphologic maps have been produced.

In situ investigations have been designed according to the geological formations found in each area. Direct tests, such as boreholes, performed using continuum boring technique and SPT, and indirect tests, such as DPSH, CPT, refraction seismic tests and MASW tests and geoelectric tomography [Compare et al. (2009)], have been carried out. After the in situ investigations, the mechanical model of the soil underlying each analyzed structure has been developed. Geological survey and classification for the characterization of the areas where the analyzed structures are located have been carried out by Dr. Silvia Fabbrocino and Dr. Fabio Todisco [Fabbrocino S. et al (2010)].

2.2.3. *Hazard information*

According to Eurocode 8 part 1 (EN 1998-1, 2003), each national territory is subdivided into seismic zones, depending on the local hazard. In each seismic zone, the hazard is assumed to be constant and is described in terms of a single parameter, i.e., the value of the reference peak ground acceleration on outcropping bedrock PGA. The reference peak ground acceleration, chosen by the National Authorities for each seismic zone, corresponds to the reference return period TR of the seismic action for the no-collapse requirement. In Eurocode 8, it is prescribed that structures in seismic regions comply with the no-collapse requirement and the damage limitation requirement. At the Italian level, the NTC presents several new terms to describe seismic hazards and seismic actions on structures. First, it introduces a reference period V_R for seismic actions, which is given by the product of the nominal life of a construction V_N and its coefficient of use C_U. It's the number of years during which a structure, if subjected to regular maintenance, should be used for the purpose for which it was designed. It is suggested that $V_N = 10$ years for temporary structures, $V_N = 50$ years for ordinary buildings and structures, and $V_N = 100$ years for large or strategic constructions.

The coefficient of use is directly linked to the class of use of the construction, from Class I (rare presence of people, construction for agriculture, $C_U = 0{,}7$) to Class II (normal presence of people, $C_U = 0{,}10$) up to Class IV (important public and strategic buildings also used for civil protection, $C_U = 2{,}0$).

Two damage limit states (SLO, SLD) and two ultimate limit states (SLU, SLC) are established in the code:

Operability limit state (SLO): after an earthquake, the entire structure, including its structural elements, nonstructural elements, and apparatuses relevant to its functionality, is neither damaged nor subject to significant interruptions in functioning.

Limit state of prompt use or Damage (SLD): after an earthquake, the entire structure, including structural elements, nonstructural elements, and apparatuses relevant to its functionality, has damage that does not compromise its stiffness and resistance against vertical and horizontal actions. The structure is ready to be used but the apparatuses might be subject to malfunctioning.

Limit state for the safeguard of human life or Ultimate state (SLU): after an earthquake, the construction is affected by failures and collapses of nonstructural components and apparatuses and significant damage to structural components that result in a significant reduction of stiffness and resistance against horizontal actions. The construction retains significant stiffness and resistance against vertical actions and retains, as a whole, a significant safety margin against collapse from horizontal seismic actions.

Limit state for Collapse prevention (SLC): after an earthquake, the construction has suffered serious failures and collapses of nonstructural components and apparatuses and very serious damage to structural components that result in a substantial loss of stiffness and a contained loss of resistance against horizontal actions. The construction retains a significant stiffness and resistance against vertical actions but has a small safety margin against collapse from horizontal actions.

According to the code, the probability of exceedance of the seismic action during the reference period varies with the limit state, as shown in Table (2.1).

It follows that the returning period of the design earthquake can be evaluated assuming a statistical distribution of seismic events. If the Poisson model is used to predict the temporal uncertainty of an earthquake, the returning period T_r is given by:

$$T_r = \frac{1}{\lambda_M} = -\frac{t_S}{\ln(1-P)} \tag{2.1}$$

In Equation (2.1), λ_M is the average rate of occurrence of the event, t_s is the time period of interest (the reference period V_r in this case) and P is the probability of a number of occurrences of a particular event during a given time interval.

Limit state		Probability P of exceedance in the reference period VR
Serviceability limit state	SLD	81%
	SLO	63%
Ultimate limit state SLU 10%	SLV	10%
	SLC	5%

Tab. 2.1 Variation of the probability of exceedance of the seismic motion for different limit states.

This way of defining the earthquake returning period is associated with a system that has recently become available in Italy, which allows visualization and querying of probabilistic seismic hazard maps of the national territory using several shaking parameters on a regular grid with a 0.05° spacing [Meletti and Montaldo, (2007)]. This system was directly incorporated into the New Building

Code. Quoting the website http://esse1-gis.mi.ingv.it/help_s1_en.html, the maps display two shaking parameters, Peak Ground Acceleration (PGA) and spectral acceleration (S_a) on stiff horizontal outcropping bedrock [Santucci de Magistris (2011)]. In this context, after the geological database editing, information about a_g (peak ground acceleration on bedrock) and PGA and S are find, using the geographical coordinates in hazard maps of the national territory. For long bridges, seismic hazard is evaluated at the beginning and at the end.

In each node a different value for peak ground acceleration (a_g), local amplification factor (F_0) and control period (T_C), upper limit of the period of the constant spectral acceleration branch, is defined.

2.2.4. ***Structural information***

Information and data about geometry and structural typology are fundamental for the evaluation of the seismic vulnerability of the structures. Thus, all the collected documents and data, including the results of in-situ survey and tests, have been analyzed. Moreover, design and construction techniques have been also analyzed.

Downstream of mentioned above census data, for each of the viaducts were prepared a document called "Level Zero Cards " [Decreto 21/10/2003] that collect all the available structure and site information. So Level Zero Cards become a solid base to identify information available and missing to achieve the desired Knowledge Level [CEN (2005), NTC(2008)]. In "Level Zero Cards ", there are different sections for collect different kind of data. For example in Tab 2.2 and Fig. 2.2 sections of Papers dedicated to the administrative information and geometry and structural details are shown.

2.3 LEVEL OF KNOWLEDGE AND IN-SITU TESTS OF BRIDGES

Seismic codes issued in early 2000s, recommend a distinct approach between the assessment of existing construction and their structural upgrading and the design of new constructions. This concept is present in many National and International codes, mainly with reference to existing buildings [CEN (2005)].

The main difference between new and existing constructions is represented by the sources of uncertainties in determining the structural modeling and mechanical material parameters.

For the aim to perform an integration between base performance of materials and structural details, in seismic point of view, deep knowledge of the structure is required.

STRUCTURE ADMINISTRATIVE INFORMATION						
BRIDGE NAME						
ADDRESS						
CITY					ID. ISTAT	
PROVINCE					ID. ISTAT	
REGION					ID. ISTAT	
GEOGRAPH. COORD.	E		N		ZONE	
OWNER/ADMINISTRATOR						
AGE OF CONSTRUCTION/USE						
DESIGN YEAR						
CONSTRUCTION YEAR						
CONSTRUCTION ULTIMATION YEAR						
MAINTAINANCE YEARS						
MAINTAINANCE DESCRIPTION						
AVAILABLE DOCUMENTATION						
TRAFFIC INFORMATION						
NUMBER OF VEHICLES TRANSITING IN HEAVY TRAFFIC HOURS						
EXISTENCE OF ALTERNATIVE ROUTES IN CASE OF INTERRUPTION OF THE ROAD						

Tab. 2.2 Example of database of administrative information’s.

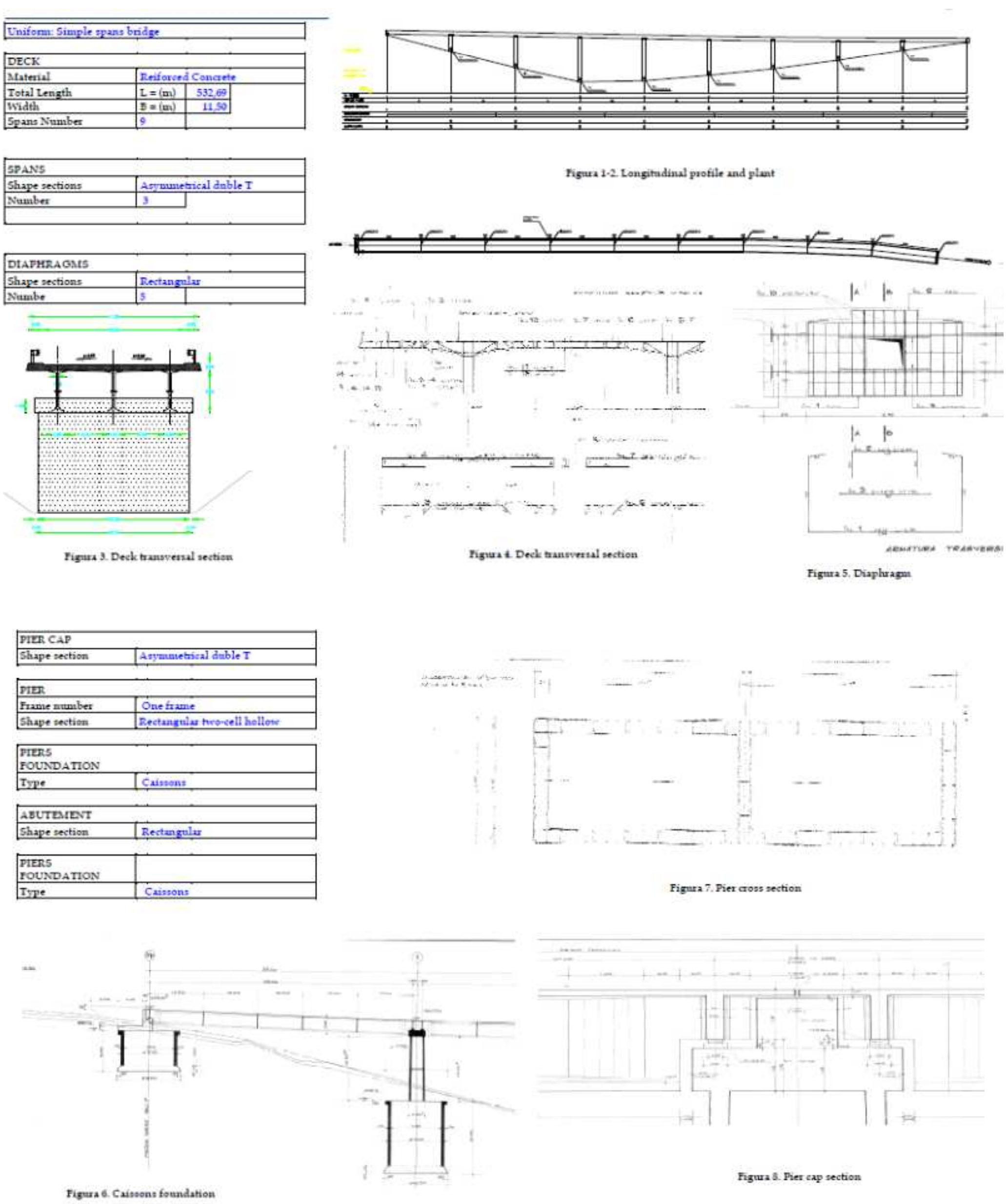


Fig. 2.2 Example of database of geometrical information's.

According to OPCM 3274 (2003) or to Eurocode 8 [(CEN (2005)], a knowledge level has to be preliminarily evaluated in order to define material properties for existing buildings and consequently to define a correct structural analysis and assessment design procedure. As indicate in Eurocode 8, and at Italian level in the O.P.C.M. 3431, for choosing the admissible type of analysis and the appropriate confidence factor value for analysis on existing buildings, three knowledge level are defined:

- KL1: Limited knowledge
- KL2: Normal knowledge
- KL3: Full knowledge

This knowledge level considers three different evaluation steps: geometry, details and materials:

- Geometry: the geometrical properties of the structural system, and of such non-structural elements as may affect structural response;
- Details: these include the amount and detailing of reinforcement in reinforced concrete, connection between members, etc.;
- Materials: the mechanical properties of the constituent materials.

For strategic bridges, the Guidelines for the evaluation of the seismic safety of existing bridges [Pinto et. al, (2009)] require the achievement of an Accurate Level of Knowledge (LC3), apart from specific cases where an Adequate Level of Knowledge (LC2) is allowable. The requirement of an Accurate Level of Knowledge is justified by the strategic relevance of bridges and by the absence of non-structural elements which make access difficult [Progetto DPC-Reluis (2005-2008)].

Mean values of mechanical parameters are recommended in combination with confidence factors (CF) dependent upon the knowledge levels (KL). Knowledge levels achievement are strictly related to the number of tests and the accuracy and extension of inspections performed on the construction. Table 2.3 illustrates the three levels of inspections related to code KL's, namely, limited (KL1), normal (KL2) and full (KL3) and extends recommendation provided for bridges [Pinto et al. (2009)] on the analogy with EC8 Part 3 for buildings (Tab. 2.5).

Knowledge Level	Geometry	Detalis	Materials	Analysis	CF
KL1	From original outline construction drawing whit sample visual survey or from full survey	Simulated design and limited in situ-inspection	Default values and from limited in situ test	LF-MRS	CF_{KL1}
KL2		From incomplete original detailed whit limited in situ inspection or from extended in situ inspection	From original design specifications with limited in situ testing or from extended in situ testing	All	CF_{KL2}
KL3		From original detailed construction whit limited in situ inspection or from comprehensive in situ inspection	From original test reports withlimited in situ testing or from comprehensive in situ testing	All	CF_{KL3}

Tab. 2.3 Knowledge levels and corresponding methods of analysis and confidence factor for buildings

For bridge structures, Tab. 2.3 can be modified as follows.

Knowledge Level	Geometry	Detalis	Materials	Analysis	CF
KL2	From original outline construction drawing whit sample visual survey or from full survey	From incomplete original detailed whit limited in situ inspection or from extended in situ inspection	From original design specifications with limited in situ testing or from extended in situ testing	All	1,20
KL3		From original detailed construction whit limited in situ inspection or from comprehensive in situ	From original test reports withlimited in situ testing or from comprehensive in situ testing	All	1,00

Tab. 2.4 Knowledge levels and corresponding methods of analysis and confidence factor for bridges.

A careful review of code provisions and proposed guidelines shows that some aspects that are not fully established and that a certain margin of interpretation exists.

This applies particularly to the design of the knowledge path and to the spatial configuration, as well as to the outcome of the test results. Moreover, the approach seems to fit requirements for detailed analysis of single structures, but cannot be easily used to large numbers of bridges belonging to road networks at regional scale.

	Relief	In situ tests
Limited in situ inspection	Structural detalis hare verified for 20% of piers (no less then 2 piers)	1 concrete core and 1 rebar sample for 20% of piers (no less then 2 piers)
Extended in situ inspection	Structural detalis hare verified for 40% of piers (no less then 3 piers)	1 concrete core and 1 rebar sample for 30% of piers (no less then 3 piers)
Comprehensive in situ inspection	Structural detalis hare verified for 60% of piers (no less then 4 piers)	1 concrete core and 1 rebar sample for 60% of piers (no less then 4 piers)

Tab. 2.5 Recommended minimum requirements for different levels of inspection and testing.

First step in seismic vulnerability assessment is the research of original documentation and design of bridges stock. Data acquisition for seismic vulnerability assessment is often not trivial. Moreover, most of the bridges have been designed and built in between the Fifties and the Eighties: since, until the beginning of the Seventies, was not obligatory by codes to deposit with the “Civil Engineers” the calculations of reinforced concrete. So the deposit of the calculation whit the Prefecture was solely at the discretion of the construction companies. As a consequence, only partial or limited documents from the original design can be found in the archives, consisting in the most of part in design, while relation about calculations are very infrequent.

2.4 STATE OF KNOLEDGE REVIEW: SIMULATED DESIGN

After the research of documentation we proceeded to simulated design whit the aim of to complete lack of information, by confirming what found, and to examine the questions and evaluate stress on critical sections, where present, generally due to the atmospherically exposition and corrosion of concrete and rebars, but sometimes may be due to the stress levels induced by cyclic loads.

In fact, in case of lack of project documentation, it is necessary to reconstruct, geometry, structural’s details and mechanical properties of materials, through the results of surveys on site and by analogy with other viaducts, by the techniques by which the structure was designed and builted, by reference to the customs regulations in period of construction.

Confirm of available documentation by simulated design, constitute the base for the completion of the information for viaduct for which any document was found.

For each viaduct, design of load patterns, overloads, loads lines and combinations, are related to officials Italian regulations at the time of construction, based on extracts of calculation reports found in the document under investigation. List below provides the institutional Italian framework of last century for the bridges design:

- *Min. LL.PP. (1916), "Norme Tecniche riguardanti le opere metalliche che interessano le ferrovie pubbliche", D.M. 06.05.1916.*
- *Min. LL.PP. (1933), Normale N.8 del 15.09.1933.*
- *Min. LL.PP. (1945), Normale N. 6018 del 09.06.1945.*
- *Direzione Generale ANAS. Circolare N. 820 del 15.03.1952.*
- *Min. LL.PP. (1916), "Norme Tecniche riguardanti le opere metalliche che interessano le ferrovie pubbliche", D.M. 06.05.1916.*
- *Min. LL.PP. (1933), Normale N.8 del 15.09.1933.*
- *Min. LL.PP. (1945), Normale N. 6018 del 09.06.1945.*
- *Direzione Generale ANAS. Circolare N. 820 del 15.03.1952.*
- *Min. LL.PP. (1962), "Norme relative ai Carichi per il Calcolo dei Ponti Stradali", Circolare n. 384, del 14 Febbraio 1962.*
- *Min. LL.PP. (1970), "Norme per la Progettazione dei Ponti Stradali in Acciaio ", Circolare n. 7091, del 4 Novembre 1970.*
- *D.M. 02.08.1980, "Criteri Generali e Prescrizioni tecniche per la Progettazione, Esecuzione e Collaudo di Ponti Stradali".*
- *Min. LL.PP. (1980), STC, Istruzioni relative alla Normativa Tecnica sui Ponti Stradali (D.M. 2.8.1980), Circ.n.220977, 11.11. 80.*
- *D.M. 04.05.1990, "Aggiornamento delle Norme Tecniche per la Progettazione, la Esecuzione e il Collaudo dei Ponti Stradali".*
- *Decreto Ministeriale 14 Gennaio 2008 (G.U. n. 29 del 4-2-2008 Suppl.*

Ordinario n.30) "Approvazione delle nuove norme tecniche per le costruzioni"

Based on the available documentation, the most of bridges under investigation are designed and builted by the implementation of indications of "*Circolare n. 384 del 14 febbraio 1962, del Ministero dei Lavori Pubblici – Consiglio Superiore:*" *Norme relative ai carichi per il calcolo dei ponti stradali*". Two bridges of the stock are designed and builted refering to "*Decreto Ministeriale 02/08/1980, Criteri generali e prescrizioni tecniche per la progettazione, esecuzione e collaudo di ponti stradali (Gazzetta ufficiale 10/11/1980 n. 308)*". One viaduct of the stock were designed and builted using code "*Normale n.6081 del 9-VI-1945 del Ministero dei Lavori Pubblici*".

2.5 IN SITU TESTS PLANNING

The in situ test represent a detailed assessment, as well as a check on the reliability of the design and construction assumptions made, and, where they are systematically confirmed, they acquire an increasingly high degree of reliability. Downstream first visual inspection in situ, and analysis of available documentation and simulated project redaction, it was designed program that provides number and location of elements to be examined, and kind of tests to be conducted, depending of the Level of Knowledge to be acquired and reliability of the test, as well as elements of viaduct really accessible. Acquired first elements of structural assessment, it is passed to define the different types of tests timeline, because they have different reliability levels.

Regarding geometry and construction details, starting with analysis of available original documentation, and following code requirements, for Knowledge Levels (§ 2.3), it was established the number and location of surveys to be carried out. Surveys plan is based also on the findings from visual surveys for the real accessibility of places. For geometry, measurements were made of the structural elements. For structural details, removal of concrete cover for area and depth necessary to check the diameter and centerline of longitudinal and confinement rebars.

Regarding materials evaluation OPCM 3274 (2003) stated that non-destructive test methods couldn't be used in place of destructive tests. This limit was removed by OPCM 3431 (2005). So the on existing structure the concrete strength assessment can be made by destructive test (consisting in localized removal material), or non destructive tests.

FOUNDATION			
	REQUIRED INFORMATIONS	**METHODS**	**TEST NUMBER**
GEOMETRY	Dimension	Shaft on inspection	0
		Non-destructive tests	0
STRUCTURAL DETAILS	Details	Concrete cover removal	0
PIERS			
	REQUIRED INFORMATIONS	**METHODS**	**TEST NUMBER**
GEOMETRY	Dimension	Non-destructive tests	0
STRUCTURAL DETAILS	Details	Concrete cover removal	2
MATERIALS MECHANICAL PROPERTIES	Concrete	Extractions of drilled cores	4
	Rebars	Extractions of rebars	2
ABUTMENT			
	REQUIRED INFORMATIONS	**METHODS**	**TEST NUMBER**
GEOMETRY	Dimension	Non-destructive tests	1
STRUCTURAL DETAILS	Details	Concrete cover removal	1
MATERIALS MECHANICAL PROPERTIES	Concrete	Extractions of drilled cores	1
	Rebars	Extractions of rebars	1

Tab. 2.6 Example of in situ survey planning

In practice, a rational way to set destructive test campaign (concrete core drilling) it is advisable to initially investigate by non-destructive test, as rebound index test and ultrasonic test, can be easily executed. These, in fact, allow the identification of the heterogeneity of materials heterogeneity (rebound index and ultrasonic velocity pulse), and internal cracks (ultrasonic velocity pulse) and then allow to identify structural elements representative of the entire seismic-resistant organism on which to perform destructive tests (concrete cores drilling).

The choice methodology to be adopted is based on costs, on the structural elements damage, on the execution time and on the degree of reliability. In Table 2.6 is a database sample on the plan investigation prepared for one of the viaducts (9 spans) analyzed. The number of tests for steel, lower than that of concrete, is related to the specific knowledge of the original project.

2.6 IN SITU TESTS DIFFICULTIES

Even in the absence of non-structural elements, in-situ investigations of bridges are made difficult by the inherent characteristics (gigantic structures) and the number of elements to be analyzed and by the required detailed assessment.

Due to ageing, atmosphere exposition and cycling loads, most of the bridges underwent also significant maintenance interventions over the years, and the original structural scheme in terms of strength and stiffness could be definitely changed by such interventions. Moreover, limited information about the interventions is often available. The maintenance interventions make the in-situ structural investigations more difficult, in particular in the case of limited information about the operated structural modifications.

As an example, when piers are reinforced, one or two levels of bars are added. If the presence of the added reinforcement is unknown, this can seriously affect the in-situ investigations planned according to the available design drawings. In fact, the strengthening interventions can make the original design drawings definitely unreliable (Fig. 2.3; Fig. 2.4).

As an additional example, the mechanical properties of the external material are often very different from those of the core material, and the amount of reinforcement can be significantly underestimated if the internal layers are not investigated.

The diagnosis phase is very difficult whenever there are access problems to go near the piers of the bridge. Bridges are often located in inaccessible areas and underneath access is sometimes impossible. Access problems lead to a significant increase in costs and time.

Fig. 2.3 Maintenance example consisting in a pier reinforcement

If the piers are located in a river, access is limited at the sides of the bridge out of the river itself. This can cause problems in achieving the required Level of Knowledge, due to the limited number of elements which can be tested. Transportation from one side to the other of the river leads also to an increase in costs. As a consequence, the unit cost of the single test increases (Fig. 2.5; Fig. 2.5). Difficulties arising during the in-situ investigations have a consequence not only on the unit cost of tests in case of a single structure analysis , but also on the costs of management, maintenance interventions and evaluation of the seismic vulnerability in case of a stock of bridges. As a consequence, even the organization of maintenance or only visual inspections for ranking of interventions becomes very expensive, taking into account that there are a few administrations which are responsible for hundreds of bridges. Investigations and diagnosis is even more complex for seismic assessment. Rehabilitation and retrofitting interventions are often associated to funding which is a function of the deck length and the seismic hazard, and the achievement of the required Level of Knowledge becomes even more expensive.

In this context, information’s required for a structural knowledge, could be integrated in a tool for management of the infrastructure stock on large or medium scale, such as the regional scale, is represented by the Bridge Management System, for the classification and management of data about the structures and their health state (§ 3). They make easier the planning of maintenance and the simulated design for the seismic assessment purposes.

Definition of geometry, structural details and material properties are to be used for simulated design, for large scale investigation, supports the in-situ investigations. It is based on technical solutions and design rules at the time of original design and construction and it allows to overcome technical and economic problems for the achievement of the required Level of Knowledge. Moreover, it allows the collection of data and information which are relevant for the seismic assessment but cannot be easily obtained from in-situ investigations, or which can be obtained at very high costs. This is the case, for instance, of foundations and bearings.

Whenever design documents are missing, acquisition of information about foundations (Fig. 2.7) is very expensive since it requires the excavation of large volumes and it often happens that only a few data about geometry of shallow foundations can be obtained. Height of footing and embedded foundations cannot be assessed. About bearings, type and dimensions can be reliably assessed only through a by bridge.

Fig. 2.4 Maintenance example consisting in a abutment reinforcement

Fig. 2.5 Bridge across the river example, for which is not possible to go near piers

Fig. 2.6 Bridge non accessible example, for which don’t exist any road to go near piers

Fig. 2.7 Foundation inspection example

Because of the difficulties during the document search, considering that for some viaduct were not found any documentation or project, and by use of simulating design, considering difficulties related to the non reachability of the entire viaducts, as well as maintenance and reparation or retrofitting over time, Normal Knowledge KL2 is achieved. A Confidence Factor equal to 1,20 is associate to such a level of knowledge.

2.7 IN SITU TESTS RESULTS

2.7.1. ***Geometry and structural details knowledge***

Geometrical, structural and mechanical results were used to examine the condition of things in relation to the original design and analysis of loads. Since the activity focused on the analysis of seismic vulnerability, attention has been focused on vertical concrete structures. Generally, deck is not significantly involved in seismic response of the structure. So surveys are to be addressed to the piers, abutment and foundations, and to the interconnection systems (bearings, seals, etc). Deck, beams and diaphragms are geometrically important, only as a weight to estimates seismic mass in the head of the SDOF system (§ 5.2.7), who represent the soil-pier-foundation system. For geometry and details knowledge tests, made by removal of concrete cover on a depth and area sufficient to evaluate rebars centerline and diameter, they were used for confirm information of original design, identify any subsequent maintenance or assessment, and, for bridges which it wasn't found documentations, to calculate seismic mass, and resisting sections and structural details (Fig. 2.8).

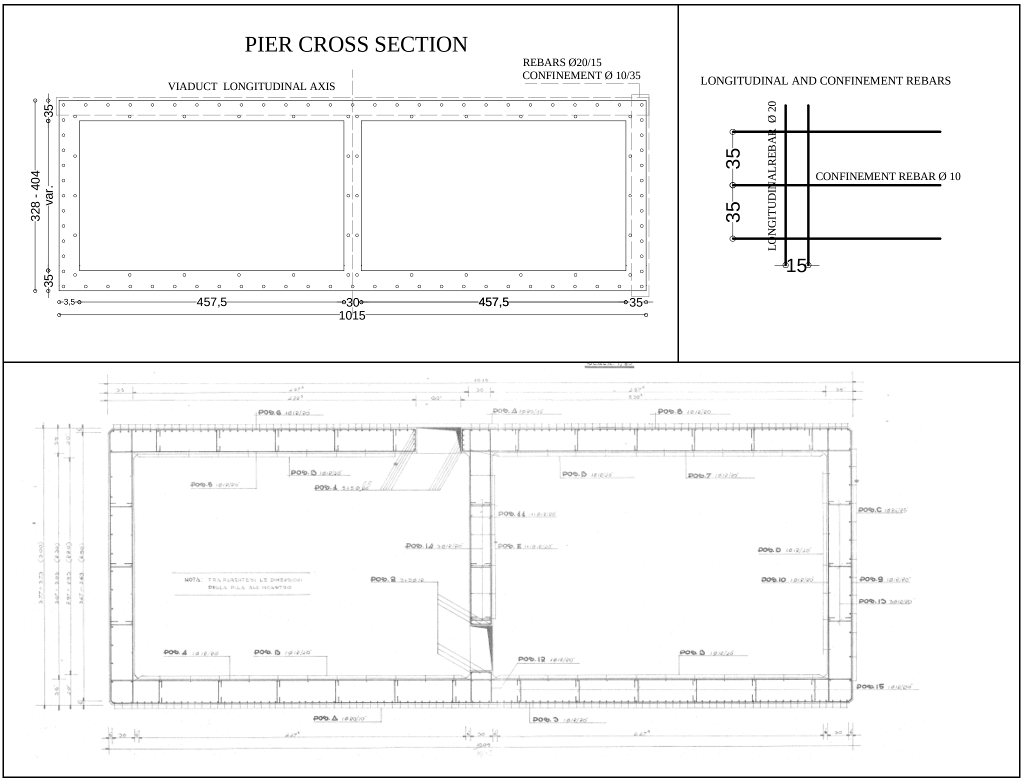


Fig. 2.8 Comparison between structural detail in original design and in situ surveys

2.7.2. *Material mechanical properties*

The most popular methods among destructive test id the concrete core drilling and extractions of steel rebars. The drill is the extraction of cylindrical specimens, performed in crushing tests Laboratory.

Among non destructive test, popular methods are the Rebound Index (IR) determination by Schmidt hammer. Thus for Rebound Index, correlations between Ultrasonic Pulse Velocity could to be established to have combination method Sonreb. Non destructive tests are efficient, because the test speed for measurements perform and limited damage allows you to examine a large number of points. However, the result is less reliable, and requires to be calibrated using the results from concrete core drilled in same element. Due to the lower reliability information, a higher number of test is required. For concrete strength performance, three Sonreb test are made for each concrete core substituted. The strength of core samples has to be converted into the corresponding in-situ concrete strength, before to being used in calculations. To convert cores resistance $f_{car,i}$ on the corresponding in-situ resistance $f_{cis,i}$, following relation can be used [Masi (2005)]:

$$f_{cis,i} = \left(C_{h/D} \cdot C_{dia} \cdot C_a \cdot C_d\right) \cdot f_{car,i} \tag{2.2}$$

where:

- $C_{h/D}$ coefficient for length to diameter ratio $h/D \neq 2$:

$$C_{h/D} = 2/\left(1{,}5 + D/h\right) \tag{2.3}$$

- C_{dia} coefficient for core diameter;
- C_a coefficient for rebar presence;
- C_d coefficient for damage drilling.

For materials material mechanical properties surveys 94 concrete cores were drilled out, 74 rebars were extracted, 66 rebound test (average of indirect measurements for a global number of 198 measurements), and 58 ultrasonic test sites were defined (average of indirect measurements for a global number of 174 measurements).

For each element, where the tests have been conducted, there have been three stations of rebound index and ultrasonic tests, and the average values are used for analysis (Tab.6).

Due to direct contact of RC elements with atmosphere (often near the sea) and pollution, relevant carbonation effects on concrete were expected and controlled by means of carbonation degree measures.

Before drilling out non destructive tests were carried out in the test site in the aim to correlate laboratory to in-situ results.

REBOUND INDEX FOR COMPRESSIVE STRENGHT														
Station	H [m]	IR												IR. Average
		1	2	3	4	5	6	7	8	9	10	11	12	
1	**2,50**	53	48	47	48	52	46	46	49	50	48	50	49	**48,83**
2	**2,50**	49	50	49	49	49	53	51	45	47	53	50	49	**49,50**
3	**2,50**	48	49	47	48	47	47	48	46	50	48	48	49	**47,92**
										IR. Average				**48,75**

ULTRASONIC PULSE VELOCITY						
Station	H [m]	Distance	L [m]	Time [μsec]	Velocity [m/s]	Average Velocity [m/s]
1	**2,50**	Indirect	0,2	75,10	2663	**2644**
		Indirect	0,2	75,70	2642	
		Indirect	0,2	76,10	2628	
2	**2,50**	Indirect	0,2	83,20	2404	**2465**
		Indirect	0,2	79,40	2519	
		Indirect	0,2	80,90	2472	
3	**2,50**	Indirect	0,2	78,60	2545	**2518**
		Indirect	0,2	82,30	2430	
		Indirect	0,2	77,50	2581	
			Average values [m/s]:		**2543**	

Tab. 2.7 Sonreb database for in situ tests

Compressive strength values is principal objective of this test series. In order to carry out each test phase, tests were analyzed in Official Laboratory expert technical people was employed and standard provisions are taken into account: UNI EN 12504-1:2002 (core drilling and testing), UNI EN 12504-2:2001 (rebound number), UNI EN 12504-4:2005 (ultrasonic pulse velocity), UNI EN 13295:2005 (resistance to carbonation).

2.7.3. *Sonreb tests calibration*

Sonreb test are calibrated using following expressions:

$$R_C = \alpha \cdot S^{\beta} \cdot V^{\gamma} \tag{2.4}$$

where coefficients α, β and γ were calibrated by non linear regression models based, using average of rebound index (S) and ultrasonic pulse velocity (V) of test were sample of concrete cores were drilled out.

Sets of correlations (Eq. 2.4) were calibrated related to family of bridges, regrouped for geographic position, design and age of construction.

In the following Table (Tab.2.8) and Figure (Fig.2.9), are presented one sets of data for Sonreb tests, and relative strength values of the concrete cores drilled.

For each test, in situ strength f_{car} coefficient for length to diameter ratio, core diameter, and rebar presence (Eq. 2.2).

Effective strength f_{cis}. Cube strength R_c is evaluated, and it is checked for a direct proportionality between the strength values obtained from the core tests and the corresponding values obtained from average V and S of non-destructive testing (Fig.2.9,a,b).

IC	IR	V	D	H	fcar,i	fcis	Rc
S3	47,08	2562	94,00	189,28	25,73	28,35	34,57
C4	48,22	2731	94,40	94,58	38,70	34,08	41,56
C5	48,00	3008	100,00	200,00	28,53	31,38	38,27
S1F	40,03	1455	94,00	189,00	20,29	24,61	30,01
C4	48,22	2731	94,40	94,58	38,70	34,08	41,56
C1	56,22	3779	94,40	186,98	51,86	56,91	69,40

Tab. 2.8 In situ and laboratory test results for structural materials of the study bridges.

In this case, the direct proportionality are controlled whit goods results, and parameters to determine the Sonreb expression are calibrated. Coefficient $\alpha, \beta$and γ, are evaluated by non linear regression and the Sonreb correlation found is:

$$R_C = 0{,}000104 \cdot S^{5{,}5267} \cdot V^{-1{,}0779} \qquad (2.5)$$

Expression of R_C cube strength is evaluated for element for which only non destructive tests are made using the Expression 2.5.

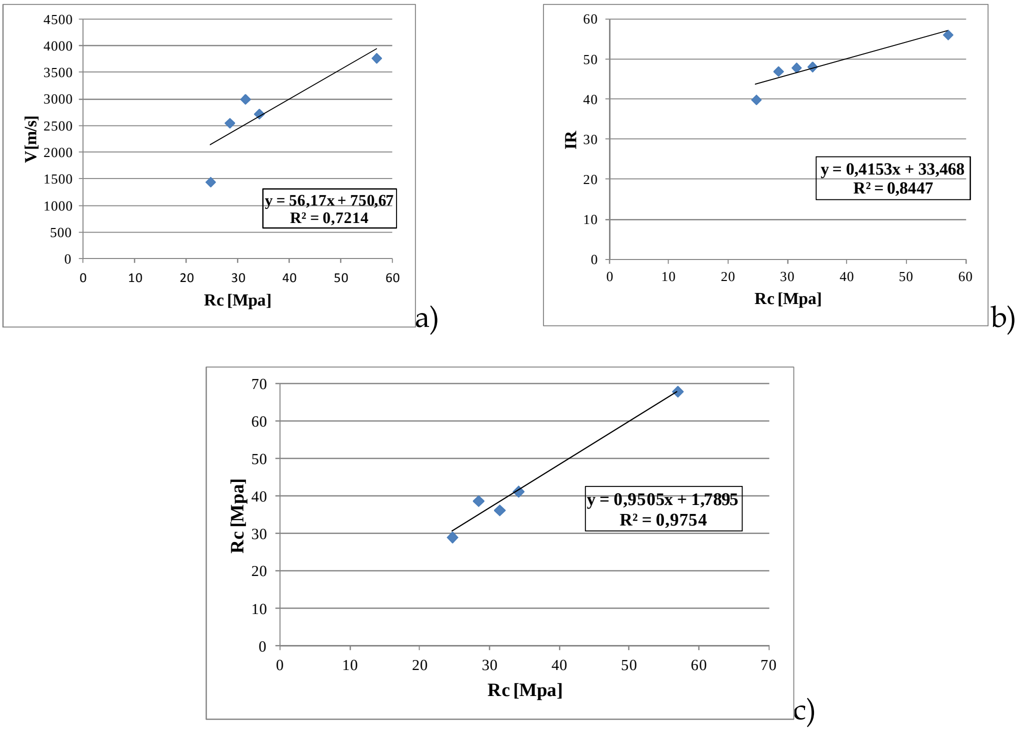


Fig. 2.9 Proportionality between the strength values obtained from the core tests and the corresponding values obtained from average V (a)and S (b) of non-destructive testing. Proportionality between the strength in situ values and calibrated Sonreb test

2.8 SIMULATED DESIGN DATABASE

Results of analysis of documentation, simulated project, and in situ surveys, are summarized in database who allows two objectives. In order to the management of the bridge stock, constitute the first step in order to Bridge Management System implementation (§ 3), to support the knowledge, maintenance, and whit the database related to the status of degree, to the interventions planning. For seismic vulnerability assessment, database contains information of structural design and geometry of entire bridge, of each class of structural elements, information’s about bearings, isolation systems, information’s about characteristics useful for estimate bridge capacity at different limits states (§ 5).

Following tables (Tab. 2.9-2.13) present geometrical database for input data. Loads and stress calculation are related to the original design, and simulated design, whit aim to identify and complete structural geometry and details by verification rates in exercise. They reflect consideration explained in this chapter.

In tables (Tab. 2.14-2.15) are presented an example of database for loads analysis and combinations. Calculations are referred to one viaduct, for which the geometry is presented in Tab. 2.9-2.13, designed and builted using "*Circolare n. 384 del 14/02/1962*". To explain what summarized, a brief excursions of loads and combinations is following presented.

Dead Load

For dead loads evaluation, structure weight is evaluated by computing elements dimension and applying unit concrete weight.

Live Load

For the overloads, the "*Circolare n. 384 del 14/02/1962*", road bridges divides into two categories:

- Roads for the transit of civil cargoes and military trucks;
- Roads subject to the transit of civil cargoes only (vicinal roads of local interest).

The overloads considered are as follows:

- Overload 1: undefined column of military trucks (12 ton);
- Overload 2: isolated steamroller (18 ton);
- Overload 3: people compact mass (400 kg/m^2);
- Overload 4: undefined column of military trucks (61,5 ton);
- Overload 5: undefined column of military trucks (32 ton);
- Overload 6: isolated military truck (74,5 ton);

Transversal length of pattern of layers n.1 and n.2 is 3,00 m. For layers n. 4, 5, and 6 is 3,50 m.

Viaduct are all in highway so layer considered is: the most onerous of the overloads 4, 5 and 6, to place side by side whit one or more columns of military trucks (12 ton), and people compact mass (400 kg/m2).

Wind load

Wind actions are estimated in codes as a pressure in horizontal direction, for the surface normally affected. Pressure is evaluated as:

- 250 Kg/m2 for bridge discharged;
- 100 Kg/m2 for bridge overloaded.

Ina overloaded case, wind in charge is considered by increasing with a continuous strip of 3,00 m, starting over the deck paving.

Snow load

With regard to the snow actions, codes indicate the need to consider load exclusively for covered structures. Snow load is not considered in the open decks as it is impossible to coexist with other high overloads.

Dynamic allowance

For 0-100 m spans, dynamic actions are introduced amplifying variable loads by following coefficient:

$$\varphi = 1 + \frac{(100 - L)^2}{100 \cdot (250 - L)} \tag{2.6}$$

where L is span length in meter. For $L \geq 100$ m, $\varphi = 1$ is assumed.

Centrifugal force

In curved bridges, centrifugal force is evaluated as:

$$F = \frac{60}{R} \text{ [t/m]} \tag{2.7}$$

Where R is radius of traffic lane. This is applied on a height equal to road pavement.

Brake Action

For road bridges brake action is considered by a horizontal force equal to 1/10 of the overloads consisting of a single indefinite column truck and not less than 0,3 of the weight of the heaviest load in the considered scheme.

Lateral actions on the barriers

The crowd along the sidewalks holding action on the barriers of 250 kg/m agents on a height equal to 1,00 m above road pavement.

Seismic Action

Seismic actions are introduced by code *“Legge 25 novembre 1962, n. 1684(Gazzetta Ufficiale 22 dicembre 1962), n. 326 Provvedimenti per l'edilizia, con particolari prescrizioni per le zone sismiche”*. It required that seismic action is applied as horizontal force in gravity center of various masses. The ratio of the horizontal forces and weights corresponding to the masses on which they act must assume equal to 0.07.

Loads combinations

In the calculation of operational demands the following load combinations are considered, in accordance with the codes and construction technique of the time. As example calculation of stresses on a bridge is shown. Is adopted for the pier cap height 1,45 m and for deck height of 3,30 m. The tables below shows the stress analysis for different combinations of loads for each pile of the bridge.

- Combination 1 – Dead loads + live loads
- Combination 2 – Dead loads +half live loads
- Combination 3 – Dead loads + discharged bridge wind
- Combination 4 – Dead loads + transversal seismic action
- Combination 5 – Dead loads + longitudinal seismic action

The first two combinations represent, respectively the maximum axial load and the maximum moment bending due to the transversal eccentric loads acting on the base of the piers. Stresses rate in exercise, are verified by domains of moment- axial loads interactions conveniently constructed.

Examples of domains are presented in Fig. 2.10-2.13.. Domains are related to the demands shown in Tab. 2.15.

GEOMETRIC IDENTIFICATION					
DESIGN		Seismic			
STRUCTURAL MATERIAL		Reinforced Concrete			
STRUCTURAL TYPE		Multi spans silmpy supported			
CURVED		RADIUS	1350	DIRECTION	-------------
SPANS NUMBER	DECK LENGTH	AV. SPAN LENGTH	MAX SPAN LENGTH	MAX PIER HEIGHT	MIN PIER HEIGHT
	m	m	m	m	m
9	533,00	59,22	60,00	43,38	11,84
BEAMS NUMBER	SPAN WIDTH	MIN PIER HEIGHT	DIAPHRAGMS NUMBER	BEARINGS	
				Mechanical	
	m	m		FOUNDATIONS	
3	11,5	26,23	6	Piles foundation	

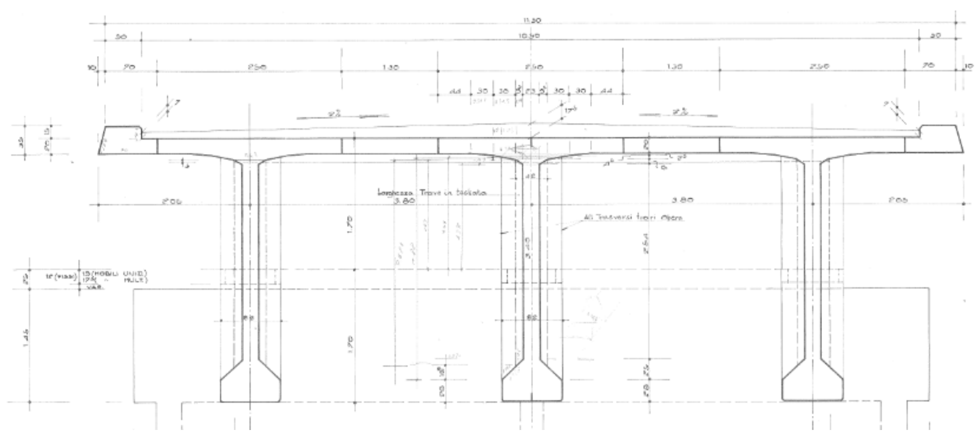
LENGTH SUPPORT OVERLAP			
LONGITUDINAL LENGTH PIER SUPPORT OVERLAP		0,80	m
TRANSVERSAL LENGTH PIER SUPPORT OVERLAP		0,60	m
LONGITUDINAL LENGTH ABUTMENT SUPPORT OVERLAP		0,80	m
TRNSVERSAL LENGTH ABUTMENT SUPPORT OVERLAP		0,60	m
BEAM-PIERCAP DISTANCE		0,10	m
DECK DISTANCE		0,05	m
LONGITUDINAL RESTRAINTS	NO	TRANSVERSAL RESTRAINTS	NO

SPANS LENGTH					
1	2	3	4	5	6
59	60	60	60	60	60
7	8	9	10	11	12
60	60	59			
13	14	15	16	17	18

PIERS HEIGHT					
1	2	3	4	5	6
14,33	25,72	37,62	43,38	32,12	27,19
7	8	9	10	11	12
17,64	11,84				

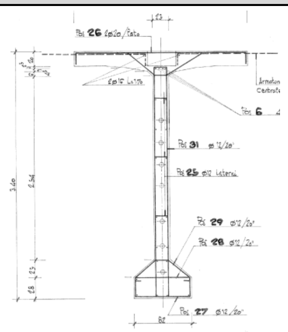
Tab. 2.9 Geometrical identification database

DECK					
STRUCTURAL TYPE	Beams				
SEISMICAL RESTRAINTS	No				
	Lenght	**Width**	**H**	**P**	**G_k**
	m	**m**	**m**	**daN/m³ daN/m**	**kN/m**
Pavement	1	11,5	0,15	1300	**2243**
Barriers	2			500	**1000**
Deck	1	4	0,2	2500	**2000**
	2	0,5	0,22	2500	**275**
			TOT		**5518**



Tab. 2.10 Deck geometric properties

BEAMS	
STRUCTURAL TYPE	Simple span
STRUCTURAL MATERIAL	Prestressed Reinforced Concrete
SHAPE SECTION	Asymmetric I beam



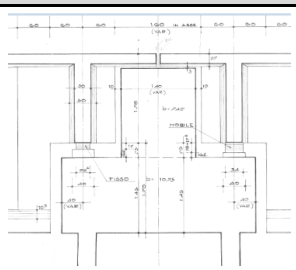
N	A	P	G_k
	mq	daN/m^3	daN/m
3	1,5	2500	11250
			444
	Tot. G_k		**11694**

BEARINGS			
	KIND	TOTAL AREA	
LEFT	FIXED		m^2
RIGHT	MOVE		m^2

DIAPHRAGMS	
STRUCTURAL MATERIAL	Prestressed Reinforced Concrete
SHAPE SECTION	Rectangular

N	L2	H	B	P	G_k
	m	m	m	daN/m^3	daN
12	2,9	3	0,25	2500	**65250**

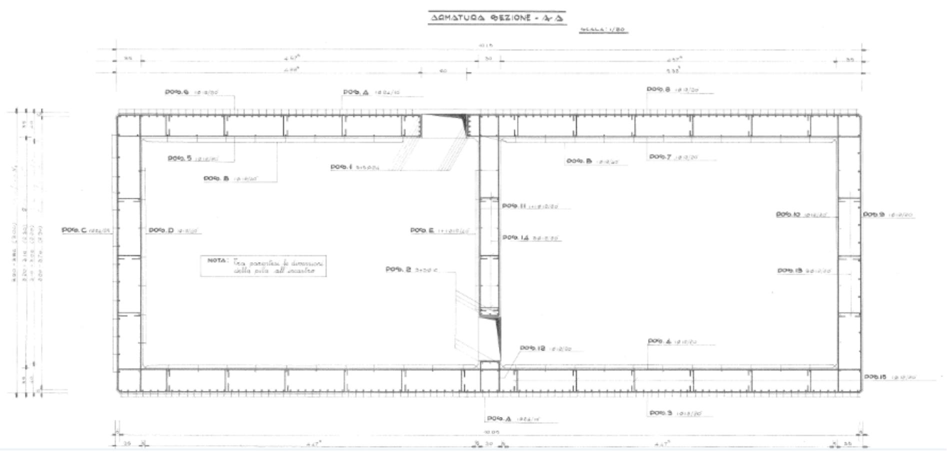
PIERCAPS	
STRUCTURAL TYPE	T upside down
STRUCTURAL MATERIAL	Reinforced Concrete
SHAPE SECTION	T upside down



H_1	B_1	L_1	G_k
m	m	m	daN
1,45	10,75	3,60	140288
1,60	1,40	8,00	44800
	Tot. G_k		**185088**

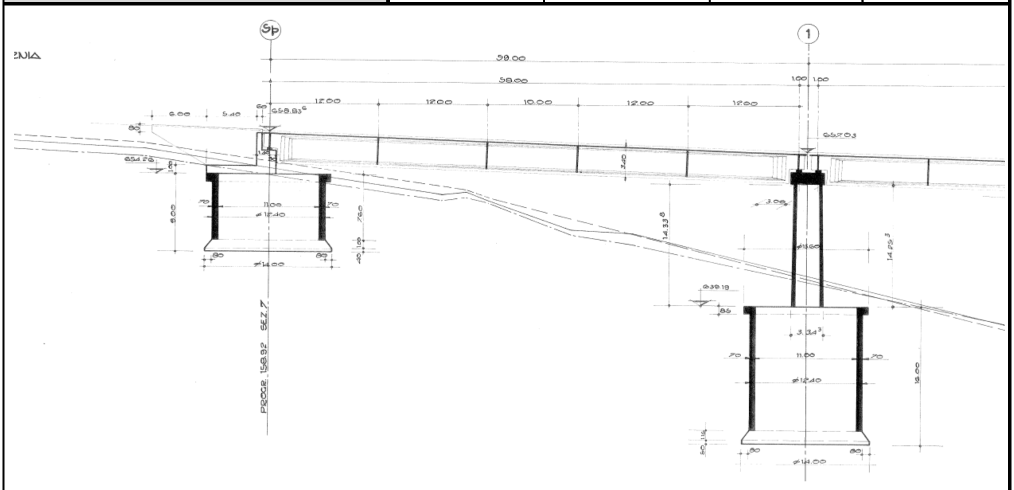
Tab. 2.11 Beams diaphragms and piercaps geometric properties

PIERS					
STRUCTURAL TYPE	One frame			N. FRAMES	1
STRUCTURAL MATERIAL	Reinforced Concrete				
SHAPE SECTION	Rectangular two-cell hollow				
RECTANGULAR	B	L	A	P	G_k
	m	m	mq	daN/m^3	daN/m
CIRCULAR	D_1	D_2	A	P	G_k
	m	m	mq	daN/m^3	daN/m
GENERALLY			A	P	G_k
			mq	daN/m^3	daN/m
VARIABLE BY HEIGHT	A_1	A_2	Am	P	G_k
	mq	mq	mq	daN/m^3	daN/m
	10,205	9,405	9,805	2500	**24513**



Tab. 2.12 Piers geometric properties

FOUNDATIONS				
STRUCTURAL TYPE	Caisson		**PILES NUM**	**0**
STRUCTURAL MATERIAL	Reinforced Concrete			
	B_1	**H_1**	**Area**	**V_1**
	m	**m**	**mq**	**mc**
		16,73	25,73	430,46
	B_2	**H_2**	**L_2**	**V_2**
	m	**m**	**m**	**mc**
		13,2	1,4	191,59
		V	**P**	**G_k**
		mc	**daN/m^3**	**daN/m**
		622,04	2500	1555109,569



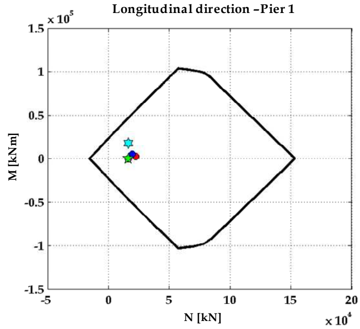
Tab. 2.13 Foundations geometric properties

LOADS CASES ANALYSIS			
DEAD LOADS			
DECK WEIGHT	Gki=	55,18	kN/m
BEAMS WEIGHT	Gktr=	116,94	kN/m
TOTAL SPAN	Gk=	10327	kN
DIAPHRAGMS WEIGHT	Gkt=	652,50	kN
PIER CAP WEIGHT	Gkp=	1851	kN
BEARINGS WEIGHT	Gb=	15,00	kN
TOTAL OVER THE PIER	GK=	12845,28	kN
PIER WEIGHT	GK=	245,13	kN/m
PIER WEIGHT	GK=	6429,63	kN
TOTAL AT THE BASE OF THE PIER	GK=	19274,90	kN
MASS	m=	1506	
LIVE LOADS			
(Circolare n. 384 del 14/02/1962)			
DYNAMIC ALLOWANCE	φ=	1,05	
CIVIL CARGOES OVERLOADS	QKn1=	42,66	kN/m
MILITARY TRUCKS OVERLOADS	QKn2=	43,8	kN/m
PEOPLE COMPACT MASS	QKn3=	4,00	kN/m^2
WIND FOR BRIDGE DISCHARGED	QKv=	2,50	kN/m^2
WIND BRIDGE	QKvs=	1,00	kN/m^2
SNOW	QKn=	0	kN/m^2
CENTRIFUGAL FORCE	Qc=	8,00	kN
BRAKE ACTION	Qf=	128	kN
ACTION ON THE BARRIERS	Qs=	2,50	kN/m
SEISMIC ACTIONS	Qp=	908	kN
TRANSVERSAL MAX ECCENTRICY LOADS	Qe=	2,92	m
ELEMENTS HEIGHT			
$H_{PIER\ CAP}$	1,45 m	H_{DECK}	3,30 m

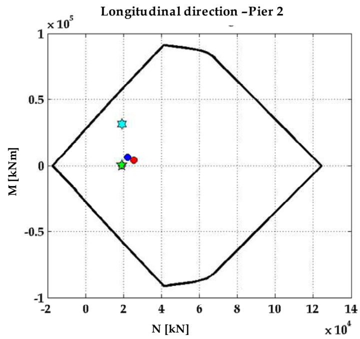
Tab. 2.14 Bridge analysis load cases

BASE PIERS LOADS																											
				1. DEAD LOADS + LIVE LOADS					2. DEAD LOADS +1/2 LIVE LOADS					3. DEAD LOADS +DISCHARGED BRIDGE WIND					4. DEAD LOADS + TRANSVERAL SEISMIC ACTION					5. DEAD LOADS + LONGIDUDINAL SEISMIC ACTION				
PIER	H_{PIER}	N BASI	MASSA	Nsd	Txsd	Tysd	Mxsd	Mysd	Nsd	Txsd	Tysd	Mxsd	Mysd	Nsd	Txsd	Tysd	Mxsd	Mysd	Nsd	Txsd	Tysd	Mxsd	Mysd	Nsd	Txsd	Tysd	Mxsd	Mysd
	m	KN		kN	kN	kN	kNm	kNm	kN	kN	kN	kNm	kNm	kN	kN	kN	kNm	kNm	kN	kN	kN	kNm	kNm	kN	kN	kN	kNm	kNm
PIER 1	14,33	16358	1417	22570	128	563	2442	22693	19464	128	394	4908	13267	16358	0	563	0	9603	16358	0	1145	0	17434	16358	1145	0	17434	0
PIER 2	25,72	19150	1502	25362	128	585	3900	29664	22256	128	416	6365	18314	19150	0	617	0	17408	19150	0	1340	0	31589	19150	1340	0	31589	0
PIER 3	37,62	22067	1591	28279	128	607	5423	37475	25173	128	438	7888	24113	22067	0	674	0	26879	22067	0	1545	0	48756	22067	1545	0	48756	0
PIER 4	43,38	23479	1635	29691	128	618	6160	41449	26585	128	449	8625	27114	23479	0	701	0	31946	23479	0	1644	0	57938	23479	1644	0	57938	0
PIER 5	32,12	20719	1550	26931	128	597	4719	33798	23825	128	428	7184	21366	20719	0	648	0	22334	20719	0	1450	0	40520	20719	1450	0	40520	0
PIER 6	27,19	19510	1513	25723	128	588	4088	30600	22616	128	419	6553	19001	19510	0	624	0	18505	19510	0	1366	0	33579	19510	1366	0	33579	0
PIER 7	17,64	17169	1442	23382	128	570	2865	24668	20276	128	401	5331	14683	17169	0	579	0	11744	17169	0	1202	0	21318	17169	1202	0	21318	0
PIER 8	11,84	15748	1398	21960	128	558	2123	21234	18854	128	389	4589	12229	15748	0	551	0	8061	15748	0	1102	0	14636	15748	1102	0	14636	0

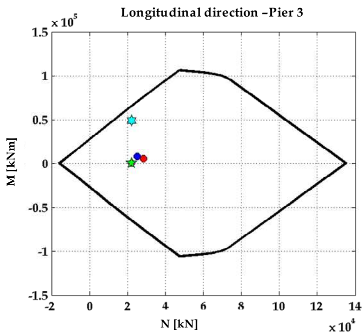
Tab. 2.15 Demands at the base of the piers



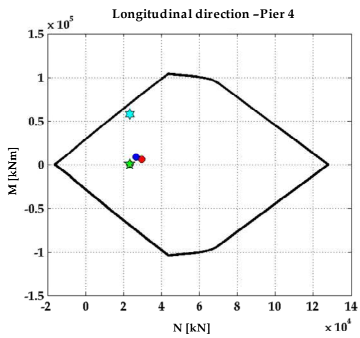
	$\sigma_{c,max}$	$\sigma_{f,max}$	$\sigma_{f,min}$	τ_{max}
Comb. 1	2.520e+000	3.777e+001	3.165e+001	4.467e-002
Comb. 2	2.410e+000	3.608e+001	2.378e+001	4.467e-002
Comb. 3	1.677e+000	2.515e+001	2.515e+001	0.000e+000
Comb. 4	1.677e+000	2.515e+001	2.515e+001	0.000e+000
Comb. 5	3.151e+000	4.700e+001	3.306e+000	3.996e-001



	$\sigma_{c,max}$	$\sigma_{f,max}$	$\sigma_{f,min}$	τ_{max}
Comb. 1	2.831e+000	4.241e+001	3.356e+001	4.135e-002
Comb. 2	2.709e+000	4.056e+001	2.611e+001	4.135e-002
Comb. 3	1.912e+000	2.868e+001	2.868e+001	0.000e+000
Comb. 4	1.912e+000	2.868e+001	2.868e+001	0.000e+000
Comb. 5	4.416e+000	6.567e+001	-3.631e+001	5.406e-001

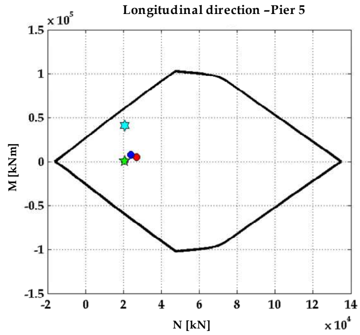


	$\sigma_{c,max}$	$\sigma_{f,max}$	$\sigma_{f,min}$	τ_{max}
Comb. 1	3.119e+000	4.672e+001	3.560e+001	3.819e-002
Comb. 2	2.987e+000	4.473e+001	2.856e+001	3.819e-002
Comb. 3	2.142e+000	3.212e+001	3.212e+001	0.000e+000
Comb. 4	2.142e+000	3.212e+001	3.212e+001	0.000e+000
Comb. 5	6.039e+000	8.956e+001	-1.094e+002	5.362e-001

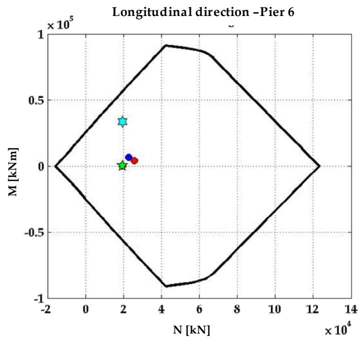


	$\sigma_{c,max}$	$\sigma_{f,max}$	$\sigma_{f,min}$	τ_{max}
Comb. 1	3.248e+000	4.866e+001	3.661e+001	3.685e-002
Comb. 2	3.113e+000	4.662e+001	2.974e+001	3.685e-002
Comb. 3	2.248e+000	3.372e+001	3.372e+001	0.000e+000
Comb. 4	2.248e+000	3.372e+001	3.372e+001	0.000e+000
Comb. 5	6.800e+000	1.008e+002	-1.476e+002	5.350e-001

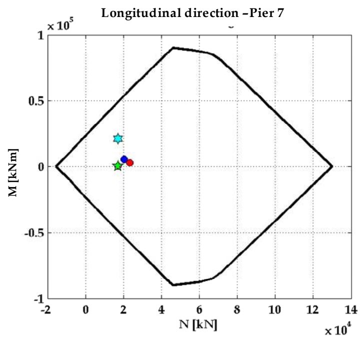
Fig. 2.10 Stress levels of piers for longitudinal load combinations



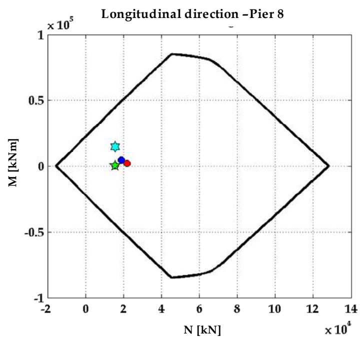
	$\sigma_{c,max}$	$\sigma_{f,max}$	$\sigma_{f,min}$	τ_{max}
Comb. 1	2.988e+000	4.476e+001	3.464e+001	3.954e-002
Comb. 2	2.860e+000	4.282e+001	2.742e+001	3.954e-002
Comb. 3	2.037e+000	3.055e+001	3.054e+001	0.000e+000
Comb. 4	2.037e+000	3.055e+001	3.054e+001	0.000e+000
Comb. 5	5.286e+000	7.848e+001	-7.343e+001	5.402e-001



	$\sigma_{c,max}$	$\sigma_{f,max}$	$\sigma_{f,min}$	τ_{max}
Comb. 1	2.866e+000	4.295e+001	3.380e+001	4.088e-002
Comb. 2	2.743e+000	4.107e+001	2.641e+001	4.088e-002
Comb. 3	1.941e+000	2.911e+001	2.910e+001	0.000e+000
Comb. 4	1.941e+000	2.911e+001	2.910e+001	0.000e+000
Comb. 5	4.612e+000	6.856e+001	-4.412e+001	5.422e-001

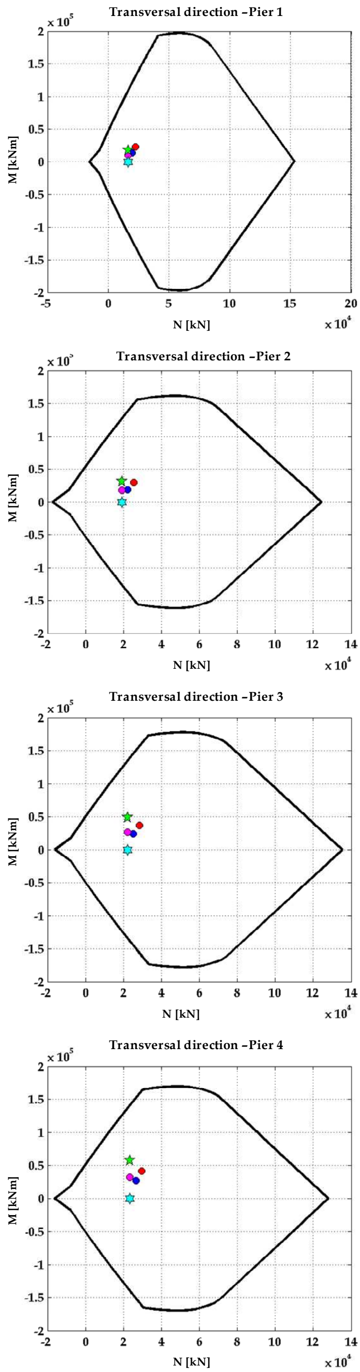


	$\sigma_{c,max}$	$\sigma_{f,max}$	$\sigma_{f,min}$	τ_{max}
Comb. 1	2.616e+000	3.919e+001	3.220e+001	4.371e-002
Comb. 2	2.502e+000	3.746e+001	2.446e+001	4.371e-002
Comb. 3	1.748e+000	2.622e+001	2.620e+001	0.000e+000
Comb. 4	1.748e+000	2.622e+001	2.620e+001	0.000e+000
Comb. 5	3.355e+000	5.001e+001	-2.848e+000	4.341e-001



	$\sigma_{c,max}$	$\sigma_{f,max}$	$\sigma_{f,min}$	τ_{max}
Comb. 1	2.452e+000	3.675e+001	3.128e+001	4.566e-002
Comb. 2	2.346e+000	3.511e+001	2.329e+001	4.566e-002
Comb. 3	1.627e+000	2.440e+001	2.438e+001	0.000e+000
Comb. 4	1.627e+000	2.440e+001	2.438e+001	0.000e+000
Comb. 5	2.899e+000	4.325e+001	5.530e+000	3.931e-001

Fig. 2.11 Stress levels of piers for longitudinal load combinations



Pier 1

	$\sigma_{c,max}$	$\sigma_{f,max}$	$\sigma_{f,min}$	τ_{max}
Comb. 1 ●	3.312e+000	4.963e+001	1.978e+001	1.037e-001
Comb. 2 ●	2.579e+000	3.865e+001	2.120e+001	7.259e-002
Comb. 3 ●	2.100e+000	3.147e+001	1.884e+001	1.037e-001
Comb. 4 ★	2.444e+000	3.662e+001	1.369e+001	4.904e+000
Comb. 5 ✡	1.677e+000	2.515e+001	2.515e+001	0.000e+000

Pier 2

	$\sigma_{c,max}$	$\sigma_{f,max}$	$\sigma_{f,min}$	τ_{max}
Comb. 1 ●	3.787e+000	5.673e+001	1.924e+001	1.069e-001
Comb. 2 ●	2.997e+000	4.491e+001	2.176e+001	7.601e-002
Comb. 3 ●	2.648e+000	3.968e+001	1.768e+001	1.127e-001
Comb. 4 ★	3.248e+000	4.864e+001	8.721e+000	4.899e+000
Comb. 5 ✡	1.912e+000	2.868e+001	2.868e+001	0.000e+000

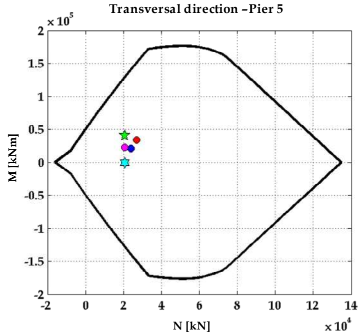
Pier 3

	$\sigma_{c,max}$	$\sigma_{f,max}$	$\sigma_{f,min}$	τ_{max}
Comb. 1 ●	4.264e+000	6.388e+001	1.845e+001	1.100e-001
Comb. 2 ●	3.421e+000	5.126e+001	2.203e+001	7.936e-002
Comb. 3 ●	3.232e+000	4.841e+001	1.583e+001	1.221e-001
Comb. 4 ★	4.119e+000	6.167e+001	2.572e+000	4.895e+000
Comb. 5 ✡	2.141e+000	3.212e+001	3.212e+001	0.000e+000

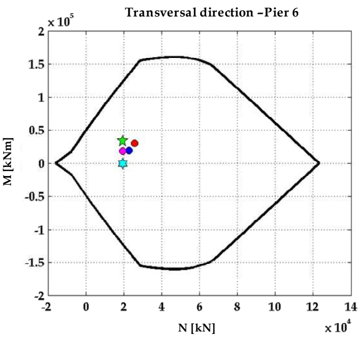
Pier 4

	$\sigma_{c,max}$	$\sigma_{f,max}$	$\sigma_{f,min}$	τ_{max}
Comb. 1 ●	4.489e+000	6.723e+001	1.804e+001	1.115e-001
Comb. 2 ●	3.622e+000	5.427e+001	2.209e+001	8.102e-002
Comb. 3 ●	3.517e+000	5.267e+001	1.476e+001	1.265e-001
Comb. 4 ★	4.400e+000	6.586e+001	-5.500e+000	5.773e+000
Comb. 5 ✡	2.248e+000	3.372e+001	3.372e+001	0.000e+000

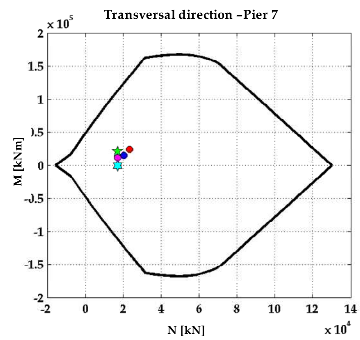
Fig. 2.12 Stress levels of piers for transversal load combinations



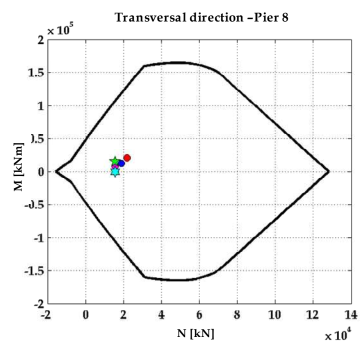
	$\sigma_{c,max}$	$\sigma_{f,max}$	$\sigma_{f,min}$	τ_{max}
Comb. 1	4.042e+000	6.055e+001	1.885e+001	1.085e-001
Comb. 2	3.224e+000	4.830e+001	2.194e+001	7.782e-002
Comb. 3	2.958e+000	4.432e+001	1.677e+001	1.178e-001
Comb. 4	3.709e+000	5.554e+001	5.550e+000	4.895e+000
Comb. 5	2.036e+000	3.054e+001	3.054e+001	0.000e+000



	$\sigma_{c,max}$	$\sigma_{f,max}$	$\sigma_{f,min}$	τ_{max}
Comb. 1	3.845e+000	5.760e+001	1.914e+001	1.073e-001
Comb. 2	3.048e+000	4.568e+001	2.180e+001	7.647e-002
Comb. 3	2.719e+000	4.073e+001	1.748e+001	1.139e-001
Comb. 4	3.353e+000	5.021e+001	8.004e+000	4.898e+000
Comb. 5	1.940e+000	2.910e+001	2.910e+001	0.000e+000



	$\sigma_{c,max}$	$\sigma_{f,max}$	$\sigma_{f,min}$	τ_{max}
Comb. 1	3.455e+000	5.176e+001	1.964e+001	1.048e-001
Comb. 2	2.703e+000	4.051e+001	2.140e+001	7.372e-002
Comb. 3	2.259e+000	3.386e+001	1.857e+001	1.064e-001
Comb. 4	2.676e+000	4.009e+001	1.233e+001	4.904e+000
Comb. 5	1.747e+000	2.621e+001	2.621e+001	0.000e+000



	$\sigma_{c,max}$	$\sigma_{f,max}$	$\sigma_{f,min}$	τ_{max}
Comb. 1	3.212e+000	4.812e+001	1.990e+001	1.032e-001
Comb. 2	2.491e+000	3.733e+001	2.108e+001	7.200e-002
Comb. 3	1.984e+000	2.975e+001	1.903e+001	1.017e-001
Comb. 4	2.277e+000	3.411e+001	1.467e+001	4.907e+000
Comb. 5	1.626e+000	2.439e+001	2.439e+001	0.000e+000

Fig. 2.13 Stress levels of piers for transversal load combinations

	LONGITUDINAL				TRANSVERSAL			
	$\sigma_{c,max}$	$\sigma_{f,max}$	$\sigma_{f,min}$	τ_{max}	$\sigma_{c,max}$	$\sigma_{f,max}$	$\sigma_{f,min}$	τ_{max}
Pier	[Mpa]	[Mpa]	[Mpa]	[Mpa]	[Mpa]	[Mpa]	[Mpa]	[Mpa]
P1	3,15	47,00	31,65	0,40	3,31	49,63	25,15	4,90
P2	4,42	65,67	36,31	0,54	3,79	56,73	28,68	4,90
P3	6,04	89,56	109,44	0,54	4,26	63,88	32,12	4,90
P4	6,80	100,75	147,59	0,54	4,49	67,23	33,72	5,77
P5	5,29	78,48	73,43	0,54	4,04	60,55	30,54	4,90
P6	4,61	68,56	44,12	0,54	3,85	57,60	29,10	4,90
P7	3,35	50,01	32,20	0,43	3,45	51,76	26,21	4,90
P8	2,90	43,25	31,28	0,39	3,21	48,12	24,39	4,91

Tab. 2.16 Stresses at base of the piers for longitudinal and transversal combinations

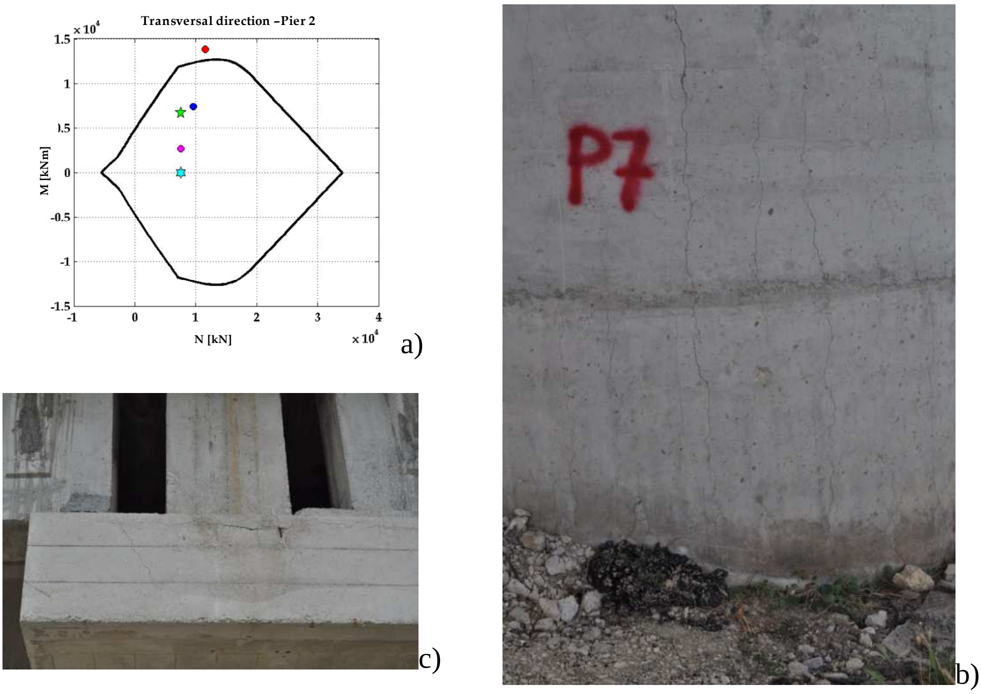


Fig. 2.14 a) N-M interaction domains at the base of the piers; b) longitudinal cracks on the piers; c) transversal cracks at the piercap.

In most cases checks revealed high operational stresses, even if they often fulfill the performance requirements of the Code [NTC (2008)] with a few exceptions. In Fig. 2.14, a sample case is presented, where the results of in-situ investigations are confirmed by analytical checks. The shown pier is characterized by a longitudinal cracking all over the height due to excessive bending stresses caused by large load eccentricity. The evolution of the cracking phenomenon can affect the performance of the structure and speed up the damage evolution.

2.9 CONCLUSIONS

As a first step in seismic vulnerability assessment is the knowledge of the structure. If the structure was built before the Seventies original design drawings can be hardly found. This is even more difficult in the case of infrastructures managed by a few administrations at the regional scale, where the organization of the archives is usually very complex. A previously presented, also in-situ survey could be difficult, and in some case not possible.

So, according to the information required by the DPC for the seismic vulnerability classification and the objectives of the bridge stock analysis, Adequate Level of Knowledge achieved, is satisfactory.

In this contest, simulated design can represent a valuable tool to overcome the issues related to a lack of information. Moreover, results of the simulated design procedures, validated on the basis of calculations found in the archives, showing the existence of high rates of stress for some viaduct, which require attention.

The results of inspections of design drawings and of simulated design allow the development of a rational plan for in-situ investigations. The latter provide the required confirmation of the information reported in the original design drawing and those obtained from simulated design. Moreover, they provide the mechanical properties of materials.

However, in some cases the desired level of knowledge could be not achievable due to economical constraints related to budgeting. Collection and inspection of design documents can be expensive and time consuming in the case of medium to large structures. In such a case, simulated design represent a valuable support to seismic vulnerability analyses. As a consequence, in some cases, due to economic constraints, only a LC1 level of knowledge can be achieved, and the level of complexity of the analysis should be calibrated according to the available information, with an accurate choice of confidence factors.

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Chapter 3

BRIDGE MAGEMENT SYSTEM

3.1 INTRODUCTION AND STATE OF THE ART

In the last decades, attention of relevant Authorities, stakeholders and professional personnel involved in transportation systems management has been mainly paid to develop rational methods able to guide maintenance and ensure serviceability both of structural and non-structural components [Godart, Vassie, (2001)].

Moreover, it is easy to recognize that information needed for seismic vulnerability evaluation of existing bridges are quite complex and need to be calibrated depending on the scope of the analysis.

The main problem concerning of a population of different structures, identified by a bridge stock, is connected to the generalized maintenance and retrofit. A network of bridges contains a lot of structures, which may be identified by the term bridge stock. For a bridge stock, the issue of retrofitting usually derive from inadequacy whit respect to structural national standards for new structures or from inadequacy of maintenance conditions for existing structures. The choice of bridges to be retrofitted, and the retrofitting level, needs a definition of the goals to be achieved: safety, minimum cost, minimum travel time for ordinary maintenance and after an earthquake, are all reasonable and desirable aims. It is not difficult then to imagine how this issue, is often dependent on Government or Public bodies, so is strictly connected to the issues of resource optimization and costs minimization, due to a lack of public resources. This context need for development of faster and accessible assessment process, easily to use by public companies (Bridges Management Systems). This tools aid in the management of bridges stock to determine the intervention on the basis of criteria related to technical and economic considerations [Godart et al., (2001)]. The resources rationalization, need the utilization of decision makers, to prioritizing and selecting bridges needing assessment and retrofitting.

The same problem is strictly connected to the issue of seismic vulnerability of existing infrastructure. In this case, the decision makers are connected to the selection bridges within the inadequate ones, and to the prioritization upgrading. The choice of bridges and of retrofitting level need a definition of objective to achieve: network importance, level of safety, retrofit cost. Different goals need different works studies and different kinds of prioritization. The absolute best process don’t exist, so a large number of different systems are present in literature, useful for the same goal, and for different goals, choosing by different subjects. In any case, prioritization techniques, can be of important help to the decision maker, giving a rotational ranking among bridges, in order to detect the critical ones, and the best upgrading levels. These data should be considered to support choices among alternatives, usually not depending on seismic issues only. BMS systems are able to provide considerations on the status of the stock and priorities on interventions quickly than the sophisticated linear and nonlinear analysis, suitable to study individual works. Therefore the largest approaches to BMS developed in recent years implies a considerable degree of subjectivity, based on visual screening of the structure. This group of methods is useful for owner to general retrofit. At Italian level, actually, available Bridge Management Systems (BMS) are often well defined [Franchetti et al. (2003), Martinello (2005), Montepara et al. (2008), Campitelli (2004)], but cover basically qualitative aspects of the problem, while quantitative aspects related to structural components and detailing is generally incomplete. Effects of earthquakes occurred in some European countries and especially in Italy in the last years modified the perception of the risk, so that a number of actions aimed at assessing the structural and seismic performance of infrastructure more in detail [Boni et al. (2009), Bordot et al. (2009), Pinto et al. (2009)]. The goal is the knowledge of status of corrosion and deterioration of the structure and the prioritization of the assessment retrofit. This prioritization, expertly used, are supporting to decision maker, to investigate critical ones, non strictly depending by seismically issues, but dangerous for structural working. The sensitive degree which are often subjected the infrastructures, due to cyclic loading, alternating whether agents, currents that feed the piers in bed, significantly affects the structure performances, and may still be quantified, providing results in the establishment of intervention priorities in terms of general maintenance and in terms of seismic upgrading also. Different methods are available in literature to check the status of structure preservation. For example in Italian overview, different implementations for numerical estimates of the state of degradation who allows the prioritization for action cases more at risk through the use of numerical indices that allow classification of structures based on the state of degradation [Proverbio et al., (2002), Franchetti et al. (2003)]. The search for specific indicators constitutes a methodology was easily implemented by the owner or the of local managers on the complex issue of the inspection and maintenance [Campitelli, (2004), Martinello,(2005)]. In order to prioritization interventions and maintenance of bridges, more approaches including the importance of the bridge in the network. In this case the approach for priority intervention is not confined only to structural degradation but can understand the need of network functionality, and the rules of the bridge in the network context is strictly connected [Montepara et al. (2008)]. The ANAS s.p.a, regarding in particular the maintenance of own networks, has created SOAWE [ANAS (2009-2010)], a proceedings who arranges the state of the works of art in relation to the degradation based on regular inspections.

It will be possible to elaborate decision algorithms in order to characterize the priorities, reduce the costs and limit maintenance operations.

Possible defects are cataloged based on the handbook available of system SOAWE. Defect gravity is by a weight (1 to 7) that expresses the level of degradation. The defects handbook contains defects papers, with the description of the damage, images, causes and the correlations, the structural range of gravity. The papers will supply, on the base of the result, a correlated final value to the total state of degradation.

Degradation Index will derive from the summary one of the weights attributed to the single defects, multiplied for of the extension and intensity coefficients. The analysis of the result, managed inside SOAWE system by numerical, will concur to program the causes elimination.

In seismic point of view, prioritization methods, useful for seismic vulnerability studies, based on simplify mechanical models employs two input information: seismic hazard (F) and bridges fragilities (R). Many different methods exists in literature. This evaluate structural failure by evaluation based on estimation of structure status of degree, using different criteria multiplied for subjective weight, and physical simplified models of possible structure fragility. More formally the prioritization P_b can be expressed:

$$P_b = f(F_b, R_b) \tag{3.1}$$

with f method dependent function.

Some methods also consider the cost of failure (C), both as direct costs (costs to rebuild the bridge, i.e.) and as indirect costs (costs of construction of alternative networks i.e.), so the previous function became:

$$P_b = f(F_b, R_b, C_b) \tag{3.2}$$

In international literature in possible to find different kind of prioritization methods, both for subjective judgment of the state of the structure, and for the choose of critical models of capacity. Models can be grouped by the followings properties:

model objectiveness in evaluating structural rate of failure via fragility end hazard, based on engineering judgments or the outcome of a physical models, or models based on consequences of structural failure of bridges. Consequences can be computated for a single bridge, or for the network.

Kawashima and Unjoh (1990) collected the damage data relative to 124 bridges for four Japanese seismic events, and develop a method in which vulnerability of i-th bridge depend from hazard and resistance. Property resistance are weighted and summed up, and the weight depend from the damage analyzed. This method was later developed introducing consideration of bridge failure costs [Unjoh (2000)].

Other kinds of methods are solely based on bridge fragility curves. Priority for the i-th bridge is the median value of the bridge fragility curve with respect to a selected limit state [Nielson (2003)].

Fragility can be studies in BMS systems by implementation of simplified mechanical models for simple frame bridges [Dutta and Mander, (1998a)] or multy-frame bridges [DesRoches and Fenves, (2001)].

At Italian level some types of BMS may be regarded as first-level analysis [Petrini, MP Boni (ANIDIS 2009)], based on the implementation of Evaluation Papers, that do not consider geological or structural elements, but based on visual surveys and direct measurements of structure, providing a bases vulnerability of each type of structure analyzed, suitably modified by the analysis of the degradation observed and geometrical irregularities.

Other kind of bridge management system are based on Monte Carlo simulations, based on the prediction of future bridge reliability using a semi-Markov deterioration model [Bordot et al., (2006)].

The prioritization of the bridges is based on the satisfaction of several conflicting objectives simultaneously, including minimum bridge condition ratings, minimum management and retrofitting costs, and maximum average daily traffic. The most relevant objectives include the minimization of the management and retrofitting costs and maximization of the bridge network reliability or condition rating.

For each structural deficiencies, the capacity is determined through simplified models of possible failure mechanisms, and the seismic demand by the site hazard. In this type of models, although simple in nature, also contribute to determining the fragility of the structures investigated specific parameters, such as hazard site, geometrical and mechanical characteristics of the work, structural details. This methods, for simple frame bridges is used, for example, in HAZUS Project, for the seismic risk mitigation.

Are still not taken into account considerations on the actual state of preservation work, assessments which may materially affect the assessment of vulnerability.

Both to conduct linear and nonlinear structures vulnerability studies on a single bridge, and to implement speedily evaluations based on simplified mechanical models, is not possible disregard the analysis of preservation status of the structure. For their own exercise, bridge structures are subjected not just the natural aging, but are exposed to fatigue due to cyclic loading, the alternation of atmospheric agents and, if they cross rivers or canals, static and dynamic actions of the current impact and of solid transport. For example, for RC bridges, these are subject to all three possible causes of degradation: mechanical, chemical and physical.

So is possible to think that the performance of the structure analyzed may be far from those of the same structure in new construction, whit same geological conditions and seismic hazard. A careful analysis of conservation state of and degradation of the structure, allows two goals. The first is to ensure that they are not workings special phenomena that may compromise the structural safety of the work, and for which the seismic capacity of the structure can be greatly cut down, the second is related to the observation of the conservation status structure, and quantification of degradation/corrosion due to natural structure working. This phenomena, if located in particulars elements, or in significant progress, can significantly affect the capacity estimation, and generate crisis for acceleration values lower than those supported by the structure in optimal conditions.

3.2 KNOWLEDGE LEVELS IN EXISTING BRIDGES

Assessment of existing structures and their upgrade is significantly different from new constructions. According to OPCM 3274 (2003) or to Eurocode 8 [CEN (2005)], a knowledge level has to be preliminarily evaluated in order to define material properties for existing buildings and consequently to define a correct structural analysis and assessment design procedure.

In this context, and both in maintenance and seismically point of view, the estimation of degradation level, with indicial parameters summarized in special Degradation Assessment Papers, allows the knowledge of the real functionality of the structure. Those findings meet two objectives: general maintenance management and, if utilized to support fragility construction, analysis of real seismic vulnerability.

For seismically vulnerability evaluation, determination of structural capacity values is required. In this context, irrespective of the nature and sophistication of the analytical method used, is necessary to define all the possible mechanisms, to characterize the evolution of the structure, in post elastic range, through different limit states. Not good structure preservation could significantly influence structural response. In this case, really structural behavior, could be different then estimated in calculation based on original design. Same example is showed in pictures below, and briefly discussed.

For example, if bearings are not correctly positioned or is damaged, the length of support calculated by original design, could be affected by error. In case of longitudinal and confinement bars not homogeneous in diameter or in distance, different by original design, or not effective, there could be error in flexural- shear ratio evaluation. Wrong evaluation of flexural- shear interaction, cause errors in classifications in ductile or brittle element.

Furthermore of longitudinal and confinement bars not effective, could cause overestimation in element ultimate strengths assessment, because don't reflect the real performance of the element, so may be that the analysis is a disadvantage of security. Same problems in stiffness evaluations could be present when concrete is cracked or very deteriorated.

Fig. 3.1 Discovered bars in piercap

Fig. 3.2 Defects on bearings.

Fig. 3.3 Defect of confinement bars, not effective.

Fig. 3.4 Diffused and significant longitudinal cracks in pier

Problem about foundations, could be present when bridge are located near river bed, and water transport discover the foot. In this case, load-bearing capacity is lower than design estimation, and plastic zone can involve at the head of piles instead of at the base of the piers.

Fig. 3.5 Foundation discovered

3.3 STRUCTURAL CONDITION ASSESSMENT

The brief discussion of KLs required by seismic codes for the assessment of existing bridges points out the relevant role of some components of the bridge system. They are actually a sub-set of all components considered in maintenance and management of road networks. In fact, bridges age, deterioration caused by heavy traffic and critical environment conditions result in a higher frequency of repairs and can impact a reduced load carrying capacity. This circumstance leads to take decisions on maintenance works generally based on inspections and engineering judgment, but also to collect a relevant amount of information and data that need to be integrated in the seismic vulnerability assessment process.

Degradation and maintenance process are critical for the definition of reliable tools to support inspections for KL achievement as well as to support decisions for seismic upgrading of vulnerable constructions.

As a result, a review of available Bridge Management Systems (BMS) appeared the basic step for the definition of a dataset able to describe the status of the bridge from a seismic standpoint and the components to be assessed and tracked during the service life of the structure. In this sense, a similar process seems to fit the requirements of the design of structural health monitoring systems and perform an integration between safety demand for users and an integrated sustainability of constructions.

Table 3.1 represents the basic matrix that links the different class of elements that play a role in the development of the seismic performance of the bridge and the defects that can be observed. Each defect can be associated to a weight W, variable 1 to 5, depending on its impact on seismic and structural performances [Martinello (2005)].

Some aspects that lead to the definition of the weight W are here reported:

- Defect develops and constitute a risk (risk present);
- Defect can affect the load capacity (risk potential);
- Defect can trigger other malfunctions and/or damage to surrounding areas (induced risk);
- Defect can trigger relevant economic losses due to repairing and upgrading (economic risk).

It is worth noting that each defect can have different influence for the class of elements considered, so it’s possible that the same degradation has different weight in each structural class.

Defects \ Component	Deck	Girder	Diaphragm	Pier cap	Pier	Abutment
No damage	0	0	0	0	0	0
Damp patch	1	1	1	1	1	1
Deteriorated concrete/crawl	2	2	2	3	4	3
Corroded/deformed longitudinal bars	5	5	4	5	5	5
Longitudinal cracks	2	2	2	2	-	2
Transverse cracks	5	-	-	-	2	-
Cracks at the beam to slab connection	2	3	-	-	-	-
Transverse/diagonal cracks	-	5	5	5	3	5
Confinement bars exposed/corroded	-	5	-	5	5	5
Longitudinal/diagonal cracks	-	-	-	-	5	-
Cracks at the pier cap connection	-	-	-	-	2	-
Cracks at the beam to diaphragm connection	-	3	-	-	-	-
Head beam bars exposed/corroded	-	5	-	-	-	-
Damage induced by supports defects	-	-	-	3	-	-
Defects in neoprene supports	-	-	-	3	-	-
Out of plumb	-	-	-	-	-	5

Tab 3.1 Summary of relevant components and related defects

Severity Level	ID.	Short–term consequences	S
L	Low	No	1
M	Medium	Functional	2
H	High	Structural	5
Diffusion Level	ID.	Frequency of occurrence	F
F1	Limited	Minor	1
F2	Medium	Moderate	2
F3	Extended	Extreme	3
Extension Level	ID.	Extension of defect	E
E1	Limited	Minor	1
E2	Medium	Moderate	2
E3	Extended	Extreme	3

Tab 3.2 Severity, Diffusion and Extension level

The level of reliability of the data is obviously related to the number of the class elements directly investigated.

As a consequence, the KL defined according to Code provisions or according to the available data can be expressed as the ratio between the number of inspected elements, N_{insp}, and the total number of elements, N_{max}, present in the bridge:

$$k = \frac{N_{insp}}{N_{\max}} \tag{3.3}$$

Similarly, the severity level of each kind of degradation, its diffusion along the bridge and its local extension have to be estimated and a parametric representation is needed [Montepara et al. (2008)]. Severity Level can be estimated depending on its short-time consequences. It is high if the damage can progress in a structural failure, average if the damage can lead to functional failure, low if the probability of damage is negligible resulting in no short-term consequences. The Diffusion Level can be associated to the frequency of occurrence of the anomaly. If the phenomenon is limited, confined in a few locations and no more than 25% of the extension of the damage elements, medium if it affects an area between 25% and 75%, widespread almost the entire class of observed elements exhibits the degradation of interest. The Level of Extension refers to the area of each element affected by each anomaly. Table 3.2 summarizes the above mentioned levels for the anomaly definition and provide their quantitative evaluation given by the parameter S, F, and E. This parameters are the factors needed for the calculation of the Element Structural Condition Index (ESCI). It can be defined for each element class, as the sum of Level of Severity, Spread and Extension, increased of the assigned weight, for all the damaged observed:

$$ESCI = \sum_{1}^{i} (W_i \cdot S_i \cdot F_i \cdot E_i) \tag{3.4}$$

The ESCI can be associated only of the elements directly observed on site, whose number can change between a bridge and another in the same stock.

An homogenization of the indexes can be based on the introduction of the reliability of the information depending on the level of knowledge KL.

In fact, since a number of elements cannot be inspected or their data are not available at the moment of the assessment, the frequency of observation can be used to weight the ESCI index based on the inspected elements using the factor FO given by the ratio:

$$FO = \frac{1}{k} \tag{3.5}$$

where k depends on between the number of inspected elements, N_{insp}, and the total number of elements, N_{max}, and the reliability of information is referred to the minimum requirements for different levels of inspection and testing (§Tab. 2.5).

For each class of elements Global Structural Condition Index can be expressed by the equation:

$$GSCI = ESCI \cdot IF \cdot AF \cdot FO \tag{3.6}$$

It is based on ESCI index, but it is corrected taking into account the reliability of information's FO, rank of the structural class within the bridge – via the IF, Importance Factor, of the structural class – and the age of the bridge using the Age Factor AF [Franchetti et al. (2005)]. Table 3.3 reports the selection criteria adopted for AF and IF factors.

For each class of elements of the viaduct, is estimated both the ESCI and GSCI.

After the evaluation of the ESCI and GSCI for the different classes of elements, they can be used both for the single structure or the entire stock.

Elements	IF	Bridge age	AF
Deck	0,8	Before 1900	1,05
Beam	1	1900-1940	1,00
Intermediate diaphragm	0,7	1941-1965	0,97
Pier cap	0,9	1965-1980	0,95
Pier	1	1981-2005	0,90
Abutment	0,8	2005 and later	0,85

Tab 3.3 Importance Factor and Bridge Age Factor

The indexes calculated, can be normalized using the absolute maximum value to facilitate the comparison, as part of the same opera, and other same classes indices the case of classification of a population of bridges. Therefore, the relevant indexes turn in:

$$\overline{ESCI} = \frac{ESCI}{ESCI_{MAX}} \cdot 100 \tag{3.7}$$

$$\overline{GSCI} = \frac{GSCI}{GSCI_{MAX}} \cdot 100 \tag{3.8}$$

If a single structure is concerned, it is possible to assess the maintenance state of each class of elements and the presence of localized problems. Then, maintenance can be carried out for selected classes of elements and developing phenomena requiring restoration interventions can be identified. If the management of the road network is of interest, the approach allows the comparison of the states of the different classes of structures and a global ranking of the structures.

3.4 STRUCTURAL CONDITION INDEXES RELEVANT RANGES

The framework presented in the previous sections leads to identify the main issues related to the maintenance level and the structural characterization of different bridge components. It provides a quantitative formulation of the observed deterioration states and is strictly dependent upon reliability of information. Combined evaluation of number of deterioration, the sum and average of severity provide an outlook on the nature of ESCI and GSCI parameters. Relation between the above mentioned estimates of the bridge condition is able to mark the nature of structural degradation. Moderate and spread damage, as well as active critical mechanisms with short-term effects can be identified. This information is certainly of interest for effective management of the infrastructure stock, for management of economic and technical resources, and planning of assessment and retrofitting. ESCI is able to describe the condition of each bridge class of components and is related only to inspections outcomes. Then, maintenance can be carried out for selected classes of elements and developing phenomena requiring maintenance and upgrading interventions can be identified as well. Moreover, ESCI makes possible a direct comparison between the maintenance needs of the elements in the same viaduct. For each elements class, ranges of intervention priority classification depends of number of damage found. ESCI values greater than 50 are not usual, as shown by a number of simulations carried out on a relevant number of real cases. In fact, a careful review of the defects catalog, and the possibility of high diffusion and extension, ESCI high values correspond to a so level of deterioration and damage that are not compatible with a safe service of the structure.

Number of damage observed	ESCI	Consequences
	0< ESCI <5	No effects
1-2	5≤ ESCI <15	Functional
	ESCI >15	Structural
	0< ESCI<10	No effects
3-4	10≤ ESCI <25	Functional
	ESCI >25	Structural
	0< ESCI <15	No effects
>5	15≤ ESCI <30	Functional
	ESCI >30	Structural

Tab 3.4 ESCI relevant ranges and performance thresholds

Number of damage observed	GSCI	Consequences
	0< GSCI <3	No effects
1-2	3≤ GSCI <10	Functional
	GSCI >10	Structural
	0< GSCI<5	No effects
3-4	5≤ GSCI <10	Functional
	GSCI >10	Structural
	0< GSCI <7	No effects
>5	7≤ GSCI <15	Functional
	GSCI >15	Structural

Tab 3.5 GSCI relevant ranges and performance thresholds

GSCI can be used to compare at a higher level the structure and take account all the main features and defects of the bridge within a stock. When the single structure is concerned, it is possible to assess the maintenance state of each class of elements and the presence of localized problems. If the management of the road network is of interest, the approach allows the comparison of the states of the different classes of structures and a global ranking of the structures.

A summary of typical ESCI and GSCI ranges are reported in Table 3.4 and Table 3.5 depending on the number of relevant observed deterioration phenomena, so that typical performance thresholds can be derived.

3.5 DEGRADATION ASSESSMENT PAPERS FOR AN EXISTING BRIDGE

In the following tables, there is an examples of Degradation Assessment Papers completed for ax existing bridge. The bridge structure is not uniform. The geographic bridge position, across the Trigno river valley does not allow all parts inspection. The abutments are not of the all reached.

Having visited 15 spans on 21, is possible to consider as the reliability on information the achievement of KL3 Level [NTC (2008)]. The bridge is affected by several damage, at the deck, beams, pier cap an pier, for most lied to the humidity phenomena, and to the cycling traffic loads.

The water presence, whit cyclic ice phenomena, has produces discovering of the bars al beams and pier caps level whit consequence of bars oxidation and size reduction. Longitudinal and transversal bars, in some section are not effective.

The spread and extension of degradation phenomena, could reduce mechanical performance of the complex, and also impact of the structural maintenance and the progress of the degradation phenomena.

At the base of the pier, diffused phenomena of expulsion of concrete cover are present. Confinement bars are oxidized and have reduces sections, so aren’t efficacies. Consequently, longitudinal bars, are discovered and affected by buckling phenomena. This kind of damage, probably correlated to the insufficient thickness of concrete cover, has structural effects on the mechanical performances both in static and in seismic point of view. Abutments are not visible, because in not possible to go near then or in the car or on foot.

STRUCTURAL ELEMENT					DECK
TOTAL NUMBER OF ELEMENTS					21
NUMBER OF ELEMENTS INVESTIGATED					15
PERCENTAGE OF INVESTIGATED ELEMENTS					71%
LEVEL OF RELIABILITY					LC3
RANK OF THE STRUCTURAL CLASS IN BRIDGE				(IF)	0,8
FREQUENCY OF OBSERVATION				(FO)	1,40
AGE OF THE BRIDGE				(AF)	0,95
NOTES:					
DEFECT			Damp patch	W	1
SEVERITY LEVEL	L	ID.	Low	VALUE	1
DIFFUSION LEVEL	F1	ID.	Limited	VALUE	1
EXTENSION LEVEL	E1	ID.	Limited	VALUE	1
DEFECT			Deteriorated concrete/crawl	W	2
SEVERITY LEVEL	M	ID.	Medium	VALUE	2
DIFFUSION LEVEL	F1	ID.	Limited	VALUE	1
EXTENSION LEVEL	E3	ID.	Extended	VALUE	3
DEFECT			Corroded/deformed long. bars	W	5
SEVERITY LEVEL	M	ID.	Medium	VALUE	2
DIFFUSION LEVEL	F1	ID.	Limited	VALUE	1
EXTENSION LEVEL	E3	ID.	Extended	VALUE	3

STRUCTURAL ELEMENT				BEAMS		
TOTAL NUMBER OF ELEMENTS						63
NUMBER OF ELEMENTS INVESTIGATED						45
PERCENTAGE OF INVESTIGATED ELEMENTS						71%
LEVEL OF RELIABILITY						LC3
RANK OF THE STRUCTURAL CLASS IN BRIDGE					(IF)	1
FREQUENCY OF OBSERVATION					(FO)	1,40
AGE OF THE BRIDGE					(AF)	0,95
NOTES :						
DEFECT			Damp patch		W	1
SEVERITY LEVEL	H	ID.	Higt	VALUE		5
DIFFUSION LEVEL	F1	ID.	Limited	VALUE		1
EXTENSION LEVEL	E2	ID.	Medium	VALUE		2
DEFECT			Deteriorated concrete/crawl		W	2
SEVERITY LEVEL	H	ID.	Higt	VALUE		5
DIFFUSION LEVEL	F1	ID.	Limited	VALUE		1
EXTENSION LEVEL	E2	ID.	Medium	VALUE		2
DEFECT			Corroded/deformed long. bars		W	5
SEVERITY LEVEL	H	ID.	Higt	VALUE		5
DIFFUSION LEVEL	F1	ID.	Limited	VALUE		1
EXTENSION LEVEL	E2	ID.	Medium	VALUE		2
DEFECT			Head girder bars exposed/corroded		W	5
SEVERITY LEVEL	H	ID.	Higt	VALUE		5
DIFFUSION LEVEL	F1	ID.	Limited	VALUE		1
EXTENSION LEVEL	E2	ID.	Medium	VALUE		2

STRUCTURAL ELEMENT		DIAPHRAGM
TOTAL NUMBER OF ELEMENTS		210
NUMBER OF ELEMENTS INVESTIGATED		150
PERCENTAGE OF INVESTIGATED ELEMENTS		71%
LEVEL OF RELIABILITY		LC3
RANK OF THE STRUCTURAL CLASS IN BRIDGE	(IF)	0,7
FREQUENCY OF OBSERVATION	(FO)	1,40
AGE OF THE BRIDGE	(AF)	0,95
NOTES:		

DEFECT			Damp patch	W	1
SEVERITY LEVEL	L	ID.	Low	VALUE	1
DIFFUSION LEVEL	F1	ID.	Limited	VALUE	1
EXTENSION LEVEL	E1	ID.	Limited	VALUE	1
DEFECT			Deteriorated concrete/crawl	W	2
SEVERITY LEVEL	L	ID.	Low	VALUE	1
DIFFUSION LEVEL	F1	ID.	Limited	VALUE	1
EXTENSION LEVEL	E1	ID.	Limited	VALUE	1

STRUCTURAL ELEMENT				PIER CAP		
TOTAL NUMBER OF ELEMENTS						20
NUMBER OF ELEMENTS INVESTIGATED						15
PERCENTAGE OF INVESTIGATED ELEMENTS						75%
LEVEL OF RELIABILITY						LC3
RANK OF THE STRUCTURAL CLASS IN BRIDGE					(IF)	0,9
FREQUENCY OF OBSERVATION					(FO)	1,33
NOTES:						
DEFECT			Damp patch		W	1
SEVERITY LEVEL	M	ID.	Medium	VALUE		2
DIFFUSION LEVEL	F2	ID.	Medium	VALUE		2
EXTENSION LEVEL	E2	ID.	Medium	VALUE		2
DEFECT			Deteriorated concrete/crawl		W	3
SEVERITY LEVEL	H	ID.	Higt	VALUE		5
DIFFUSION LEVEL	F2	ID.	Medium	VALUE		2
EXTENSION LEVEL	E2	ID.	Medium	VALUE		2
DEFECT			Corroded/deformed long. bars		W	5
SEVERITY LEVEL	H	ID.	Higt	VALUE		5
DIFFUSION LEVEL	F2	ID.	Medium	VALUE		2
EXTENSION LEVEL	E2	ID.	Medium	VALUE		2
DEFECT			Confinement bars exposed/corroded		W	5
SEVERITY LEVEL	H	ID.	Higt	VALUE		5
DIFFUSION LEVEL	F2	ID.	Medium	VALUE		2
EXTENSION LEVEL	E2	ID.	Medium	VALUE		2

STRUCTURAL ELEMENT		PIER
TOTAL NUMBER OF ELEMENTS		20
NUMBER OF ELEMENTS INVESTIGATED		15
PERCENTAGE OF INVESTIGATED ELEMENTS		75%
LEVEL OF RELIABILITY		LC3
RANK OF THE STRUCTURAL CLASS IN BRIDGE	(IF)	1
FREQUENCY OF OBSERVATION	(FO)	1,33
AGE OF THE BRIDGE	(AF)	0,95
NOTES:		

DEFECT			Damp patch	W	1
SEVERITY LEVEL	L	ID.	Low	VALUE	1
DIFFUSION LEVEL	F1	ID.	Limited	VALUE	1
EXTENSION LEVEL	E1	ID.	Limited	VALUE	1
DEFECT			Deteriorated concrete/crawl	W	4
SEVERITY LEVEL	H	ID.	Higt	VALUE	5
DIFFUSION LEVEL	F1	ID.	Limited	VALUE	1
EXTENSION LEVEL	E2	ID.	Medium	VALUE	2
DEFECT			Corroded/deformed long. bars	W	5
SEVERITY LEVEL	H	ID.	Higt	VALUE	5
DIFFUSION LEVEL	F2	ID.	Medium	VALUE	2
EXTENSION LEVEL	E2	ID.	Medium	VALUE	2
DEFECT			Confinement bars exposed/corroded	W	5
SEVERITY LEVEL	H	ID.	Higt	VALUE	5
DIFFUSION LEVEL	F2	ID.	Medium	VALUE	2
EXTENSION LEVEL	E2	ID.	Medium	VALUE	2

STRUCTURAL ELEMENT		ABUTMENT
TOTAL NUMBER OF ELEMENTS		2
NUMBER OF ELEMENTS INVESTIGATED		0
PERCENTAGE OF INVESTIGATED ELEMENTS		0%
LEVEL OF RELIABILITY		LC1
RANK OF THE STRUCTURAL CLASS IN BRIDGE	(IF)	0,8
FREQUENCY OF OBSERVATION	(FO)	0,00
AGE OF THE BRIDGE	(AF)	0,95
NOTES:		

DEFECT			No damage	W	0
SEVERITY LEVEL	L	ID.	Low	VALUE	1
DIFFUSION LEVEL	F1	ID.	Limited	VALUE	1
EXTENSION LEVEL	E1	ID.	Limited	VALUE	1

In Table 3.6 numerical index results are show. The ESCI and GSCI indices allows the knowledge and the prioritization of maintenance in the bridge. Using the indices, in accordance whit total and average of severity is possible to identify damage nature, and decide maintenance economic plans.

	DECK	BEAM	DIAPH.	P. CAP	PIER	ABOUT.
LEVEL OF RELIABILITY	LC3	LC3	LC3	LC3	LC3	NV
NUMBER OF DEFECTS	3 /6	4 /8	2 /5	4 /7	4 /7	0 /6
TOTAL SEVERITY FOUND	8 /17	13 /28	3 /15	14 /21	15 /20	0 /20
SEVERITY AVERAGE	2,67	3,25	1,50	3,50	3,75	0,00
ELEMENT STRUCTURAL CONDITION INDEX	43	130	3	268	241	0
GLOBAL STRUCTURAL CONDITION INDEX	46	173	3	306	305	0
ESCI (0-100)	5,62	9,32	0,48	22,06	22,31	0,00
GSCI (0-100)	3,15	5,22	0,27	11,76	11,90	0,00

Tab 3.6 BMS numerical index results for an existing bridge

In this case, considering only one viaduct, ESCI Index are discussed. For deck and diaphragms, low Index values reveal diffused degradation lied to not good structural maintenance. For beams high values of number and average of defects, as associated with a mean value ESCI, denote a condition of damage who can involve whit structural consequences, but localized. In fact, for beams, defects are concentrated at the external bulbs.

For piercaps and piers, ESCI index, significantly larger than the other, show problem who could have involve whit structural consequences in short times, so need urgent restoration.

Moreover, at network level, index could be used as decision makers to identify problems and consequences for same class of elements in different bridges, and different element class in the same bridge.

The presented Bridge Management System were prepared for a population of existing bridges. Degradation Assessment Papers were completed based on visual inspections and investigations, carried out for each bridge. Degradation Assessment Papers are omitted, but in the following tables, only some index results are shown.

	DECK	GIRDER	DIAPH.	P. CAP	PIER	ABOUT.
LEVEL OF RELIABILITY	LC3	LC3	LC3	LC3	LC3	LC2
NUMBER OF DEFECTS	1 /6	0 /8	0 /5	3 /7	1 /7	2 /6
TOTAL SEVERITY FOUND	1 /17	0 /28	0 /15	7 /21	5 /20	4 /20
SEVERITY AVERAGE	1,00	0,00	0,00	2,33	5,00	2,00
ELEMENT STRUCTURAL CONDITION INDEX	9	0	0	38	100	4
GLOBAL STRUCTURAL CONDITION INDEX	9	0	0	43	127	6
ESCI (0-100)	1,18	0,00	0,00	3,13	9,26	0,34
GSCI (0-100)	0,61	0,00	0,00	1,67	4,94	0,27
	DECK	**GIRDER**	**DIAPH.**	**P. CAP**	**PIER**	**ABOUT.**
LEVEL OF RELIABILITY	LC3	LC3	LC3	LC3	LC3	LC3
NUMBER OF DEFECTS	3 /6	0 /8	0 /5	1 /7	0 /7	0 /6
TOTAL SEVERITY FOUND	8 /17	0 /28	0 /15	3 /21	0 /20	0 /20
SEVERITY AVERAGE	2,67	0,00	0,00	3,00	0,00	0,00
ELEMENT STRUCTURAL CONDITION INDEX	18	0	0	90	0	0
GLOBAL STRUCTURAL CONDITION INDEX	14	0	0	77	0	0
ESCI (0-100)	2,35	0,00	0,00	7,41	0,00	0,00
GSCI (0-100)	0,94	0,00	0,00	2,96	0,00	0,00
	DECK	**GIRDER**	**DIAPH.**	**P. CAP**	**PIER**	**ABOUT.**
LEVEL OF RELIABILITY	LC3	LC3	LC3	LC3	LC3	LC3
NUMBER OF DEFECTS	3 /6	0 /8	0 /5	2 /7	0 /7	0 /6
TOTAL SEVERITY FOUND	8 /17	0 /28	0 /15	10 /21	0 /20	0 /20
SEVERITY AVERAGE	2,67	0,00	0,00	5,00	0,00	0,00
ELEMENT STRUCTURAL CONDITION INDEX	18	0	0	95	0	0
GLOBAL STRUCTURAL CONDITION INDEX	14	0	0	81	0	0
ESCI (0-100)	2,35	0,00	0,00	7,82	0,00	0,00
GSCI (0-100)	0,94	0,00	0,00	3,13	0,00	0,00
	DECK	**GIRDER**	**DIAPH.**	**P. CAP**	**PIER**	**ABOUT.**
LEVEL OF RELIABILITY	LC3	LC3	LC3	LC3	LC3	LC3
NUMBER OF DEFECTS	3 /6	5 /8	1 /5	4 /7	4 /7	4 /6
TOTAL SEVERITY FOUND	8 /17	18 /28	1 /15	14 /21	19 /20	11 /20
SEVERITY AVERAGE	2,67	3,60	1,00	3,50	4,75	2,75
ELEMENT STRUCTURAL CONDITION INDEX	18	172	2	215	269	183
GLOBAL STRUCTURAL CONDITION INDEX	14	163	1	184	256	139
ESCI (0-100)	2,35	12,33	0,32	17,70	24,91	15,64
GSCI (0-100)	0,94	4,93	0,13	7,08	9,96	6,26
	DECK	**GIRDER**	**DIAPH.**	**P. CAP**	**PIER**	**ABOUT.**
LEVEL OF RELIABILITY	LC3	LC3	LC3	LC3	LC3	LC3
NUMBER OF DEFECTS	0 /6	0 /8	0 /5	3 /7	2 /7	2 /6
TOTAL SEVERITY FOUND	0 /17	0 /28	0 /15	9 /21	9 /20	4 /20
SEVERITY AVERAGE	0,00	0,00	0,00	3,00	4,50	2,00
ELEMENT STRUCTURAL CONDITION INDEX	0	0	0	51	29	48
GLOBAL STRUCTURAL CONDITION INDEX	0	0	0	44	28	36
ESCI (0-100)	0,00	0,00	0,00	4,20	2,69	4,10
GSCI (0-100)	0,00	0,00	0,00	1,68	1,07	1,64

	DECK	GIRDER	DIAPH.	P. CAP	PIER	ABOUT.
LEVEL OF RELIABILITY	LC3	LC3	LC3	LC3	LC3	LC3
NUMBER OF DEFECTS	2 /6	2 /8	0 /5	3 /7	3 /7	2 /6
TOTAL SEVERITY FOUND	3 /17	6 /28	0 /15	9 /21	10 /20	4 /20
SEVERITY AVERAGE	1,50	3,00	0,00	3,00	3,33	2,00
ELEMENT STRUCTURAL CONDITION INDEX	16	18	0	69	65	48
GLOBAL STRUCTURAL CONDITION INDEX	15	21	0	59	62	36
ESCI (0-100)	2,09	1,29	0,00	5,68	6,02	4,10
GSCI (0-100)	1,05	0,65	0,00	2,27	2,41	1,64
	DECK	GIRDER	DIAPH.	P. CAP	PIER	ABOUT.
LEVEL OF RELIABILITY	LC3	LC3	LC3	LC3	LC3	LC3
NUMBER OF DEFECTS	3 /6	0 /8	0 /5	4 /7	0 /7	1 /6
TOTAL SEVERITY FOUND	8 /17	0 /28	0 /15	14 /21	0 /20	1 /20
SEVERITY AVERAGE	2,67	0,00	0,00	3,50	0,00	1,00
ELEMENT STRUCTURAL CONDITION INDEX	8	0	0	630	0	12
GLOBAL STRUCTURAL CONDITION INDEX	6	0	0	539	0	9
ESCI (0-100)	1,05	0,00	0,00	51,85	0,00	1,03
GSCI (0-100)	0,42	0,00	0,00	20,74	0,00	0,41
	DECK	GIRDER	DIAPH.	P. CAP	PIER	ABOUT.
LEVEL OF RELIABILITY	LC3	LC3	LC3	LC3	LC3	LC3
NUMBER OF DEFECTS	3 /6	1 /8	3 /5	3 /7	2 /7	1 /6
TOTAL SEVERITY FOUND	8 /17	5 /28	7 /15	9 /21	5 /20	1 /20
SEVERITY AVERAGE	2,67	5,00	2,33	3,00	2,50	1,00
ELEMENT STRUCTURAL CONDITION INDEX	8	225	14	330	50	18
GLOBAL STRUCTURAL CONDITION INDEX	6	214	9	282	48	14
ESCI (0-100)	1,05	16,13	2,22	27,16	4,63	1,54
GSCI (0-100)	0,42	6,45	0,89	10,86	1,85	0,62
	DECK	GIRDER	DIAPH.	P. CAP	PIER	ABOUT.
LEVEL OF RELIABILITY	LC3	LC3	LC3	LC3	LC3	LC3
NUMBER OF DEFECTS	3 /6	4 /8	3 /5	4 /7	3 /7	2 /6
TOTAL SEVERITY FOUND	8 /17	13 /28	7 /15	14 /21	10 /20	4 /20
SEVERITY AVERAGE	2,67	3,25	2,33	3,50	3,33	2,00
ELEMENT STRUCTURAL CONDITION INDEX	77	405	14	410	14	45
GLOBAL STRUCTURAL CONDITION INDEX	59	385	9	351	13	34
ESCI (0-100)	10,07	29,03	2,22	33,74	1,30	3,85
GSCI (0-100)	4,03	11,61	0,89	13,50	0,52	1,54
	DECK	GIRDER	DIAPH.	P. CAP	PIER	ABOUT.
LEVEL OF RELIABILITY	LC3	LC3	LC3	LC3	LC3	LC1
NUMBER OF DEFECTS	3 /6	4 /8	3 /5	4 /7	4 /7	1 /6
TOTAL SEVERITY FOUND	8 /17	13 /28	7 /15	14 /21	15 /20	1 /20
SEVERITY AVERAGE	2,67	3,25	2,33	3,50	3,75	1,00
ELEMENT STRUCTURAL CONDITION INDEX	22	130	28	232	39	12
GLOBAL STRUCTURAL CONDITION INDEX	25	185	28	317	59	0
ESCI (0-100)	2,88	9,32	4,44	19,09	3,61	1,03
GSCI (0-100)	1,73	5,59	2,67	12,22	2,31	0,00

	DECK	GIRDER	DIAPH.	P. CAP	PIER	ABOUT.
LEVEL OF RELIABILITY	LC3	LC3	LC3	LC3	LC3	NV
NUMBER OF DEFECTS	3 /6	4 /8	2 /5	4 /7	4 /7	0 /6
TOTAL SEVERITY FOUND	8 /17	13 /28	3 /15	14 /21	15 /20	0 /20
SEVERITY AVERAGE	2,67	3,25	1,50	3,50	3,75	0,00
ELEMENT STRUCTURAL CONDITION INDEX	22	130	3	268	217	0
GLOBAL STRUCTURAL CONDITION INDEX	23	173	3	306	275	0
ESCI (0-100)	2,88	9,32	0,48	22,06	20,09	0,00
GSCI (0-100)	1,61	5,22	0,27	11,76	10,72	0,00
	DECK	**GIRDER**	**DIAPH.**	**P. CAP**	**PIER**	**ABOUT.**
LEVEL OF RELIABILITY	LC3	LC3	LC3	LC3	LC3	LC2
NUMBER OF DEFECTS	3 /6	1 /8	2 /5	4 /7	0 /7	1 /6
TOTAL SEVERITY FOUND	8 /17	1 /28	3 /15	14 /21	0 /20	1 /20
SEVERITY AVERAGE	2,67	1,00	1,50	3,50	0,00	1,00
ELEMENT STRUCTURAL CONDITION INDEX	8	1	3	232	0	12
GLOBAL STRUCTURAL CONDITION INDEX	6	1	2	198	0	18
ESCI (0-100)	1,05	0,07	0,48	19,09	0,00	1,03
GSCI (0-100)	0,42	0,03	0,19	7,64	0,00	0,82
	DECK	**GIRDER**	**DIAPH.**	**P. CAP**	**PIER**	**ABOUT.**
LEVEL OF RELIABILITY	LC3	LC3	LC3	LC3	LC3	LC2
NUMBER OF DEFECTS	3 /6	2 /8	0 /5	3 /7	3 /7	0 /6
TOTAL SEVERITY FOUND	8 /17	3 /28	0 /15	9 /21	10 /20	0 /20
SEVERITY AVERAGE	2,67	1,50	0,00	3,00	3,33	0,00
ELEMENT STRUCTURAL CONDITION INDEX	48	3	0	52	116	0
GLOBAL STRUCTURAL CONDITION INDEX	55	4	0	71	176	0
ESCI (0-100)	6,27	0,22	0,00	4,28	10,74	0,00
GSCI (0-100)	3,76	0,13	0,00	2,74	6,87	0,00
	DECK	**GIRDER**	**DIAPH.**	**P. CAP**	**PIER**	**ABOUT.**
LEVEL OF RELIABILITY	LC3	LC3	LC3	LC3	LC3	LC3
NUMBER OF DEFECTS	3 /6	3 /8	3 /5	3 /7	0 /7	0 /6
TOTAL SEVERITY FOUND	8 /17	8 /28	7 /15	9 /21	0 /20	0 /20
SEVERITY AVERAGE	2,67	2,67	2,33	3,00	0,00	0,00
ELEMENT STRUCTURAL CONDITION INDEX	211	16	50	205	0	0
GLOBAL STRUCTURAL CONDITION INDEX	160	15	33	175	0	0
ESCI (0-100)	27,58	1,15	7,94	16,87	0,00	0,00
GSCI (0-100)	11,03	0,46	3,17	6,75	0,00	0,00
	DECK	**GIRDER**	**DIAPH.**	**P. CAP**	**PIER**	**ABOUT.**
LEVEL OF RELIABILITY	LC3	LC3	LC3	LC3	LC2	LC1
NUMBER OF DEFECTS	3 /6	4 /8	3 /5	3 /7	4 /7	0 /6
TOTAL SEVERITY FOUND	8 /17	13 /28	7 /15	9 /21	15 /20	0 /20
SEVERITY AVERAGE	2,67	3,25	2,33	3,00	3,75	0,00
ELEMENT STRUCTURAL CONDITION INDEX	14	130	28	168	77	0
GLOBAL STRUCTURAL CONDITION INDEX	17	201	30	215	146	0
ESCI (0-100)	1,83	9,32	4,44	13,83	7,13	0,00
GSCI (0-100)	1,19	6,06	2,89	8,30	5,70	0,00

	DECK	GIRDER	DIAPH.	P. CAP	PIER	ABOUT.
LEVEL OF RELIABILITY	LC2	LC2	LC2	LC2	LC2	LC1
NUMBER OF DEFECTS	3 /6	4 /8	3 /5	4 /7	4 /7	0 /6
TOTAL SEVERITY FOUND	8 /17	13 /28	7 /15	14 /21	15 /20	0 /20
SEVERITY AVERAGE	2,67	3,25	2,33	3,50	3,75	0,00
ELEMENT STRUCTURAL CONDITION INDEX	40	130	28	455	30	0
GLOBAL STRUCTURAL CONDITION INDEX	71	288	43	778	57	0
ESCI (0-100)	5,23	9,32	4,44	37,45	2,78	0,00
GSCI (0-100)	4,88	8,70	4,15	29,96	2,22	0,00
	DECK	GIRDER	DIAPH.	P. CAP	PIER	ABOUT.
LEVEL OF RELIABILITY	LC3	LC3	LC3	LC3	LC3	LC2
NUMBER OF DEFECTS	3 /6	0 /8	0 /5	1 /7	0 /7	1 /6
TOTAL SEVERITY FOUND	8 /17	0 /28	0 /15	3 /21	0 /20	1 /20
SEVERITY AVERAGE	2,67	0,00	0,00	3,00	0,00	1,00
ELEMENT STRUCTURAL CONDITION INDEX	8	0	0	90	0	18
GLOBAL STRUCTURAL CONDITION INDEX	9	0	0	107	0	27
ESCI (0-100)	1,05	0,00	0,00	7,41	0,00	1,54
GSCI (0-100)	0,61	0,00	0,00	4,10	0,00	1,23
	DECK	GIRDER	DIAPH.	P. CAP	PIER	ABOUT.
LEVEL OF RELIABILITY	LC2	LC2	LC2	LC2	LC2	LC2
NUMBER OF DEFECTS	2 /6	2 /8	1 /5	2 /7	1 /7	1 /6
TOTAL SEVERITY FOUND	3 /17	4 /28	1 /15	10 /21	5 /20	1 /20
SEVERITY AVERAGE	1,50	2,00	1,00	5,00	5,00	1,00
ELEMENT STRUCTURAL CONDITION INDEX	22	39	9	230	150	18
GLOBAL STRUCTURAL CONDITION INDEX	33	74	12	492	356	27
ESCI (0-100)	2,88	2,80	1,43	18,93	13,89	1,54
GSCI (0-100)	2,30	2,24	1,14	18,93	13,89	1,23
	DECK	GIRDER	DIAPH.	P. CAP	PIER	ABOUT.
LEVEL OF RELIABILITY	LC3	LC3	LC3	LC3	LC3	LC2
NUMBER OF DEFECTS	2 /6	2 /8	1 /5	2 /7	1 /7	1 /6
TOTAL SEVERITY FOUND	3 /17	4 /28	1 /15	10 /21	5 /20	1 /20
SEVERITY AVERAGE	1,50	2,00	1,00	5,00	5,00	1,00
ELEMENT STRUCTURAL CONDITION INDEX	22	39	9	230	150	18
GLOBAL STRUCTURAL CONDITION INDEX	22	49	8	229	166	27
ESCI (0-100)	2,88	2,80	1,43	18,93	13,89	1,54
GSCI (0-100)	1,53	1,49	0,76	8,83	6,48	1,23

3.6 CONCLUSIONS

Safety of existing constructions is a very relevant problem, especially in areas exposed to seismic risk. The present procedure specific refers to the structural characterization in view of quantitative assessment of real performances. The topic is well documented as the analysis of single structures, but some issues related to management of road networks are not well developed from a structural and seismic point of view. A procedure able to provide quantitative comparative data for networks at regional scale has been presented.

Outcomes of the work can be used for structural management of the network, as well as a support to the design of inspections and tests for structural characterization of existing bridges.

The first information required for the objective classification of the damaged state of existing bridges is the reliability of information. It requires accurate inspections, concerning the whole structure and in particular of each structural class. This condition cannot be easily obtained for the inherent characteristics of the infrastructures which, outside the urban areas, are located on complex geological conditions. In this framework and in view of the use of the results for the assessment of the seismic performance of the structure, the reliability of information is defined through the above mentioned intervals/ranges, in agreement with those defined by the codes for the Level of Knowledge. The Investigated Elements Index and the Global Maintenance Index of the investigated class of elements provides an information about their health state according to the results of in-situ visual inspections.

A careful analysis of conservation state of and degradation of the structure, allows two goals. The first is to ensure that they are not workings special phenomena that may compromise the structural safety of the work, and for which the seismic capacity of the structure can be greatly cut down, the second is related to the observation of the conservation status structure, and quantification of degradation/corrosion due to natural structure working.

This phenomena, if located in particulars elements, or in significant progress, can significantly affect the capacity estimation, and generate crisis for acceleration values lower than those supported by the structure in optimal conditions.

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Chapter 4

SOIL STRUCTURE INTERACTION

4.1 INTRODUCTION

When subjected to dynamic loads, foundations oscillate in a way depending on the nature and deformability of the supporting ground, on the geometry and inertia of the foundation and superstructure, and on the nature of the dynamic excitation. Excitation may be in the form of support motion due to wave saving thrush the ground during an earthquake, or it may result from the dynamic forces imposed directly or indirectly on the foundation. The development of analytical methods for soil structure interaction evaluation has principally been driven by the demands of offshore oil production activities and partially embedded nuclear power. For offshore applications, where cyclic wave loading applies lateral loads to pile-supported marine structures, a limited series of field and model tests has established the empirically-based and widely accepted “p-y” method of laterally loaded pile analysis.

Not only marine structures are subjected on dynamic loads, but excitation may has the wave form of an earthquake, so also foundations are subjected on cyclic loading conditions. So this static loading analysis method has been modified and extended to cyclic loading conditions, and is also routinely applied to dynamic or earthquake loading cases [Meymand et al. (1998)].

Generally for civil structures, design of foundation is conceived as rigid element embedded in soil. For infrastructural engineering, geotechnical consideration about construction site could be fundamental in structural design. The foundation deformability effect, not important for structure subjected to gravity loads only, may be relevant for structure subjected to seismic loading.

For bridge structures, considering particular shape, subjected to high lateral loads, the error of neglecting soil-structure interaction in mechanical performances, may not be totally negligible. Effects of soil deformability can have relevant effects on superstructure ductility evaluation. If considering two SDOF systems of equal mass and stiffness characteristics, one rigid to the base, and other with a deformable foundation, the ductility response could be significantly different and rigid system ductility overestimated (§ 5.2). However, if the stiffness of foundations can lead to an overestimation of the ductility of the system, at the same time an underestimation of the seismic action from the response spectrum can be achieved. The change in the natural period can decrease the expected value of the seismic action. This is the reason why the Code requires, for ordinary structures, a fully restrained foundations.

At the Italian level, codes suggest that generally is possible to consider foundation rigid at the base. Soil structure must be consider in the seismic analysis in case of three conditions occur simultaneously [NTC (2008)]:

- Class structure III or IV;
- Class soil D or worse;
- High seismicity, $a_g \geq 0{,}15g$.

4.2 STATE OF THE ART

There are many approaches to approximate soil-structure interactions, and static, cyclic, and dynamic loading are all considered in the problem. The approaches employed vary widely in complexity and applicability. Empirical methods approximate the soil and foundation as springs, and there are useful for approximate solution for soil-structure interaction in simples configurations. Extrapolation of empirical solutions for complex foundation systems and piles in different layers of soil type are affected of many complications. A brief review of conventional methods is showed.

Beam-on-elastic-foundation solutions in the form of the governing fourth-order differential equation is originally presented by Hetenyi [Hetenyi (1946)]. As is the case with the elastic continuum method, analytical solutions are not available for arbitrary distributions of soil or pile stiffness. This method has been applied to static lateral pile loading problems, and is therefore used for the determination of pile head stiffness terms.

Matlock and Reese (1960) presented a generalized iterative solution method for rigid and flexible laterally loaded piles embedded in soils with two forms of varying modulus with depth. Broms (1964a, b) describes a method for analyzing lateral pile response in cohesive and cohesionless soils. His method for computing ground surface deflections of rigid and flexible fixed and free head piles was based on a modulus of subgrade reaction using values suggested by Terzaghi (1955). Jamilokowski and Garassino (1977) provided a state-of-the-art discussion on soil modulus and ultimate soil resistance for laterally loaded piles. Randolph and Houlsby (1984) used classical plasticity theory to derive lower and upper bound values of the limiting pressure on an undrained laterally loaded pile.

For the beams Winkler's foundation that each layer of soil responds independently to adjacent layers, a beam and discrete spring system may be adopted to model pile lateral loading. In this method, the soil-pile contact is discredized to a number of points where combinations of springs and dashpots represent the soil-pile stiffness and damping at each particular layer. These soil-pile springs may be linear elastic or nonlinear; p-y curves typically used to model nonlinear soil-pile stiffness have been empirically derived from field tests, and have the advantage of implicitly including pile installation effects on the surrounding soil, unlike other methods. A singular disadvantage of a beam-on-Winkler-foundation model is the two-dimensional simplification of the soil-pile contact, which ignores the radial and three dimensional components of interaction.

Kagawa and Kraft (1980) developed a nonlinear dynamic Winkler model using the equivalent linear method, with input excitation applied as lateral ground displacements at the end of the near-field soil elements. The pile was modeled by a continuous beam with near field soil elements comprised of parallel springs and dashpots, and with superstructure elements that generated the inertial component of response.

The nonlinear soil model was formulated as an effective stress model, and cyclic degradation of soil resistance was governed by pore pressure generation in 1981 by Kagawa and Kraft [Kagawa and Kraft (1981)].

Hybrid near field/far field soil-pile interaction models for dynamic loading is developed by Nogami. Solutions for single pile and pile group axial and lateral response are formulated in both the time and frequency domains, incorporating nonlinear soil-pile response, degradation, gapping, slip, radiation damping, and loading rate effects [Nogami et al., (1991); Nogami et al., (1992)]. In Nogami (1985) and Nogami and Konagi (1988), the transfer matrix approach was described that was used to solve the equations of motion for a pile subject to soil-pile interaction forces, functions of the near field and far field soil element properties.

A so-called macroscopic model based on the Bouc-Wen model of visco-plasticity, is introduced by Makris and Badoni, which used distributed nonlinear springs to approximate the soil-pile reaction [Makris and Badoni (1995a)]. Limits of soil resistance were based on the work of Broms (1964), Randolph and Houlsby (1984), and Matlock (1970). Radiation damping was provided by a frequency dependent viscous dashpot that attenuated at large pile deflections. The model accommodated pile head loading, and required that two parameters be fit by experimental data. Makris (1994) has also presented an analytical solution for pile kinematic response due to the passage of Rayleigh waves, applicable to near field earthquake response.

An elastic continuum analytical method is based on closed form solution for the application of point loads to a semi-infinite mass is proposed by Mindlin's [Mindlin's

(1936)]. The accuracy of these solutions is directly related to the evaluation of the Young's modulus and the other elastic parameters of the soil. This approach is limited in the sense that nonlinear soil-pile behavior is difficult to incorporate, and it is more appropriately applied for small strain, steady state vibration problems. In addition, layered soil profiles cannot be accommodated, and only solutions for constant, linearly increasing, and parabolically increasing soil modulus with depth have been derived. Poulos has been a major progenitor of elastic solutions for soil and rock mechanics, and has worked extensively on all aspects of pile foundation response to axial and lateral loads. In Poulos (1971a, b) he first published elastic continuum solutions for laterally loaded single piles and groups under static loading. Poulos and Davis (1980) presented a comprehensive set of analysis and design methods for pile foundations based on elastic continuum theory. Poulos (1982) described a procedure for degradation of soil-pile resistance under cyclic lateral loading and compared it to several case studies.

Gazetas and Dobry (1984) derived a method for substructuring the soil structure interaction problem into kinematic and inertial components from a parametric finite element study based on the work of Blaney et al. (1976). For the inertial interaction component, they described the pile head dynamic stiffness by a complex valued impedance function of the form

$$\tilde{K} = \overline{K} + i\omega C \tag{4.1}$$

where $\overline{K}$ is the soil-pile dynamic stiffness, ω is the excitation frequency, C is the coefficient of equivalent viscous damping. Constant, linearly varying, and parabolically varying soil modulus with depth cases were studied for single piles, surface and embedded foundations and caissons foundations subjected to vertically propagating shear waves. This method, applied in the present study for soil structure interaction evaluation, is detailed presented afterwards in this chapter. The finite element method potentially provides the most powerful means for conducting soil-structure interaction analyses, but is has not yet been fully realized as a practical tool.

Yegian and Wright (1973) implemented a finite element analysis with a radial soil-pile interface element that described the nonlinear lateral pile response of single piles and pairs of piles to static loading. Wong et al. (1989) modeled soil-drilled shaft interaction with a specially developed 3D thin layer interface element. Bhowmik and Long (1991) devised 2D and 3D finite element models that used a bounding surface plasticity soil model and provided for soil-pile gapping.

4.3 SOIL STRUCTURE INTERACTION: STIFFNESS EVALUATIONS

For bridge structures, considering particular shape, subjected to high lateral loads, the error of neglecting soil-structure interaction effects in mechanical performances, may not be totally negligible. In this point of view, the method proposed by Gazetas (1991) must adequately reflect the following key characteristic of the foundation-soil system:

- The shape of the foundation-soil interface (circular, rectangular, arbitrary);
- The amount of embedded (surface, partially or fully embedded foundation);
- The nature of the soil profile (deep uniform or layered deposit, shallow stratum over bedrock);
- The mode of vibration and the frequency of excitation.

The steady state reponse of such systems to armonic external forces and moments can be computed whit well established methods of structural dynamics once the matrix of dynamic impedance foundations $\tilde{S}(\omega)$ has been determined for the frequency(ies) of interest.

For each particular harmonic excitation, the dynamic impedance is defined as the ratio between force (or moment) R and the resulting-steady state displecement (or rotation) U at the centroid of the base of the massless foundation.

So, for each impedance component, is possible to define:

$$\tilde{S}_x = \frac{R_x(t)}{U_x(t)} \tag{4.2}$$

In which $R_x = R_x e^{i\omega t}$ and $U_x = U_x e^{i\omega t}$ are the armonical orizontal force and displacement of the soil-foundation interface.

Similary, is possible to define the other orizontal component, S_x, vertical component S_z, and, for rotational motion, S_{rx}, rocking impedance for rotational motion about the long centroid axis (x) of the foundation basemat, S_{ry} rocking impedance for rotational motion about the long centroid axis (y), and S_t, torsional impedance for rotational oscillation about the vertica axis (z). Moreover, mainly in embedded foundation and piles, horizontal forces alon principal axes, induce rotational in addition ti transational oscillations;hence, two more cross coupling horizontalrocking impedances exist: $S_{x\text{-}ry}$ and $S_{y\text{-}rx}$. They are negligibly small in surface and shallow foundations, but their effects may become appreciable for greater depths of embedment owing to the moments about the base axes produced by horizontal soil reactions against the sidewalls. In piles, such cross-coupling impedances are important as the direct impedances [Gazetas, (1991)].

Because of the presence of the radiation and material damping in the system of all modes of vibration, R is generally out of fase whit U. It has become traditional to introduce complex notation and to express each impedance in the form:

$$\tilde{S} = \tilde{K} = \overline{K} + i\omega C$$

(4.3)

in which $\overline{K}$ and C are functions of the frequency of excitation ω.

The real component $\overline{K}$ reflect the stiffness and inertia of the supporting soil; its dependence on frequency is attributed solely to the influence that frequency exerts on inertia, since soil properties are pratically frequency independent.

The dynamic stiffness $\overline{K} = \overline{K}(\omega)$ can be estimated as a product of static stiffness K and the dynamic coefficient $k = k(\omega)$:

$$\overline{K} = K \cdot k(\omega) \tag{4.4}$$

The dashpot coefficient C, in the immaginary component, reflects the radiation and material damping generated in the system due to energy carried by waves spreading away from the foundation and energy dissipated in the soil by hysteretic action. The coefficient do not include the soil hysteretic damping, β. To incorporate this component one simply adds the corresponding material dahspot to the C value:

$$totalC = C + \frac{2\overline{K}}{\omega}\beta \tag{4.5}$$

For each mode of oscillation, model of soil-structure interaction can be represented by sistems of springs and dashpots whit characteristic moduli equal to $\overline{K}$ and C, respectively.

4.3.1 Shallow and embedded foundations

Expressions to estimate the moduli of springs and dashpots, are referred in the in the following Tables (Tab. 4.1, Tab.4.2) for surfaces and embedded foundation in homogeneous half-space.

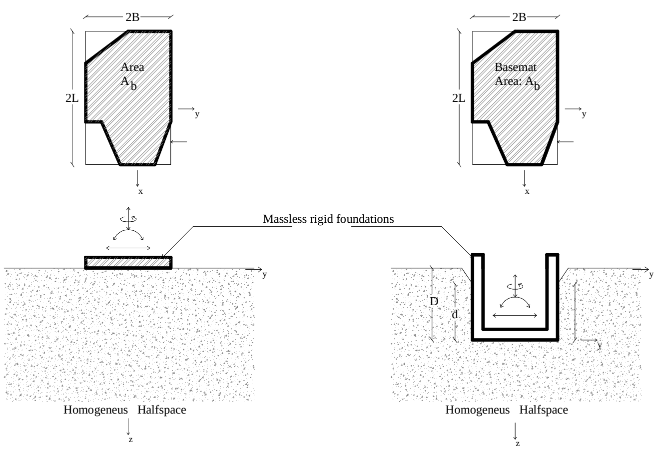


Fig. 4.1 a) Surface foundation of arbitrary shape, b) Embedded foundation of arbitrary shape.

Vibration mode (1)	Static stiffness, K (2)	Dynamic stiffness coefficient, k ($0 \le a_0 \le 2$) (3)	Radiation dashpot coefficient, C (4)
Vertical (z)	$K_z = [2GL/(1-\nu)](0.73 + 1.54\chi^{0.75})$ with $\chi = A_b/4L^2$	$k_z = k_z(L/B, \nu; a_0)$ is plotted in Fig. 2(a)	$C_z = (\rho V_{La} A_b) \cdot \tilde{c}_z$ where $\tilde{c}_z = \tilde{c}_z(L/B; a_0)$ is plotted in Fig. 2(c)
Horizontal (y) (lateral direction)	$K_y = [2GL/(2-\nu)](2 + 2.50\chi^{0.85})$	$k_y = k_y(L/B; a_0)$ is plotted in Fig. 2(b)	$C_y = (\rho V_s A_b) \cdot \tilde{c}_y$ where $\tilde{c}_y = \tilde{c}_y(L/B; a_0)$ is plotted in Fig. 2(d)
Horizontal (x) (longitudinal direction)	$K_x = K_y - [0.2/(0.75-\nu)]GL[1-(B/L)]$	$k_x \simeq 1$	$C_x = \rho V_s A_b$
Rocking (rx) (about the longitudinal, x-axis)	$K_{rx} = [G/(1-\nu)] I_{bx}^{0.75} (L/B)^{0.25} [2.4 + 0.5(B/L)]$	$k_{rx} \simeq 1 - 0.20 a_0$	$C_{rx} = (\rho V_{La} I_{bx}) \cdot \tilde{c}_{rx}$ where $\tilde{c}_{rx} = \tilde{c}_{rx}(L/B; a_0)$ is plotted in Fig. 2(e)
Rocking (ry) (about the lateral, y-axis)	$K_{ry} = [3G/(1-\nu)] I_{by}^{0.75} (L/B)^{0.15}$	$\nu < 0.40$: $k_{ry} \simeq 1 - 0.26 a_0$ $\nu \simeq 0.50$: $k_{ry} \simeq 1 - 0.26 a_0 (L/B)^{0.30}$	$C_{ry} = (\rho V_{La} I_{by}) \cdot \tilde{c}_{ry}$ where $\tilde{c}_{ry} = \tilde{c}_{ry}(L/B; a_0)$ is plotted in Fig. 2(f)
Torsion (t)	$K_t = 3.5 G I_{bz}^{0.75} (B/L)^{0.4} (I_{bz}/B^4)^{0.2}$	$k_t = 1 - 0.14 a_0$	$C_t = (\rho V_s I_{bz}) \cdot \tilde{c}_t$ where $\tilde{c}_t = \tilde{c}_t(L/B; a_0)$ is plotted in Fig. 2(g)

Tab. 4.1 Dynamic stiffness and dashpot coefficients for arbitrarily shaped foundations on surface of homogeneous half-space [Gazetas (1991)]

Vibration mode (1)	Static stiffness, K_{emb} (2)	Dynamic stiffness coefficient, $k_{emb}(\omega)$ (3)	Radiation dashpot coefficient, $C_{emb}(\omega)$ (4)
Vertical (z)	$K_{z,emb} = K_z[1 + (1/21)(D/B)(1 + 1.3\chi)][1 + 0.2(A_w/A_b)^{2/3}]$ where $K_z \equiv K_{z,surface}$ is obtained from Table 1. A_w = actual sidewall-soil contact area; for constant effective-contact height, d, along the perimeter: $A_w = (d) \times$ (perimeter); $\chi = A_b/4L^2$	($\nu \le 0.40$): fully embedded: $k_{z,emb} \simeq k_z[1 - 0.09(D/B)^{3/4} a_0^2]$ in a trench: $k_{z,tre} \simeq k_z[1 + 0.09(D/B)^{3/4} a_0^2]$ ($\nu \simeq 0.48$): fully embedded with $L/B = 1 - 2$: $k_{z,emb} \simeq k_z[1 - 0.09(D/B)^{3/4} a_0^2]$ fully embedded with $L/B > 3$: $k_{z,emb} \simeq k_z[1 - 0.35(D/B)^{1/2} a_0^{2.5}]$ in a trench: $k_{z,tre} \simeq k_z$, where $k_z = k_{z,surf}$ from Table 1. All ν, partially embedded: interpolate	$C_{z,emb} = C_z + \rho V_s A_w$ where $C_z = C_{z,surface}$ is obtained from Table 1 and the associated chart of Fig. 2.
Horizontal (y) and (x)	$K_{y,emb} = K_y[1 + 0.15(D/B)^{0.5}][1 + 0.52[(h/B)(A_w/L^2)]^{0.4}]$ $K_{x,emb} = K_x \cdot (K_{y,emb}/K_y)$ where $K_y = K_{y,surface}$ and $K_x = K_{x,surface}$ are obtained from Table 1.	$k_{y,emb}$ and $k_{x,emb}$ can be estimated in terms of L/B, D/B, and d/B for each value of a_0 from the plots in Fig. 3	$C_{y,emb} = C_y + 4\rho V_s Bd + 4\rho V_{La} Ld$ $C_{x,emb} = C_x + 4\rho V_{La} Bd + 4\rho V_s Ld$ where $C_y = C_{y,surface}$ and $C_x = C_{x,surface}$ are obtained from Table 1 and the associated chart of Fig. 2.
Rocking (rx) and (ry)	$K_{rx,emb} = K_{rx}\{1 + 1.26(d/B)[1 + (d/B)(d/D)^{-0.2}(B/L)^{0.5}]\}$ $K_{ry,emb} = K_{ry}\{1 + 0.92(d/L)^{0.6}[1.5 + (d/L)^{1.9}(d/L)^{-0.6}]\}$ where $K_{rx} \equiv K_{rx,surface}$ and $K_{ry} = K_{ry,surface}$ are obtained from Table 1.	$k_{rx,emb} \simeq k_{rx}$ $k_{ry,emb} \simeq k_{ry}$ The surface-foundation k_{rx} and k_{ry} are obtained from Table 1.	$C_{rx,emb} = C_{rx} + \rho I_{bx}(d/B)\{V_{La}(d^2/B^2) + 3V_s + V_s(B/L)[1 + (d^2/B^2)]\} \cdot \eta_{rx}$ where $\eta_{rx} = 0.25 + 0.65\sqrt{a_0}$ $(d/D)^{-a_0/2}(D/B)^{-1/4}$ $C_{ry,emb}$ is similarly evaluated from C_{ry} after replacing x by y, and interchanging B with L in the foregoing two expressions. In both cases $a_0 = \omega B/V_s$.
Swaying-rocking (x-ry)(y-rx)	$K_{x\text{-}ry,emb} = (1/3) d K_{x,emb}$ $K_{y\text{-}rx,emb} \cong (1/3) d K_{y,emb}$	$k_{x\text{-}ry,emb} = k_{y\text{-}rx,emb} \simeq 1$	$C_{x\text{-}ry,emb} = (1/3) d C_{x,emb}$ $C_{y\text{-}rx,emb} = (1/3) d C_{y,emb}$
Torsion (t)	$K_{t,emb} = K_t \cdot \Gamma_w \cdot \Gamma_{tre}$ where $K_t = K_{t,surface}$ is obtained from Table 1. $\Gamma_w = 1 + 0.4(D/d)^{0.5}(j_t/j_r)(B/D)^{0.6}$ $\Gamma_{tre} = 1 + 0.5(D/B)^{0.1}(B^4/I_{bz})^{0.13}$ $j_t = (4/3)d(B^3 + L^3) + 4BLd(L + B)$ $j_r = (4/3)BL(B^2 + L^2)$	$k_{t,emb} \cong k_{t,surface}$	$C_{t,emb} = C_t + 4\rho d[(1/3)V_{La}(L^3 + B^3) + V_s BL(L + B)] \cdot \eta_t$, where $C_t \equiv C_{t,surface}$ is obtained from Table 1 and Fig. 2 $\eta_t \equiv (d/D)^{-0.5} \cdot a_0^2/[a_0^2 + (1/2)(L/B)^{-1.5}]$

Tab. 4.2 Dynamic stiffness and dashpot coefficients for arbitrarily shaped foundations on surface of homogeneous half-space [Gazetas (1991)]

Values of coefficients c_z, c_y, c_{rx}, c_{ry}, c_t, can be found in Fig.4.2 respectively in diagrams 2c, 2d, 2e, 2f and 2g.

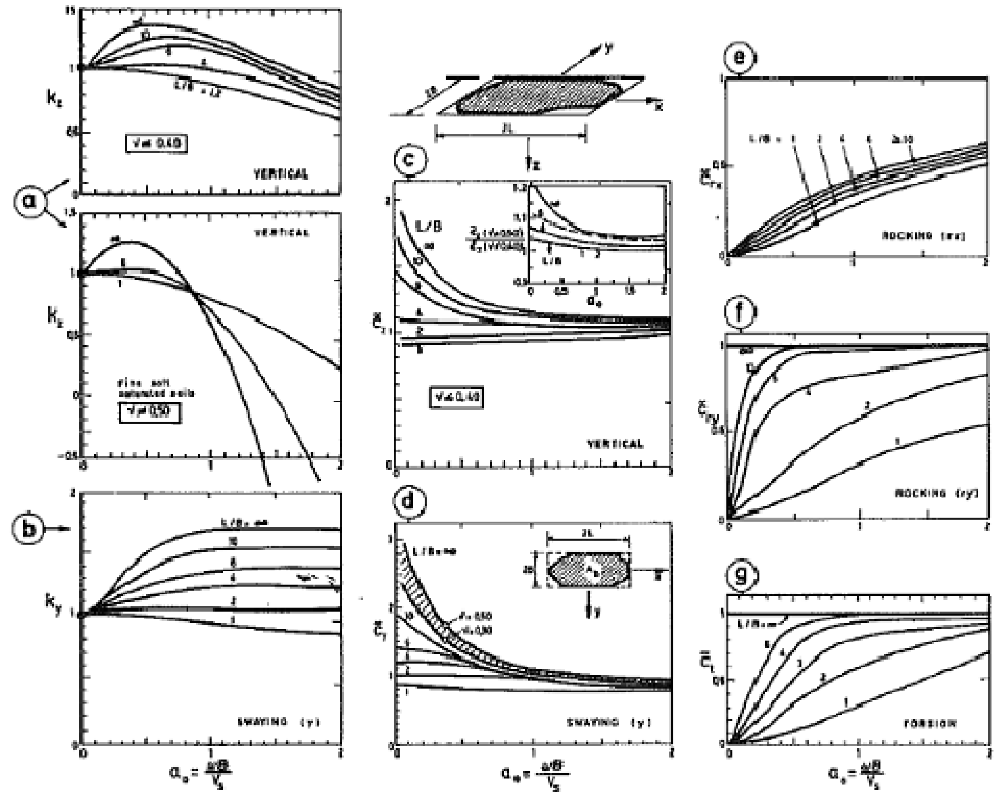


Fig. 4.2 Dimensionless graphs for determinig dynamic stiffness and damping coefficients of surface foundation s

To compute the impedence of the modes of vibration, all he needs is the values of the following:

- L; B half width and half-length of the circumscribed rectangle;
- d; D effective soil-structure contact, structure height; A_b contact foundation base area; A_w sidewall contact area;
- I_{bx}, I_{by}, I_{bz}, area moments of inertia about the x, y, and z axes of the actual soil foundation contact surface;
- G, ν the shear modulus and Poisson's ratio;
- V_s, V_{La} shear wave velocity and Lymer's analog wave velocity whit:

$$V_{La} = \frac{3{,}4}{\pi(1-\upsilon)} V_s \tag{4.6}$$

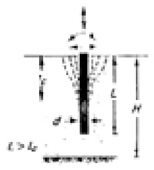
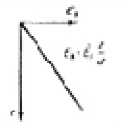
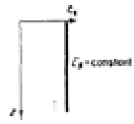
- ω circular frequency (radians/second) of the applied force (one of the frequencies dominant in case of the seismic excitation);
- $a_0 = \frac{\omega B}{V_s}$ dimensionless frequency factor (4.7)
- $\chi = \frac{A_b}{4L^2}$ (4.8)

4.3.2 Foundations on piles

Piles impedance express the harmonic force amplitude which must be applied at the top of the pile, to produce unitary amplitude harmonic motion. The expressions are valid for flexible piles whose length exceeds active length [Associazione Geotecnica Italiana, (2005)]. The response of laterally loaaded is indipendent by length. Only the uppermost part of the pile, of length l_c has apprecciable displacement [Gazetas, (1991)].

It is along this "active" length l_c that the imposed load is transmitted to the supporting soil l_c is typically of the order of 5 to 10 pile diameters, and for a given soil profile l_c is a function of the pile with respect to the soil.

For three characteristic soil profiles, Tab. 4.3 presents simple algebraic expressions for estimating l_c of a circular solid pile with diameter d and Young's modulus E_p. For each profile, the only soil parameter that affects l_c is the reference Young's modulus E_s.

	Linear Increase of Soil Modulus with Depth*	Parabolic Increase of Soil Modulus with Depth*	Constant Soil Modulus at All Depths
	 $V_s = [E_s/2\rho(1+\nu)]^{1/2}$	 $V_s = [E_s/2\rho(1+\nu)]^{1/2}$	
"Active length"	$l_c \approx 2d(E_p/E_s)^{0.20}$	$l_c \approx 2d(E_p/E_d)^{0.22}$	$l_c \approx 2d(E_p/E_s)^{0.25}$
Natural shear frequency of deposit	$f_1 = 0.19V_{sH}/H$ where V_{sH} = the S-wave velocity at depth $z = H$ (bottom of stratum)	$f_1 = 0.223V_{sH}/H$ where V_{sH} = the S-wave velocity at depth $z = H$ (bottom of stratum)	$f_1 = 0.25V_s/H$
Static lateral (swaying) stiffness	$K_{HH} = 0.6dE_s(E_p/E_s)^{0.35}$	$K_{HH} = 0.8dE_d(E_p/E_d)^{0.28}$	$K_{HH} = dE_s(E_p/E_s)^{0.21}$
Lateral (swaying) stiffness coefficient	$k_{HH} \approx 1$	$k_{HH} \approx 1$	$k_{HH} \approx 1$
Lateral (swaying) coefficient $C_{HH} = 2K_{HH}D_{HH}/\omega$	$D_{HH} \approx 0.60\beta + 1.80fdV_s^{-1}$, for $f > f_1$; $D_{HH} \approx 0.60\beta$, for $f \leq f_1$	$D_{HH} \approx 0.70\beta + 1.20fd(E_p/E_d)^{0.08}V_s^{-1}$, for $f > f_1$; $D_{HH} \approx 0.70\beta$, for $f \leq f_1$	$D_{HH} \approx 0.80\beta + 1.10fd(E_p/E_s)^{0.17}V_s^{-1}$, for $f > f_1$; $D_{HH} \approx 0.80\beta$, for $f \leq f_1$
Static rocking stiffness	$K_{RR} = 0.15d^3E_s(E_p/E_s)^{0.80}$	$K_{RR} = 0.15d^3E_d(E_p/E_d)^{0.77}$	$K_{RR} = 0.15d^3E_s(E_p/E_s)^{0.75}$
Rocking stiffness coefficient	$k_{RR} \approx 1$	$k_{RR} \approx 1$	$k_{RR} \approx 1$
Rocking dashpot coefficient $C_{RR} = 2K_{RR}D_{RR}/\omega$	$D_{RR} \approx 0.20\beta + 0.40fdV_s^{-1}$, for $f > f_1$; $D_{RR} \approx 0.20\beta$, for $f \leq f_1$	$D_{RR} \approx 0.12\beta + 0.35fd(E_p/E_d)^{0.10}V_s^{-1}$, for $f > f_1$; $D_{RR} \approx 0.12\beta$, for $f \leq f_1$	$D_{RR} \approx 0.35\beta + 0.35fd(E_p/E_s)^{0.20}V_s^{-1}$, for $f > f_1$; $D_{RR} \approx 0.35\beta$, for $f \leq f_1$
Static swaying-rocking cross stiffness	$K_{HR} = K_{RH} \approx -0.17d^2E_s(E_p/E_s)^{0.60}$	$K_{HR} = K_{RH} \approx -0.24d^2E_d(E_p/E_d)^{0.53}$	$K_{HR} = K_{RH} \approx -0.22d^2E_s(E_p/E_s)^{0.50}$
Swaying-rocking cross stiffness coefficient	$k_{HR} = k_{RH} \approx 1$	$k_{HR} = k_{RH} \approx 1$	$k_{HR} = k_{RH} \approx 1$
Swaying-rocking dashpot coefficient $C_{HR} = 2K_{HR}D_{HR}/\omega$	$D_{HR} \approx 0.30\beta + fdV_s^{-1}$, for $f > f_1$; $D_{HR} \approx 0.30\beta$, for $f \leq f_1$	$D_{HR} \approx 0.60\beta + 0.70fd(E_p/E_d)^{0.05}V_s^{-1}$, for $f > f_1$; $D_{HR} \approx 0.35\beta$, for $f \leq f_1$	$D_{HR} \approx 0.80\beta + 0.85fd(E_p/E_s)^{0.18}V_s^{-1}$, for $f > f_1$; $D_{HR} \approx 0.50\beta$, for $f \leq f_1$

Tab. 4.3 Dynamic stiffness and damping coefficients for flexible piles (l>lc) [Gazetas (1991)]

For the three lateral impedances K_{HH}, K_{MM}, K_{HM}, defined in Fig.4.3, Tab.4.3 presents formulas, which, are valid only for piles with length $l > l_c$.Such piles are described as "flexible" piles in the literature. But note a that a good majority of real-life piles, even some with large diameters, would fall into this category. Among the exceptions are short piers and caissons.

From a theoretical point of view, most of the formulas in Table 4.3 are reasonably accurate, as they are basically curve fits to rigorous numerical results. The real difficulty, however, is to select the proper profile and modulus for the supporting soil. Even with a uniform top layer, the secant soil modulus will change with the magnitude of induced strains, which decreases with depth. Other nonlinear phenomena, such as development of a gap between pile and soil near the ground surface, further complicate the problem [Gazetas, (1991)].

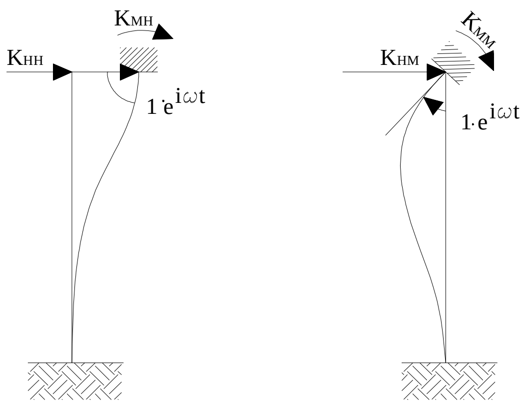


Fig. 4.3 Definition of pile head impedances [Gazetas, (1991)]

4.3.3 Caisson foundations

The caisson foundations were generally used as massive foundations for bridge piers and abutments, offshore structures and tower structure, when a strong lateral action was expected on the superstructure.

These elements are very high stiffness, which stuck in the ground behave as squat columns are capable of absorb horizontal actions transmitted by soil layer mobilized.

These structures are generally considered as intermediate between surfaces and embedded foundation and foundations on piles, and transfer the superstructure loads to the deeper soil layers, which are stiffer and resistant. For the on ground bridges, this type of foundation was frequently used when the bridge passes through unstable slopes and in seismic areas; for offshore bridges, it was adopted when strong actions of wind and water waves on the structure was expected. The avantage to use caissons foundations are:

- reaching soil layers with high stiffness
- limit the foundation plant dimensions
- stabilize the surface layer on the slopes
- protect piles from the landslides

- protect the batteries from the static and dynamic actions due to landslides.

The caissons were generally made by masonry or concrete, with circular, elliptic or rectangular sections; the in-plane dimensions of the caisson are generally larger in the transverse direction of the bridge alignment, in order to increase the stiffness in that direction. Generally, they have circular section whit $6<\Phi<20$ diameter meter.

The design of caissons foundation has many uncertainties, both for the complexity of the resistant mechanism (soil-structure interaction) both for the difficulty of quantifying of the actions related to the possible landslide. The stabilizing component due to the weight, prevails on the evaluation on the ultimate strength, related to the deep of the caisson.

Caissons foundation go subordinates to verifications to rotation sliding failure, and overall stability. The Eurocode and the Italian codes relative to the design and control of geotechnical structures, does not contain explicit indications for caisson foundation, but only for shallow and pile foundation.

The lateral and seismic response of bridge foundations was obtained with a number of methods of varying degrees of accuracy. However, few of them concerned the caissons. The methods of solution developed for (rigid) surface embedded foundation and for (flexible) piles have been frequently adapted to deal with the caisson problem. The evaluation of the horizontal bearing capacity of caisson foundation was usually based on an old formulation, centered on simplified hypotheses about geometry and soil/structure interaction.

Gazetas (1991) obtained semi−analytical expressions and charts for stiffness and damping of horizontally and rotationally loaded arbitrarily−shaped rigid foundations embedded in homogeneous soil. Gerolymos et al. (2006) focused on caissons, developing a Winkler model accounting the ultimate horizontal resistance of a cohesive soil.

For arbitraly shaped embedded foundation, circuscribed by a rectangle of width B and length $L(L>B)$ the impedence, whit respect of the center of the base mat, can be expressed in the form of equation 4.3:

$$\widetilde{K}_{emb} = K_{emb} k_{emb}(\omega) + i\omega C(\omega) \tag{4.8}$$

Gerolymos et al. (2006) focused on rigid caissons foundation, developing a generalized Winkler type method described by four spring and associated dashpot, and calibrated whit the elastodynamic solution proposed by Gazetas (1991) for foundations embedded in homogeneous soil (Tab.4.3). Dynamic Winkler four spring model incorporate distribuited traslational (lateral) and rotational (rocking) springs and dashpots, and concentrated shear traslational and rotational springs and dashpots at the base of the caissons.

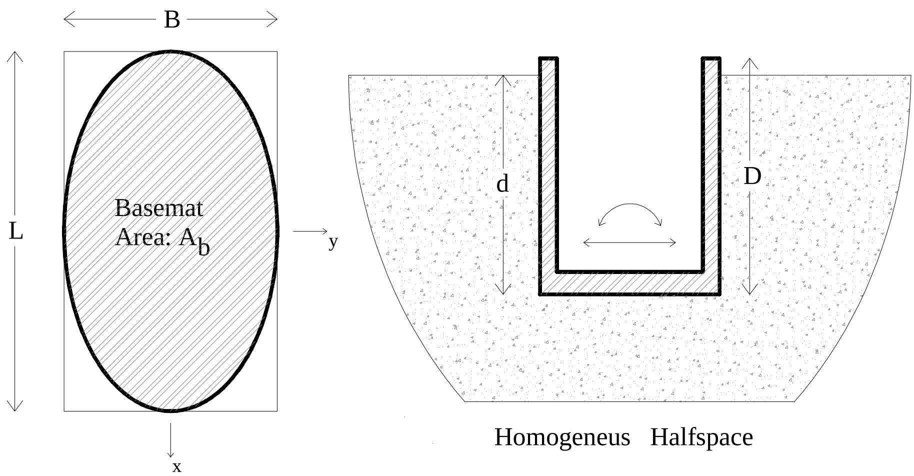


Fig. 4.4 Geometry of a rigid foundation arbitrary-shaped in plan embedded in a homogeneous elastic-half space

These four types of springs and dashpots are related to the resisting forces acting on the caisson shaft and base, as follow:

- k_x, c_x distributed lateral spring and dashpots, associated whit the horizontal soil reaction on the circumference of the caisson;
- k_θ, c_θ distributed lateral spring and dashpots, associated whit the moment produced by the vertical shear tractions on the circumference of the caisson;

- K_{HH} , C_{HH} resultant base shear translational spring and dashpot, associated whit the horizontal shearing force on the base of the caisson;
- K_{MM} , C_{MM} resultant base rotational spring and dashpot, associated whit the moment produced by normal pressures on the base of the caisson.

These coefficients are frequency depended and are function of both caisson geometry and soil stiffness.

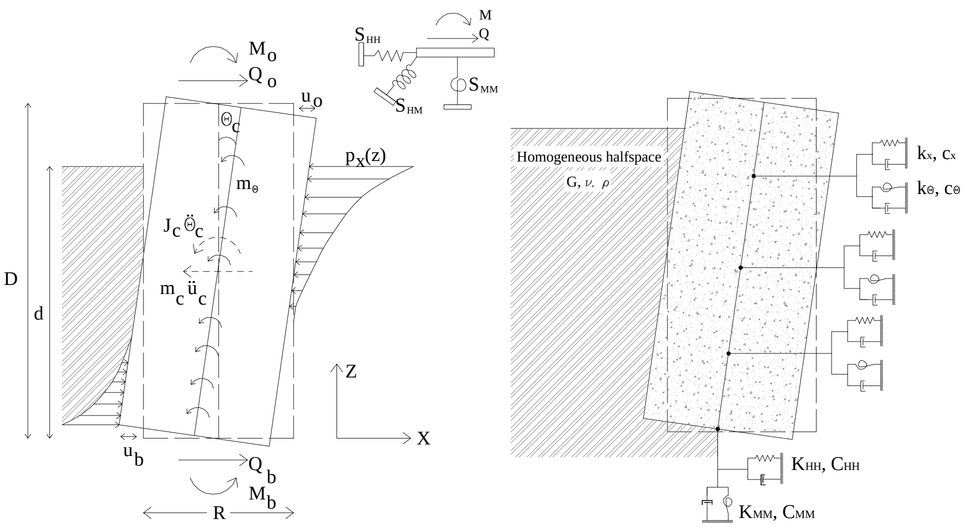


Fig. 4.5 The four types of springs and dashpots for the analysis of inertially and kinematically loaded caissons [Gerolymos, Gazetas, (2006)]. Elastic response of a caisson subjected to lateral dynamic loading M_0,Q_0 ai its top. Schematic definition of the global stiffness in lateral translation S_{HH}, rotation S_{MM}, and cross-coupling of the translation and rotation

Resultant base shear translational spring, rotational spring and on dashpots, for orthogonal or cylindrical caisson fully embedded in homogeneous half space, can be expressed as the following expressions (in the longitudinally –x axes):

$$\overline{K}_{HH} = \overline{K}_{x,emb} = K_x\left[1+0{,}15\left(\frac{D}{B}\right)^{0,5}\right]\left\{1+0{,}52\left[\left(\frac{d}{B}\right)\left(\frac{A_W}{L^2}\right)\right]^{0,4}\right\}k_x(\omega) \quad (4.10)$$

$$\overline{C}_{HH} = \overline{C}_{x,emb} = C_x + 4\rho V_{La} Bd + 4\rho VsLd \quad (4.11)$$

$$\overline{K}_{MM} = \overline{K}_{rx,emb} = K_{cr}\left\{1+0{,}26\left(\frac{d}{B}\right)\left[1+\left(\frac{d}{B}\right)\left(\frac{d}{D}\right)^{-0,2}\left(\frac{B}{L}\right)^{0,5}\right]\right\}k_{r\,x}(\omega) \quad (4.12)$$

$$\overline{C}_{MM} = \overline{C}_{r\,x,emb} = C_{r\,x} + I_{bx}\left(\frac{D}{B}\right)\begin{Bmatrix} V_{La}\left(\frac{D}{B}\right)^{2} + \\ +3V_s + V_s\left(\frac{B}{L}\right)\left[1+\left(\frac{D}{B}\right)^{2}\right] \end{Bmatrix}\eta_r \quad 4.13)$$

where:

K_x; K_{cr} spring translational and rotational stiffness at the caisson base on surface of homogeneous half-space;

C_x; C_{cr} spring translational and rotational dashpots at the caisson base on surface of homogeneous half-space;

$$\eta_r = 0{,}25 + 0{,}65\sqrt{a_0}\left(\frac{D}{B}\right)^{-\frac{1}{4}} \quad (4.14)$$

The coupled swaying-rocking complex impedance is approximated by:

$$\tilde{K}_{HM} \approx \frac{1}{3} d\tilde{K}_{HH} \quad (4.15)$$

The complex dynamic impedance matrix of the caisson referred to the base can be calculated as:

$$\tilde{K}_{emb} = \begin{bmatrix} \tilde{K}_{HH} & \tilde{K}_{HM} \\ \tilde{K}_{HM} & \tilde{K}_{MM} \end{bmatrix} \quad (4.16)$$

To obtain distributed lateral spring and dashpots formulae, is required to study the lateral response of caisson (rectangular or circular) embedded in homogeneous elastic soil over a deformable bedrock, and subjected to lateral dynamic excitation at is top: Q_0 and M_0 [Gerolymos (2006)]. The four spring model is used to for simulating the soil-structure interaction.

Dynamic equilibrium of the shear forces with respect to the base of the caisson gives:

- $$Q_0 - m\ddot{u}_c - P_x - Q_b = 0 \tag{4.17}$$

  where:

- $u_c = u_c(t)$ is the dispalcement of the caisson at the center of the gravity;
- $P_x = P_x(t)$ is the resultant sidewall horizontal resistance due to the lateral soil reaction;
- Q_b is the shear resistance at the base of the caissons;
- Dynamic moment equilibrium with respect to the base of the caisson gives:

$$M_0 + Q_0 D - J_c\ddot{\theta}_c - m\frac{D}{2}\ddot{u}_c - M_x - M_\theta - M_b = 0 \tag{4.18}$$

Where $M_x = M_x(z,t)$ and $M_\theta = M_\theta(z,t)$ are the sidewall resisteng moments arising from the orizontal soil reaction p_x ahd the vertical shear stresses τ_{xz} or τ_{rz}

Equation (4.17) and (4.18) can be wtitten as:

$$M_b\begin{Bmatrix}\ddot{u}_b\\ \ddot{\theta}_b\end{Bmatrix} + C_b\begin{Bmatrix}\dot{u}_b\\ \dot{\theta}_b\end{Bmatrix} + K_b\begin{Bmatrix}u_b\\ \theta_b\end{Bmatrix} = P_b \tag{4.19}$$

where:

$$M_b = \begin{bmatrix} m & m\frac{D}{2} \\ m\frac{D}{2} & J_c + m\frac{D^2}{2}\end{bmatrix};\ C_b = \begin{bmatrix} C_{hh} & C_{hr} \\ C_{hr} & C_{rr}\end{bmatrix}; K_b = \begin{bmatrix} K_{hh} & K_{hr} \\ K_{hr} & K_{rr}\end{bmatrix}$$

are respectively mass, damping and stiffness matrix, and P_b the external forces vector.

In the frequency domain, the complex stiffness matrix of the caisson is:

$$\tilde{K}_b = \overline{K}_b + i\omega C_b \tag{4.20}$$

and this matrix can be written as:

$$\tilde{K}_b = \begin{bmatrix} \tilde{K}_h + \tilde{k}_x D & \tilde{k}_x \dfrac{D^2}{2} \\ \tilde{k}_x \dfrac{D^2}{2} & \tilde{K}_r - \tilde{k}_\theta D + \dfrac{1}{3}\tilde{k}_x D^3 \end{bmatrix} \tag{4.21}$$

To obtain lateral spring functions $\tilde{k}_x, \tilde{k}_\theta$ the simple way is to equate the diagonal terms in the matrices (4.16) and (4.21) [Gerolymos (2006)]:

$$\tilde{k}_x = k_x + i\omega C_x = \frac{1}{D}\left(\tilde{K}_{HH} - \tilde{K}_x\right) \tag{4.22}$$

$$\tilde{k}_\theta = k_\theta + i\omega C_\theta = \frac{1}{D}\left(\tilde{K}_{MM} - \tilde{K}_r + \frac{1}{3}D^2\tilde{K}_x - \frac{1}{3}\tilde{K}_{HH}D^2\right) \tag{4.23}$$

4.4 STIFFNESS EVALUATION ANALYSIS

To determine foundations stiffness, is necessary to define geological features, identifying geological, geomorphological, hydrogeological and geotechnical characteristics. This characteristics can provide information on the geological model, the site stratigraphy, and, in the seismically point of view, potential risk expected. The geological study was developed in different steps (§ 2.2.2).

Downstream investigation, it was possible to identify for each viaduct, soil features, stratigraphy, and mechanical characteristics of the various layers. Examples are shown in following Tables, which were compiled for each viaduct. Identified geotechnical model, is possible to proceed with the evaluation of mechanical and seismic parameters for each layer. For foundation on piles and caissons, analysis for the determination of stiffness were carried out using mechanical parameters derived from the weighted average of the parameters for the different soil layers.

The soil nonlinearity, are considered by reducing the longitudinal and shear stiffness moduli, according to the Ramberg & Osgood (1943) theories.

A accurate procedure, in order to calculate the maximum shear strain, was proposed in some recently published works of conference proceedings [Bilotta et al.

(2007), Bilotta et al. (2007)]. The $\gamma_{\max}$ values were evaluated as the ratio between the maximum shear stresses $\tau_{\max}$ and the elastic shear moduli G.

The shear stress profile was computed through two different approaches .The first approach uses the following expression:

$$\tau_{\max}(z) = r_{\mathrm{d}}(z)\frac{a_{\max,s}}{g}\sigma_v(z) \tag{4.24}$$

as used in simplified approaches of the liquefaction potential [Santucci de Magistris (2005)]. Beyond the vertical stress σ_v, the other factors are the peak ground acceleration on surface $a_{max,s}$ and reductive coefficient r_d which takes into account the soil stiffness and can be computed for instance according to the formula [Iwasaki et al. (1978)]:

$$rd(z) = 1 - 0{,}015z \quad \text{(z in m)} \tag{4.25}$$

The peak ground acceleration at surface $a_{max,s}$ can be simply obtained from the peak ground acceleration at the bedrock, $a_{max,b}$, multiplied for the site amplification factor *S* [EN 1998-1 2003, NTC (2008)].

The shear strain $\tau_{\max}(z)$ is calculated through the horizontal equilibrium of a soil column, between the surface and the depth *z*, as:

$$\tau_{\max}(z) = \int_0^z \rho_s a_{\max}(z)dz \tag{4.26}$$

where ρ is the soil density. In the simplest application the profile of maximum acceleration can be assumed linear from $a_{\max b}$ at bedrock to $Sa_{\max b}$ at surface. Using the pseudo-static approaches, linear and linear-equivalent analyses were carried out, adopting a visco-elastic behavior for the investigated soil. In the linear analyses, the shear modulus G was assumed as the small strain modulus G_0; in the linear equivalent analyses G was referred to a degradation curve $G(\gamma)/G_0$, depending on shear strain level. In order to evaluate the maximum strain of the soil of the linear equivalent analyses, the Ramberg & Osgood (1943) model was considered, in which the shear strain was correlated with the maximum shear stress, using the expression:

$$\gamma_{max}(z) = \frac{\tau_{max}(z)}{G_0} + C\left[\frac{\tau_{max}(z)}{G_0}\right]^R \tag{4.27}$$

In the (4.28), C and R are parameters depending on the particular subsoil considered, which can be calculated by fitting the degradation curve $G(\gamma)/G_0$. The second addend of the equation (4.27) represented the increment of shear stain due to non-linearity of the soils.

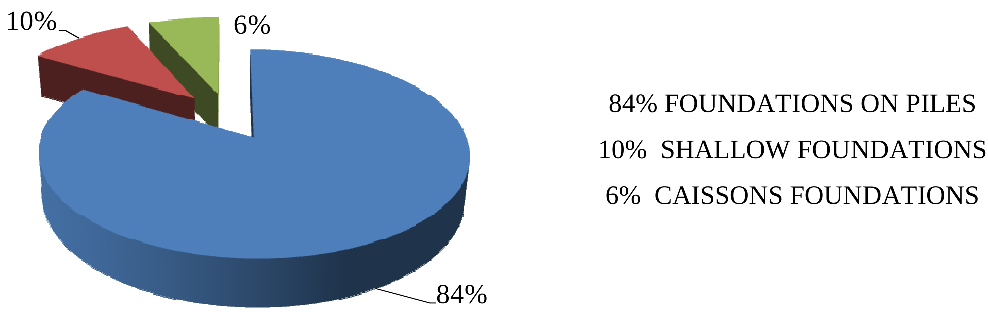


Fig. 4.6 Pier foundations distributions

The stock analyzed for seismic vulnerability, present a great heterogeneity with regard to the piers and abutments. In particular these are shallow foundations, on pile foundation and caissons Totally 244 foundations were analyzed. Generally foundations are homogeneous for each viaduct. However, in some case of very long viaduct, whose piers insist on different geological sites, there is the coexistence of different kind of foundations.

Shallow foundations consist of isolated footings of rectangular section. The foundations on piles, which make up the most part of the stock, consist of groups of piles in number, length and diameter variable depending on the characteristics of the ground, and on the horizontal and vertical loads transmitted from the superstructure.

For caisson foundations, only one of the viaducts have this type of foundation on all piers, probably due to the landslides presence. Other caissons are present below some piers in other viaducts.

In the following paragraphs, details of the analysis performed for each of the three foundations types is described. For exposition, database implemented after analysis of the documentation available and in situ tests analysis, are shown. Database are used for collection and management of mechanical and seismically sites characteristics resulting from analysis of existing documentation and on-site tests. Tables are reported in following paragraphs, as one example of existing viaduct existing for each type of foundations representative, of the work done. The same tables were implemented for the entire sample of viaducts. For each type of foundation were also performed parametric sensitivity analysis, that is evaluation of the influence of the foundations deformability on the period of SDOF system representing the piers, for shallow foundations and foundations on piles. For caissons, parametric analysis are made to the evaluation of the horizontal ultimate load capacity in order to the different caissons slenderness.

4.4.1. *Shallow foundations*

In the bridge’s stock analyzed, there are a family of two structures, and some other pier in other bridges, whit shallow foundations (Fig.4.7). By analysis of existing documentation and in situ tests, for each of the two, as well as for all other bridges, database of parameter necessary for seismically issue, and characteristic of site and mechanical soil model are filled (Tab. 4.4;Tab.4.5).

N	Layers	from-to (m)	h_i (m)	V_S (m/s)	ϕ' (°)	C_U (kPa)	E_{ed} (kPa)
1	Overburden	0m-2m	2	200	29	66	6958
2	Sand/silt	2m-5m	3	242,5	32	223	23657
3	Marl/sandst.	5m-16m	11	475			
4	Marl/sandst.	16m-30m	14	655			

Tab. 4.4 Geotechnical parameters of soil layers

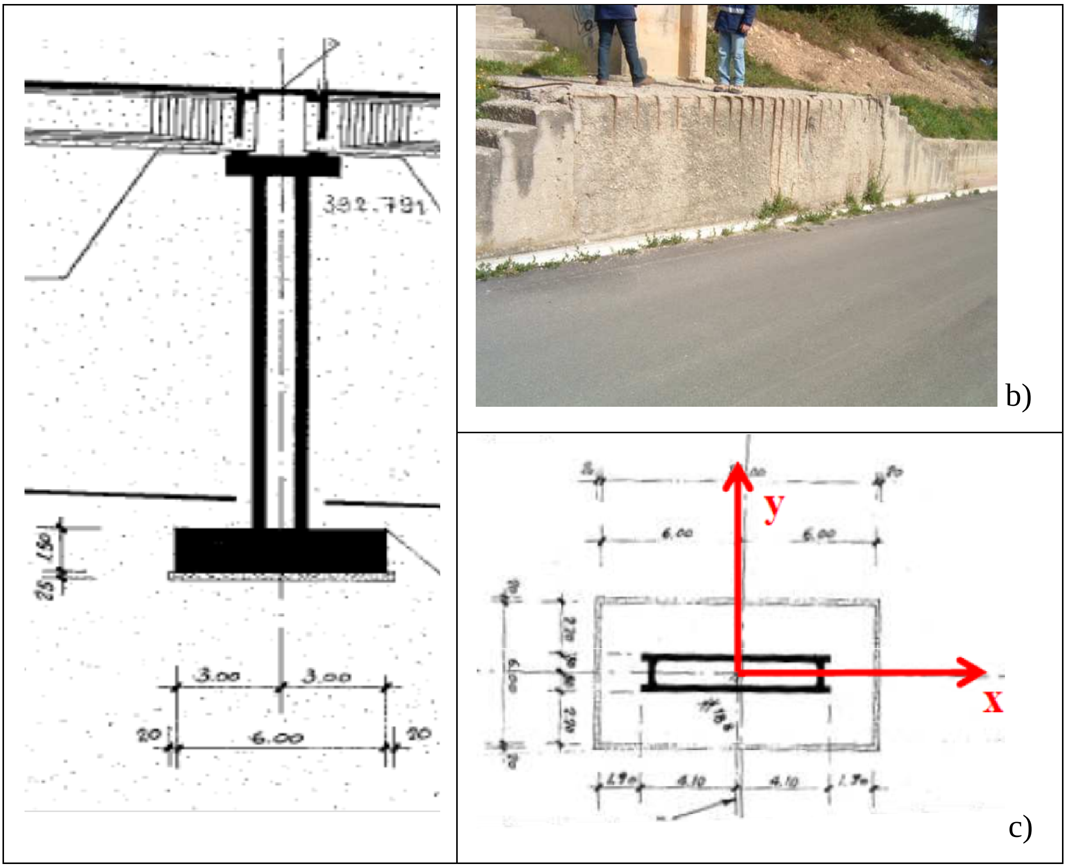


Fig. 4.7 Examples of viaduct whit shallow foundations. a) Longitudinal pier section; b) Picture of foundation; c) Foundation dimensions in plant view of the foundations. The dimension are in meter.

At Italian Level, Public Works Ministry, provides a seismic map, based on data from INGV [Group of work (2004)], depending of site geographic coordinates. In each node a different value for peak ground acceleration (a_g), local amplification factor (F_0) and control period (T_c), upper limit of the period of the constant spectral acceleration branch is defined.

Pier Aboutment	x (m)	z (m)	i (°)	Cat. Top.	Soil class
Ab. A	0,00	390,640	4,270	B	T1
Pier 1	33,20	388,161	3,770	B	T1
Pier 2	67,60	386,190	0,180	B	T1
Pier 3	102,00	387,947	4,530	B	T1
Pier 4	136,40	391,642	6,300	B	T1
Pier 5	170,80	395,548	3,680	B	T1
Pier 6	205,20	396,069	0,450	B	T1
Pier 7	239,60	396,089	0,270	B	T1
Pier 8	274,00	396,398	2,100	B	T1
Ab. B	307,20	398,564	3,730	B	T1

Tab. 4.5 Soils and topography categories for each pier

Due the limited spatial variability of hazard parameters and the limited viaduct extension, to Limit States of interest (§ 2.2.3) and considering:

- nominal life $V_N = 100$;
- use class IV;
- coefficient of use $C_U = 2{,}0$,

At Italian level current code indicate that the site amplification factor S for is obtained as:

$$S = S_S \cdot S_T \tag{4.28}$$

Where S_S is a coefficient depending on the soil class and S_T is dependent on topographic category.

Data obtained, and the amplified acceleration a_s on the surface with reference to the beginning of viaduct are shown in Tab 4.6.

As previously explained, the effects of non-linearity are considered through the reduction of longitudinal and shear stiffness moduli. In this case, for sand soil, the parameters C and R [Ramberg & Osgood (1943)] are:

R=2,63

C=800000

Degradation curve G/G_0 are shown in Tab. 4.7.

T_R (years)	START VIADUCT			END VIADUCT		
	a_g (g)	F_0	T_C*	a_g (g)	F_0	T_C*
30	0,054	2,343	0,301	0,054	2,341	0,301
50	0,071	2,436	0,308	0,071	2,436	0,308
72	0,085	2,483	0,313	0,085	2,484	0,313
101	0,100	2,500	0,32	0,099	2,501	0,32
140	0,116	2,511	0,323	0,115	2,511	0,323
201	0,135	2,518	0,329	0,135	2,518	0,329
475	0,194	2,477	0,342	0,194	2,478	0,343
975	0,258	2,451	0,35	0,256	2,452	0,35
2475	0,360	2,441	0,362	0,359	2,442	0,362

Tab. 4.6 Peak ground acceleration, local amplification factor, and upper limit of the period of the constant spectral acceleration for beginning and end of viaduct.

BEGINNING VIADUCT						
LIMIT STATE	T_R	a_g (g)	F_0	T_C*	S_s	S_t
SLO	120	0,108	2,506	0,321	1,20	1,00
SLD	201	0,135	2,518	0,329	1,20	1,00
SLV	1898	0,327	2,444	0,358	1,08	1,00
SLC	2475	0,360	2,441	0,362	1,05	1,00
LIMIT STATE	S	a_s (g)	τmax (kPa)	γmax	G (kPa)	G/G_0
SLO	1,20	0,130	8,575	2,04E-05	4,21E+05	0,98
SLD	1,20	0,162	10,686	2,56E-05	4,18E+05	0,98
SLV	1,08	0,354	23,351	5,94E-05	3,93E+05	0,92
SLC	1,05	0,378	24,935	6,40E-05	3,90E+05	0,91

Tab. 4.7 Seismically parameters and a degradation curve G/G_0

Stiffness results for [Gazetas (1991)] a shallow foundation of geometric characteristics shown in Tab. 4.8, are presented in Tab. 4.9, referred to the Prevention of Collapse Limit State.

The seismic vulnerability analysis are conducted for all viaducts considering soil-structure interaction effects. In addition, sensitivity analysis is conducted to evaluate the effects of the rate of deformation due to foundations deformability on the fundamental period of the simply oscillator simply representative of the on degree system (§ 5.2).

Shear modulus	**G**	390047	kPa
Poisson ratio	**ν**	0,3	
Half lenght of circumscribed rectangle	**L**	6	m
Half base of circumscribed rectangle	**B**	3	m
Height	**D**	3	m
Effective soil-striucture contact	**a**	0,5	
Soil density	**r**	1,9	kg/m^3
Shear wave velocity	**V_s**	475	m/s
Linear shear modulus	**G_0**	428688	kPa
Degradation curve	**G/G_0**	0,91	

Tab. 4.8 Geometrical foundations and mechanical soil characteristics

Vertical (z)	**K_z**	1,199,E+07	KN/m
Horizontal (y)	**$K_{y(long)}$**	1,630,E+07	KN/m
Horizontal (x)	**$K_{x(trasnv)}$**	1,539,E+07	KN/m
Rocking (rx)	**$K_{rx(long)}$**	1,866,E+08	KNm
Rocking (ry)	**$K_{ry(transv)}$**	4,927,E+08	KNm
Swaying (x-ry)	**K_{x-ry}**	7,696,E+06	KNm
Swaying (x-ry)	**K_{y-rx}**	8,150,E+06	KNm
Torsional	**K_t**	2,411,E+06	KNm

Tab. 4.9 Static foundation stiffness.

The Tab. 4.10 lists the values of the natural period of the system to one degree of freedom, representative for each pier of viaducts with shallow foundations.

For each pier, height, the mass at the top (§ 5.2.7), shear waves velocity observed in situ, rotational and translational values stiffness of the foundation, yield stress moment at the base of the pier, and parameters of stiffness and fundamental period of SDOF systems are shown. The latter in case of rigid and deformable foundation. Is possible to see how the rate of deformation due to the foundation becomes more important with decreasing height of the piers.

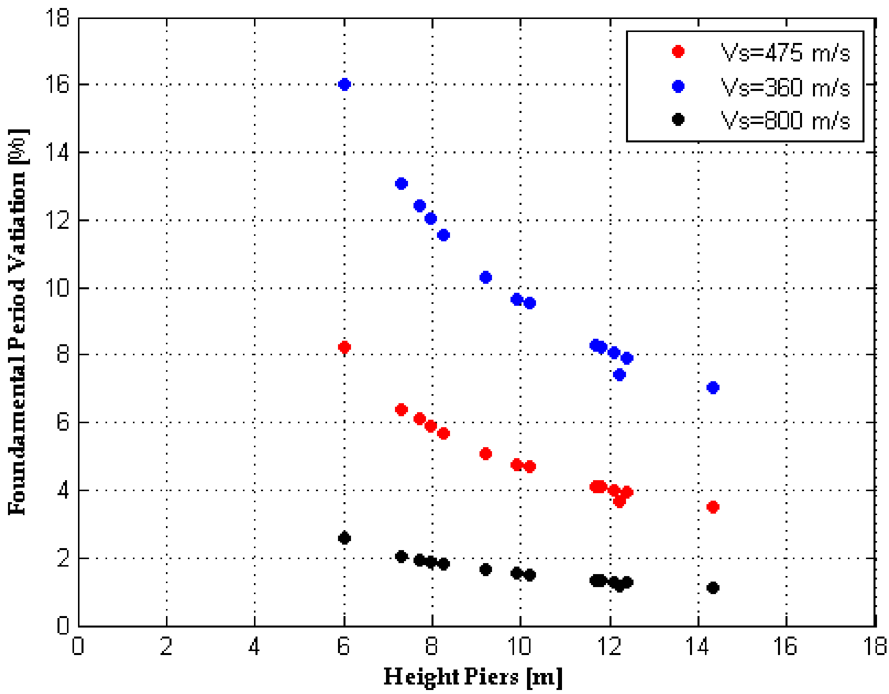


Fig. 4.8 Soil foundations effects for shallow foundation.

In fact, foundations designed to actions of tallest piers are repeated for all piers, as usual in the design practice, with the same geometry and reinforcement details. For piers lowest, the contribution of the foundation deformability, becomes more comparable to the flexural deformation of the pier, and so less negligible in view of the total deformation evaluation.

Analyses were conducted for the piers, as a function of soil class, maintaining a speed of shear waves comparable with the condition of shallow foundations.

										WHIT SSI EFFECTS				WITHOUT SSI EFFECT					
Npil	m	Vs	Hpil	B	H	Hplinto	Ky	Krx	My	Fy	dy	K	T	dhy	dry	dy	K	T	ΔT
		(m/s)	(m)	(m)	(m)	(m)	(kNm)	(kNm)	(kNm)	(kN)	(m)	(kN/m)	(s)	(m)	(m)	(m)	(kN/m)	(s)	%
1	722	450	6	6	12	1,5	1,54E+07	4,93E+08	58259	6654	0,0085	782823,53	0,191	6,31E-04	7,10E-04	7,16E-03	9,29E+05	0,175	-8,22
3	735	450	9,2	6	12	1,5	1,54E+07	4,93E+08	59849	5019	0,0155	323806,45	0,299	4,23E-04	1,12E-03	1,40E-02	3,60E+05	0,284	-5,10
4	746	450	11,7	6	12	1,5	1,54E+07	4,93E+08	60997	4236	0,0221	191674,21	0,392	3,39E-04	1,45E-03	2,03E-02	2,09E+05	0,376	-4,13
5	749	450	12,4	6	12	1,5	1,54E+07	4,93E+08	61314	4063	0,0242	167892,56	0,420	3,21E-04	1,54E-03	2,23E-02	1,82E+05	0,403	-3,93
6	748	450	12,1	6	12	1,5	1,54E+07	4,93E+08	61178	4135	0,0233	177467,81	0,408	3,28E-04	1,50E-03	2,15E-02	1,93E+05	0,391	-4,01
7	746	450	11,8	6	12	1,5	1,54E+07	4,93E+08	61041	4210	0,0224	187946,43	0,396	3,36E-04	1,46E-03	2,06E-02	2,04E+05	0,380	-4,10
8	740	450	10,2	6	12	1,5	1,54E+07	4,93E+08	62026	4803	0,0182	263901,1	0,333	3,95E-04	1,28E-03	1,65E-02	2,91E+05	0,317	-4,73
1	743	450	12,21	6	12	1,5	1,54E+07	4,93E+08	61447	4123	0,0255	161686,27	0,426	3,27E-04	1,52E-03	2,37E-02	1,74E+05	0,410	-3,70
2	752	450	14,33	6	12	1,5	1,54E+07	4,93E+08	63014	3706	0,0307	120716,61	0,496	2,86E-04	1,83E-03	2,86E-02	1,30E+05	0,478	-3,51
3	733	450	9,93	6	12	1,5	1,54E+07	4,93E+08	60978	4823	0,0175	275600	0,324	3,99E-04	1,23E-03	1,59E-02	3,04E+05	0,309	-4,77
4	724	450	7,73	6	12	1,5	1,54E+07	4,93E+08	59943	5727	0,0122	469426,23	0,247	5,04E-04	9,41E-04	1,08E-02	5,32E+05	0,232	-6,11
5	725	450	7,95	6	12	1,5	1,54E+07	4,93E+08	60048	5620	0,0127	442519,69	0,254	4,91E-04	9,69E-04	1,12E-02	5,00E+05	0,239	-5,92
6	726	450	8,26	6	12	1,5	1,54E+07	4,93E+08	60193	5477	0,0134	408731,34	0,265	4,73E-04	1,01E-03	1,19E-02	4,60E+05	0,250	-5,69
7	722	450	7,32	6	12	1,5	1,54E+07	4,93E+08	59571	5939	0,0114	520964,91	0,234	5,29E-04	8,85E-04	9,99E-03	5,95E+05	0,219	-6,41
8	717	450	6,14	6	12	1,5	1,54E+07	4,93E+08	58999	6457	0,0159571	404673,23	0,265	6,24E-04	7,35E-04	1,46E-02	4,42E+05	0,253	-4,36

Tab. 4.10 Variance of period for rigid and deformable foundations

The influence of the deformability of foundations with respect to the flexural deformation of the piers does not exceed percentage of 15-16%.

4.4.2. *Foundations on piles*

The foundation on piles are the most in the viaducts stock analyzed. As well as for shallow foundations, tables for geotechnical and seismic classification of the site have been compiled. For example, analysis conducted for a viaduct who is part of a family of six viaducts with the same project, are shown. In the same method for shallow foundations, tables and graphs for sensitivity analysis conducted for the foundation on piles, in relation to stratigraphy actually present on site, are presented.

The stiffness, calculated according to the Gazetas formulations [Gazetas (1991)], are referred to the single pile. In order to obtain a unique value of horizontal and rotational stiffness, representative of the entire group of piles, stiffness of individual piles has to be combined according to the arrangement of the piles in the plant.

Total horizontal stiffness can be calculated as the sum of the singles piles less than a coefficient E of efficiency of the group. Efficiency generally depends on the number and on interaction between piles. Poulos and Davis (1985) suggest E=0,25 for groups of piles of number of piles major or equal to 4, and centerline distance - diameter ratio equal to 3.

$$K_h^{tot} = E \cdot \sum_{i=1}^{n} K_h^i \tag{4.29}$$

This condition occurs in most cases for the foundations examined. For global rotational stiffness should be taken into account both the rotational stiffness of individual piles, and vertical stiffness. The expression used in this case is:

$$K_r^{tot} = E \cdot \frac{L_V}{L_V + h} \cdot \left(\sum_{i=1}^{n} K_v^i d_i^2 + \sum_{i=1}^{n} K_r^i \right) \tag{4.30}$$

Where L_V is the shear span and d_i is the center to center piles distance.

Efficiency is nearly uniform, and was put E=0,9.

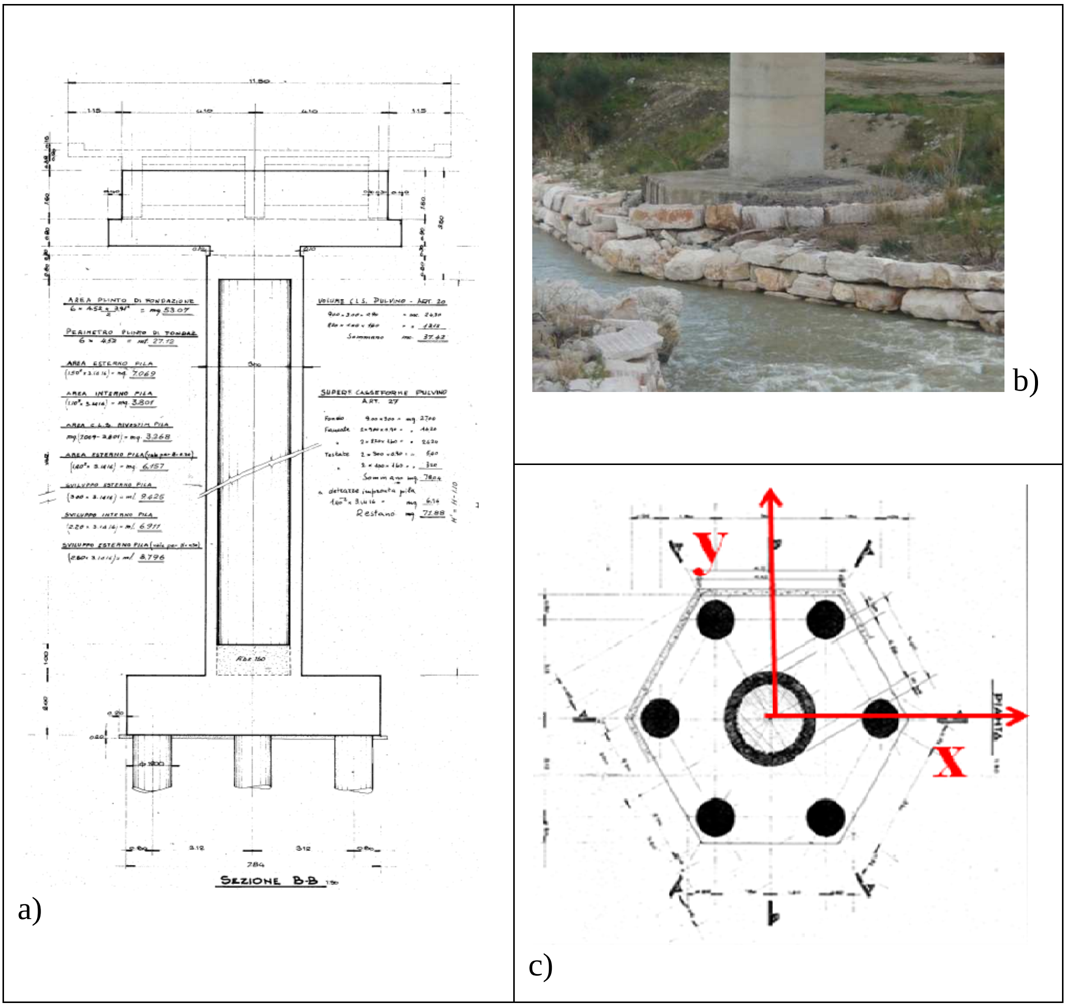


Fig. 4.9 Examples of viaduct whit on piles foundations. a) Longitudinal pier section; b) Picture of foundation; c) Foundation dimensions in plant. Dimensions are in meters.

N	Layers	from-to (m)	h_i (m)	V_S (m/s)	ϕ' (°)	C_U (kPa)	E_{ed} (kPa)
1	Gravel	0m-8,5m	8,5	390	28	0	13680
2	Sand clay	8,5m-18m	9,5	545		140	
3	Clay	18m-30m	12	635		286	

Tab. 4.11 Geotechnical parameters of soil layers

Pier Aboutment	x (m)	z (m)	i (°)	Cat. Top.	Soil class
Ab. A	0,00	242	5,110	B	T1
Pier 1	33,55	239	5,110	B	T1
Pier 2	67,05	236	0,000	B	T1
Pier 3	100,65	239	3,840	B	T1
Pier 4	134,15	241	3,410	B	T1
Pier 5	167,70	243	3,840	B	T1
Ab. B	201,21	245	3,420	B	T1

Tab. 4.12 Soils and topography categories for each pier

T_R (anni)	BEGINNING VIADUCT			END VIADUCT		
	a_g (g)	F_0	T_C*	a_g (g)	F_0	T_C*
30	0,054	2,424	0,292	0,054	2,424	0,292
50	0,068	2,426	0,324	0,068	2,427	0,324
72	0,080	2,452	0,339	0,080	2,452	0,339
101	0,094	2,457	0,345	0,094	2,458	0,346
140	0,110	2,465	0,346	0,110	2,466	0,346
201	0,129	2,499	0,351	0,128	2,501	0,351
475	0,181	2,525	0,366	0,180	2,527	0,366
975	0,237	2,502	0,383	0,236	2,502	0,384
2475	0,329	2,476	0,402	0,328	2,478	0,402

Tab. 4.13 Peak ground, acceleration, local amplification factor, and upper limit of the period of the constant spectral acceleration for beginning and end of viaduct

As presented for shallow foundations, the Fig. 4.10 show the difference of natural period of SDOF for rigid and deformable foundations. For each pier, height, the mass at the top (§ 5.2.7), shear waves velocity observed in situ, rotational and translational values stiffness of the foundation, yield stress moment at the base of the pier, and parameters of stiffness and fundamental period of the simply oscillator are evaluated in case of rigid and deformable foundation.

T_R (anni)	START VIADUCT			END VIADUCT		
	a_g (g)	F_0	T_C*	a_g (g)	F_0	T_C*
30	0,054	2,424	0,292	0,054	2,424	0,292
50	0,068	2,426	0,324	0,068	2,427	0,324
72	0,080	2,452	0,339	0,080	2,452	0,339
101	0,094	2,457	0,345	0,094	2,458	0,346
140	0,110	2,465	0,346	0,110	2,466	0,346
201	0,129	2,499	0,351	0,128	2,501	0,351
475	0,181	2,525	0,366	0,180	2,527	0,366
975	0,237	2,502	0,383	0,236	2,502	0,384
2475	0,329	2,476	0,402	0,328	2,478	0,402

Tab. 4.14 Seismic parameters and a degradation curve G/G_0

Shear wave velocity	**Vs**	547	m/s
Poisson ratio	**ν**	0,3	
Soil density	**ρ**	1,9	kg/m^3
Pile diameter	**d**	1,2	m
Pile length	**l**	23	m
Pile slenderness	**l/d**	19,16667	
Pile Young modulus	**E_p**	3,0E+07	kPa
Number of pier	**n**	6	
Center group distance	**δ**	3,6	m
Linear Young modulus	**E_0**	1,48E+06	kPa
Linear shear modulus	**G**	5,68E+05	kPa
Degradation curve	**G/G_0**	0,71	
Shear modulus	**G**	4,04E+05	kPa

Tab. 4.15 Geometrical foundations and mechanical soil characteristics

Analyses were conducted for the piers, as a function of soil class, maintaining a speed of shear waves comparable with the condition of foundations on piles. The influence of the deformability of foundations with respect to the flexural deformation of the piers does not exceed percentage of 15-16% for soft soil, and about 5-6% for the real soil for foundations analyzed.

SINGLE PILE				GROUP OF PILES	
HORIZONTAL STIFFNESS		ROCKING STIFFNESS		HORIZONTAL STIFFNESS	
K_{HH}/dE_s	2,02	K_Z/dE_s	1,436	K_H [kN/m]	7,55E+06
K_{HH} [kN/m]	2,55E+06	K_Z [kN/m]	1,81E+06		
ROCKING STIFFNESS		ROCKING STIFFNESS		ROCKING STIFFNESS	
K_{MM}/d^3E_s	1,85	K_{HM} [kNm]	2,49E+06	K_M [kN/m]	9,43E+07
K_{MM} [kNm]	3,36E+06	K_{MH} [kNm]	2,49E+06		

Tab. 4.16 Static foundations stiffness

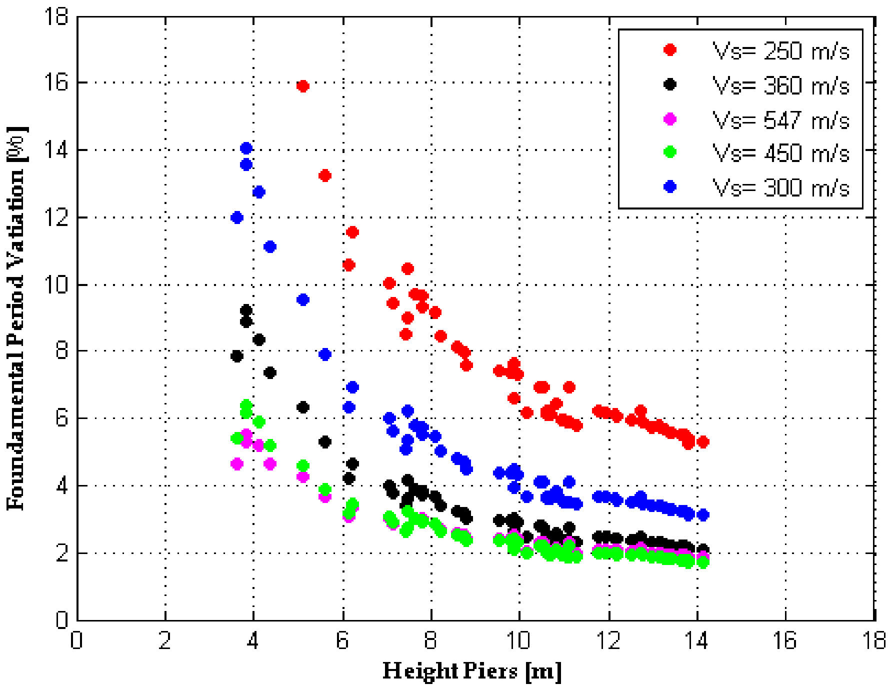


Fig. 4.10 Soil foundations effects for shallow foundation.

4.4.3. *Caissons foundations*

Assessments carried out for the viaduct with the homogenous caissons foundations in shown in tables below. The caisson foundations as well as ensure a high to lateral loads, restrict the movement of the landslide slopes, as in the case of these viaducts, for which the piers are attested near the slope. Some geometric characteristic is shown in Tab. 4.17 where R_b and R_t are the radius at the base and at the trunk of the caisson s is the cross section thickness, and D is the foundering.

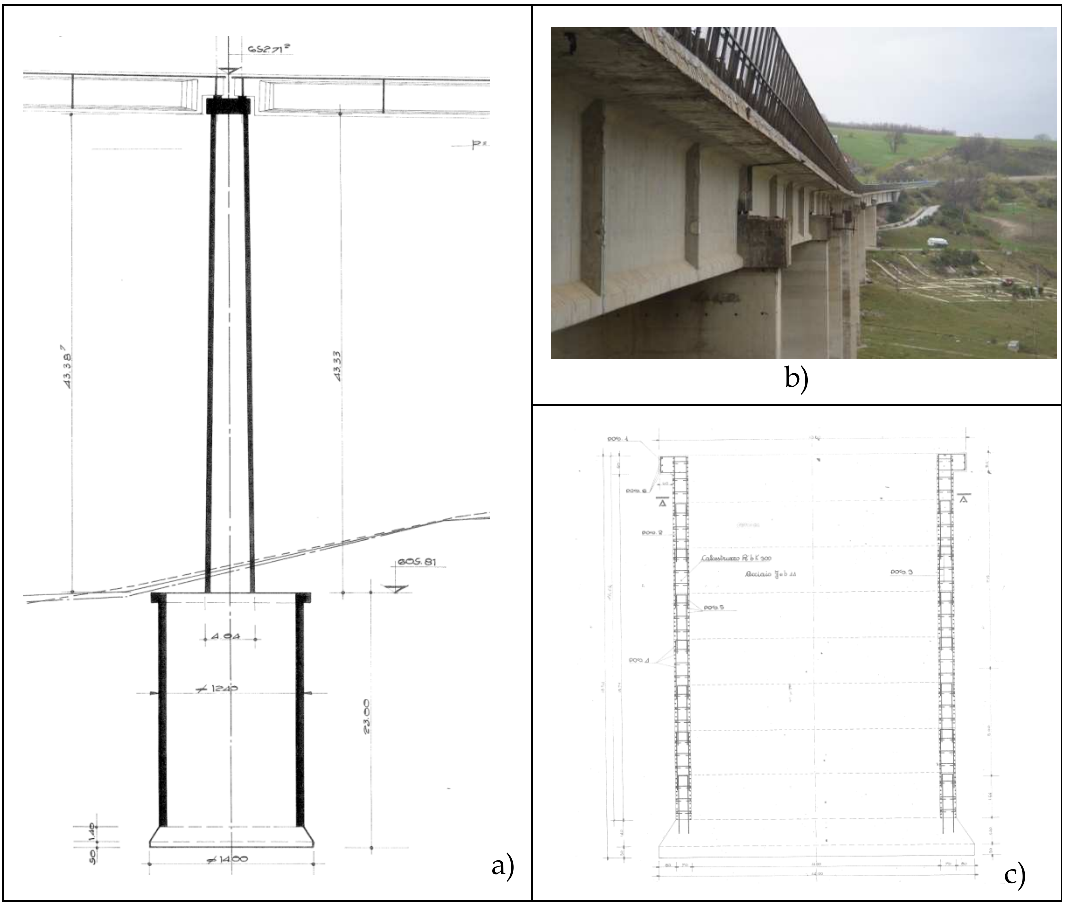


Fig. 4.11 Examples of viaduct whit caissons foundations. a) Longitudinal pier section; b) Picture of viaduct; c) Transversal section of caisson. Dimensions are in meters

N°Pier	Shape section	R_b (m)	R_t (m)	s (m)	H (m)	D (m)
1	Circular	7,00	6,20	0,70	16	~20
2	Circular	7,00	6,20	0,70	16	~20
3	Circular	7,00	6,20	0,70	16	~20
4	Circular	7,00	6,20	0,70	23	~27
5	Circular	7,00	6,20	0,70	23	~27
6	Circular	7,00	6,20	0,70	23	~27
7	Circular	7,00	6,20	0,70	14	~18
8	Circular	7,00	6,20	0,70	14	~18

Tab. 4.17 Geometric characteristic of caisson foundations

Pier Aboutment	x (m)	z (m)	i (°)	Cat. Top.	Soil class
Ab. A	0,00	660	14,26	C	T1
Pier 1	59,00	645	14,26	C	T1
Pier 2	118,00	630	13,34	C	T1
Pier 3	177,10	617	5,80	C	T1
Pier 4	236,15	618	2,91	C	T1
Pier 5	295,25	623	5,31	C	T1
Pier 6	354,43	629	6,75	C	T1
Pier 7	413,48	637	8,16	C	T1
Pier 8	472,93	646	7,93	C	T1
Ab. B	531,93	654	7,24	C	T1

Tab. 4.18 Soils and topography categories for each pier

T_R (anni)	BEGINNING VIADUCT			END VIADUCT		
	a_g (g)	F_0	T_C*	a_g (g)	F_0	T_C*
30	0,061	2,424	0,286	0,061	2,427	0,286
50	0,079	2,393	0,311	0,079	2,395	0,311
72	0,095	2,394	0,323	0,094	2,398	0,324
101	0,111	2,414	0,330	0,110	2,417	0,330
140	0,129	2,434	0,338	0,128	2,435	0,339
201	0,153	2,413	0,346	0,152	2,411	0,346
475	0,225	2,404	0,366	0,223	2,405	0,367
975	0,302	2,395	0,386	0,299	2,395	0,386
2475	0,428	2,390	0,428	0,425	2,384	0,429

Tab. 4.19 Peak ground, acceleration, local amplification factor, and upper limit of the period of the constant spectral acceleration for beginning and end of viaduct.

LIMIT STATE	T_R	a_g (g)	F_0	T_C*	S_s	S_t	S	a_s (g)	τmax (kPa)	γmax	G (kPa)	G/G0
SLO	120	0,12	2,425	0,334	1,50	1,00	1,50	0,18	24,65	1,85E-04	132998	0,72
SLD	201	0,154	2,413	0,346	1,48	1,00	1,48	0,227	31,54	2,70E-04	116849	0,63
SLV	1898	0,388	2,392	0,416	1,14	1,00	1,14	0,443	68,29	1,12E-03	60711	0,33
SLC	2475	0,428	2,390	0,428	1,09	1,00	1,09	0,465	73,29	1,31E-03	56114	0,30

Tab. 4.20 Seismic parameters and a degradation curve G/G_0

Shear modulus	**G**	56114	kPa
Young modulus	**E**	145896	kPa
Poisson ratio	**ν**	0,3	
Width of circumscribed rectangle	**L**	12,4	m
Lenght of circumscribed rectangle	**B**	12,4	m
Effective soil-striucture contact	**D**	4	m
Height	**H**	14	m
Soil density	**ρ**	1,9	kg/m^3
Shear wave velocity	V_s	311,625	m/s
Linear shear modulus	G_0	184509	kPa
Degradation curve	G/G_0	0,30	

Tab. 4.21 Geometrical foundations and mechanical soil characteristics

K_{HH}	1,076E+07	KN/m
K_{MM}	6,066E+08	KN/m
K_h	3,822E+05	KN/m^2
K_θ	1,264E+07	KN/m^2

Tab. 4.22 Static foundations stiffness

The basic and most common seismic analysis of geotechnical systems consists in pseudo-static calculations, in which the soil/structure seismic interaction is studied modeling the dynamic action as an equivalent static force.

In the following part, numerical analyses were described, in which the ultimate horizontal capacity of the caisson was evaluated in order to obtain the maximum horizontal force which could be applied to the caisson as pseudo-static action.

The caisson model was created using Plaxis 8.0 [Brinkgreve, R.B.J, Plaxis 2D (2002)], a FEM code optimized for geotechnical problems. The model was built in plane strain conditions, differently from the 3D behavior of the caisson foundation. The caisson has cylindrical shape with a base diameter of D_e=14 m and a trunk diameter of D_i=12,4m. The height of the foundation structure is H=16m. The superstructure was constituted by a 30 m high pier.

The soil and caissons materials properties used for the analyses were showed in Tab. 4.23. The soil was considered as a purely frictional material (loose sand).

A Mohr-Coulomb model was adopted to control the soil failure due to the horizontal force. No water table was considered in the analyses; therefore the analyses were carried out in total/effective stresses. In the Tab. 4.23, γ is the unit weight, E is the Young modulus, ν is the Poisson ratio and ϕ is the friction angle of the soil. At first, in the numerical analyses the ratio between the friction at soil/caisson interface δ and the friction of the soil ϕ was set as $\delta / \phi = 0{,}1$.

Parameter		Soil	Foundation material
γ	(kN/m^3)	19	19,11
E	(kN/m^2)	14600	30000000
ν	(-)	0,3	0,3
ϕ	(°)	31,5	-

Tab. 4.23 Material properties for the numerical analyses.

From this assumption a strong reduction of the interface friction angle δ was adopted, which was 10% of the soil friction angle.

In the Tab. 4.23, the mechanical properties of the caisson itself were showed: the parameters were chosen considering the caisson made by concrete. The material model for the structure was linear elastic. Concerning the value of unit weight of the foundation γ_f, the original value for the concrete was modified in order to account for the differences between the 2D model and the 3D real foundation.

The caisson model were submitted on the top to a set of loads: a vertical force N_0, which was derived by the superstructure loads (pier weight, beam load and overload); a horizontal load H_0, which represented the horizontal bearing capacity of the caisson; a bending moment M_0, which was given by the product of the horizontal force H_0 and the arm between the point of application of H_0 and the top of the caisson (height of the pier).

The horizontal and vertical forces were applied as point loads, instead the bending moment was applied as a linear distribution of loads. The results of the analyses were showed in terms of normal stresses on the lateral walls of the caisson (Fig. 4. 2). The horizontal stresses were compared with the theoretical values obtained from the Rankine's theory (in the hypothesis of no soil/structure friction). The horizontal stresses followed the closed-form Rankine's distribution in the top part of the caisson. The change occurred because the rotation centre is inside the caisson: the counter-rotation caused in the bottom part of the caisson (from the rotation centre to the bottom side) an increment of horizontal stresses on the left side, and a reduction in the right side. Net graphs of the normal horizontal stresses, difference between left and right components, were also plotted in Fig. 4.12 together with the analytical distribution. The numerical and analytical results gave a good agreement: the graph inversion point was similar between the two distributions, which determined a very similar kinematics (same height of the rotation point).

A set of parametric analyses were performed, starting from the initial one, varying the geometry of the caisson and the interface properties.

Seven different geometries were prepared, considering a constant value of the base of the caisson and changing its height, in order to obtain different slenderness ratios (λ=H/D=0,5; 0,75; 1; 1,25; 1,5; 2; 3).

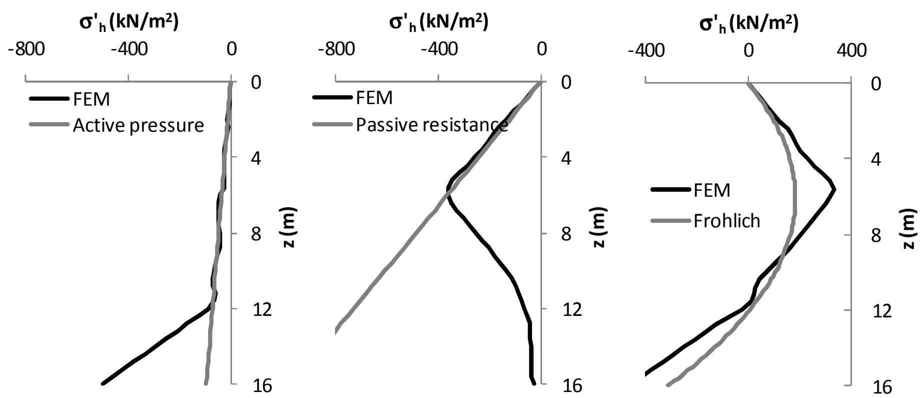


Fig. 4.12 Distribution of horizontal pressure along the caisson: left, right and net diagram.

This ratio values were organized to cover possible construction ranges of these structures, from the squat caisson (low λ) to the slender caisson (high λ). Therefore this type of structures is an ideal hyphen between the shallow foundations (squat caisson) to single large diameter pile (slender caisson).

For each geometry three different interface friction ratios were considered (δ / ϕ=0,1; 0,5; 1). Totally 21 models were analyzed. In each analysis the vertical force value N was obtained as the summation of the initial N_0, which was kept constant in all the analyses, and the weight of the caisson W, which was variable depending on the caisson geometry. Also the bending moment M_0 was evaluated considering a constant value for the force arm.

In Fig. 4.13 the dimensionless ratio H_u/N, between ultimate horizontal load and total vertical force were displayed against the slenderness ratio H/D. Three curves were plotted for the three values of the interface friction ratio δ/ϕ.

The trend of the H_u/N, curves was clearly linear with the value of λ. Same increments were observed together with the increment of friction ratio δ/ϕ.

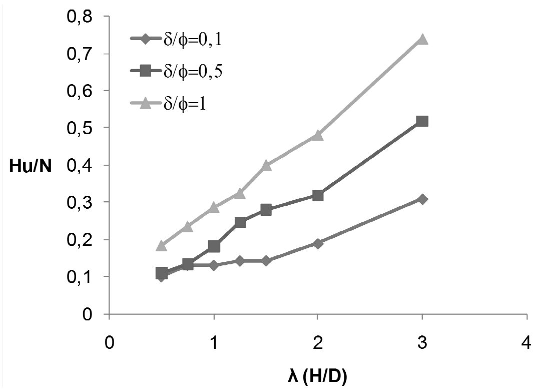


Fig. 4.13 Dimensionless ratio H_u/N against slenderness ratio H/D

In order to plot the net diagrams of the normal horizontal stresses σ'_h for all the models, these pressure were reported against a dimensionless height of the caisson z/H in Fig.4.14 for two values of friction ratio δ/ϕ (0,1 and 0,5).

The maximum values of the net horizontal stresses were increased with the value of the slenderness ratio, because of the increments of the horizontal ultimate load capacity. The rotation centre varied with λ, observing a downward shift with the increment of the slenderness ratio. For the δ/ϕ=0,1, the rotation centre were located in a range between 70÷80% of the total height; for the δ/ϕ=0,5, the range was between 86÷93% of H .

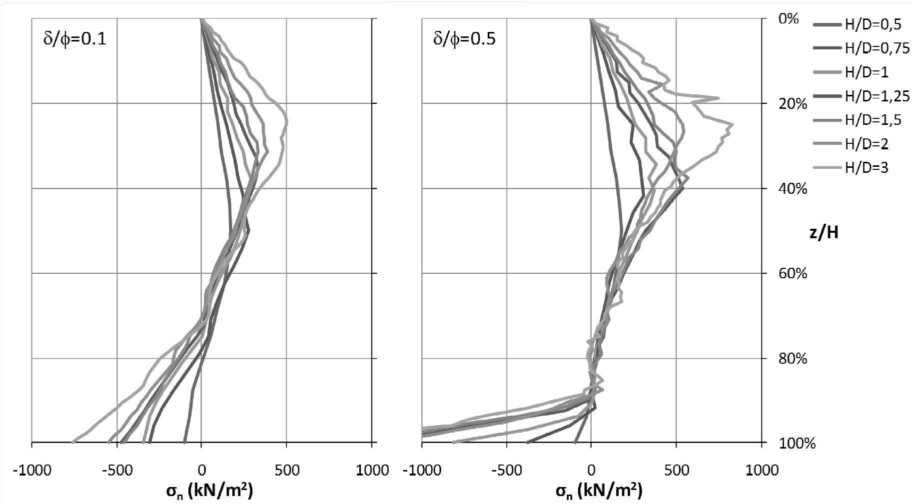


Fig. 4.14 Horizontal net pressure against z/H

4.5 CONCLUSIONS

In the case of distributed infrastructures, such as bridges, both in design of new structures and assessment of existing one the assumption of rigid foundations is sometimes impossible or even wrong, and the stiffness of the soil-structure system has to be evaluated. An equivalent stiffness to take into account the soil-structure interaction has to be properly defined, since it has an influence of the natural period of the structure. In fact, rigid foundations lead to an overestimation of the global ductility of the system. After the in-situ geological and geophysical investigations, a database of information required for the mechanical characterization of soils and the development of geotechnical models and the seismic characterization of the sites have been set. A population of 244 foundations has been analyzed, classified in three different types: shallow, foundations on piles, and caissons. For each bridge, the equivalent static stiffness to take into account the soil-structure interaction has been evaluated for the above mentioned three type of foundations, in agreement with the level of analysis (§5) and the methodologies developed by Gazetas. The obtained values for stiffness have been used for the seismic vulnerability assessment of the bridges and the computation of the displacement on top of the structures taking into account the contribution of the deformable foundation.

For each structural typology sensitivity analyses have been carried out. For shallow and on pile foundations they allowed the evaluation of the deformability of foundations on the natural period of the SDOF system for the soils in the database and even for additional soils compatible with the structural typologies.

For shallow foundations, the influence on the natural period can be quantified in a variation of about 4-6% according to the characteristics of the soil. The difference is higher for short piers, with a variation up to 8%. For the low shear wave velocities such a difference with respect to the case of rigid foundations can be up to 15-16%. For on piles foundations, same results are checked.

For caissons numerical analysis have been carried out, in which the ultimate horizontal capacity was evacuate in order to obtain the maximum horizontal force which could be applied to the caisson as pseudo-static action.

The analyses showed a good agreement with the analytical formulation despite of the model simplification (2D instead of 3D). The following research step will be the execution of 3D pseudo-static analyses, and dynamic analyses in order to account the effect of kinematic soil/structure interaction on the response of the caisson

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Chapter 5

SEISMIC VULNERABILITY ASSESSMENT

5.1 INTRODUCTION

The most common analysis method for bridges is modal spectral analysis. A common design consideration in Europe and U.S. is that the superstructure must resist the seismic elastically. The deck is capacity designed to remain elastic when the ductile behavior of a bridge is chosen. Bridges that can be modeled as SDOF oscillators, such as single span bridges, multi span bridges consisting of simply supported span, or multi span bridges in general can be analyzed using the equivalent static method.

For populations of bridges and viaducts, advanced methods for studying the vulnerability through BMS systems, is the implementation of simplified mechanical models (HAZUS), where the seismic demand is characterized by the use of response spectra, while the capacity is determined through simplified models of possible failure mechanisms. The objective of HAZUS project is to define a bridge classification that can utilize the available data and can provide damage and repair cost estimates comparable with the damage data observed in the past earthquakes. The following tasks are performed as part of this project:

- Review available bridge classifications
- Develop an improved bridge classification
- Generate damage functions for the new classification
- Refine available damage state-repair cost ratio relationship.

The ATC-13 [1985] study and HAZUS [1997] include the three bridge classifications, multiple simple spans, continuous monolithic included simple spans and greater than 500 ft span, which are currently used for vulnerability assessment of bridges. This is a very broad classification and neglects various structural characteristics that affect the seismic performance of a bridge, such as structure type and material, pier bearing types. The bridge classification in HAZUS does not address the effect of the structural material and type, substructure type, and design details, such as column reinforcement and/or seat width. Basöz and Kiremidjian (1996) developed a more detailed classification in which bridges in the same sub-category are expected to experience similar damage under a given seismic loading. In their classification, bridges are grouped according to number of spans, superstructure type, substructure type and material, abutment type, and span continuity. Then, these bridges were further classified into sub-categories based on other structural characteristics, such as number of spans, abutment type, column bent type and span continuity. Empirical damage probability matrices and fragility curves were developed for each of these bridge sub-categories using the damage data from the Northridge and Loma Prieta earthquakes [Basöz and Kiremidjian, (1997)]. As part of this project, a new bridge classification is developed to be used in HAZUS. The new bridge classification is based on the following structural characteristics of bridges:

- Number of spans: single vs. multiple span bridges
- Structure type: concrete, steel, others
- Pier type: multiple column bents, single column bents and pier walls
- Abutment type and bearing type: monolithic vs. non-monolithic; high rocker bearings, low steel bearings and neoprene rubber bearings
- Span continuity: continuous, discontinuous (in-span hinges), simply supported.

The seismic design of a bridge is taken into account in terms of the spectrum modification factor, strength reduction factor due to cyclic motion, drift limits, and longitudinal reinforcement ratio. The proposed bridge classification is an improvement over the one currently used in HAZUS, as it is more general. Furthermore, it incorporates various structural characteristics that affect damage into fragility analysis and provides a means to obtain better fragility curves when data become available.

An overview of Italian literature, existent Guidelines for Evaluation of seismic safety for existing bridges works [Pinto et al., (2009)] useful for single structure and for population's analysis.

5.2 SEISMICALLY PERFORMANCES EVALUATIONS

Structural model must reflect state of the structures. It is defined in order to adequately describe the relevant degrees of freedom that characterize the structural response under seismic actions.

For bridges and viaducts, generally, the desk is not significantly involved in the seismic response of the structure. It follows that attention has to be sent so prevalent in substructures (piers and abutments) and foundations, appropriate restraint systems, and interconnection between structural elements (bearings, couplings, etc.) [DPC-Reluis 2005-2008 Linea 3]. Considerations about limits states achievement is referred at piers, abutments, bearings, and foundations. The use of the bearings provide choice as to how and where seismic forces has to be resisted. Problems with attracting excessive force to short stiff piers can be solved placing bearings between columns and superstructures. When elastomeric bearings are utilized, it is possible to compensate for different stiffness of different piers by adjusting the bearings stiffness at the top of the piers.

The most of the viaducts under study, belong to the category of simply supported bridges, which are a widespread type of structure in the country.

The bridges studied belong to this category, on simple columns. In these conditions, is possible to define, accurate analysis methods characterized by a level of complexity consistent with the purpose of the seismic vulnerability and easily useful also for other structural types, as multi columns bent. Tridimensional multi column bent in plant are not present in the stock.

When the support system consist of single column bents, response in longitudinal and transversal directions consist of basic vertical cantilever behavior.

If the superstructures is bearing supported, and pier cross section is symmetric, response characteristic can be made equally transversal and longitudinal, optimizing the seismic design. However, if shear keys are used to restraint transverse displacement, transverse and longitudinal period and hence seismic force can be different. Since there will be only plastic hinge location, the behavior is easy to determine, whit a high degree to accuracy. Using multicolumn design the behavior evaluation is a little more complicated, but moment induced at the base and displacement at the top will be lower.

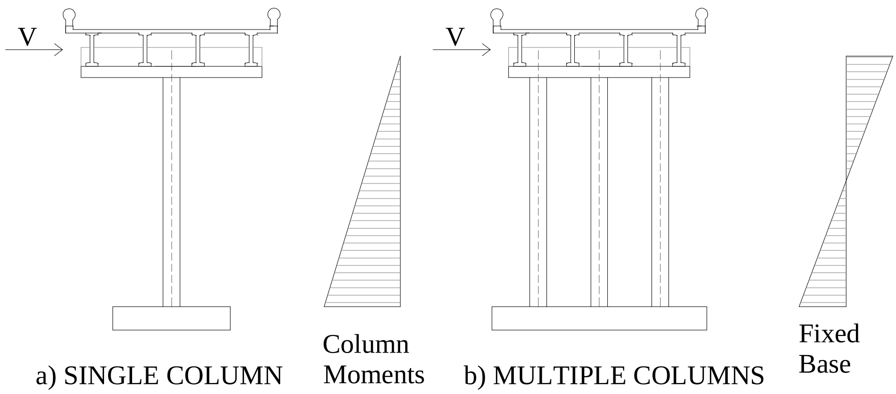


Fig. 5.1 Different bent configurations

The procedure presented should in general be applied to very simple bridges and to a preliminary design phase of bridges for which the coupling effects of the deck can be neglected, in which case each pier is considered a SDOF system. The seismic force is represented by an acceleration/displacement spectrum. The bridge model results from the appropriate combination of stiffness, mass, and damping of the structural elements. The period of vibration and damping of the substitute structure will allow acceleration and displacement to the read directly from the spectra. The characteristics of the substitute structure are mass, effective global stiffness, and damping. Geometrical considerations, including the effects of foundation flexibility, influence the relationship between structural displacement ductility factor and member ductility factor, which may be expressed in curvature, rotation, or displacement units. Using the displacement-based design procedure, it is the plastic rotation of potential plastic hinges that is of greatest interest.

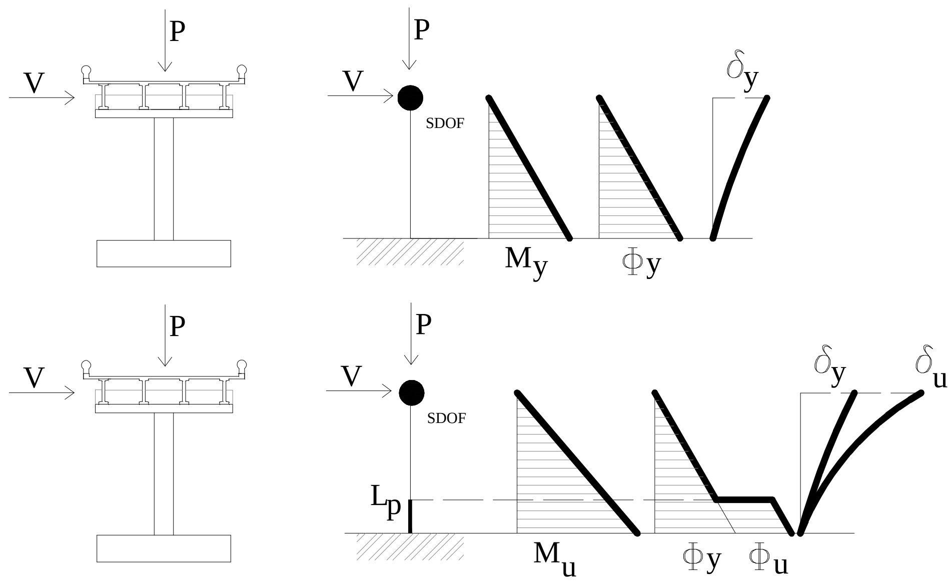


Fig. 5.2 Assessment of structural displacements in the isolated piers

From this, the displacements at the top of SDOF system is calculated, and hence the basic force requirements established.

Analysis are conducted by a model of non-linear static analysis on displacement based design procedure who take into account :

- Lumped plastic hinges model mechanisms;
- Mechanical non linear effects due to materials;
- Geometrical non linear effects due to the element slenderness;
- Longitudinal and rocking foundations deformability.

If considering two SDOF systems of equal mass and stiffness characteristics, one of which whit rigid foundation and the other with a deformable foundation, the response in terms of column displacement ductility factor could be different.

Available ductility is given by:

$$\mu = 1 + \frac{\Delta p}{\Delta y} \tag{5.1}$$

where:

- Δp is system's ductility ;
- Δy is yield ductility.

Value of yield ductility of the rigid system is less than of the system with deformable foundation, so, with the same deformability of the system Δp, it result:

$$\mu^A > \mu^B \tag{5.2}$$

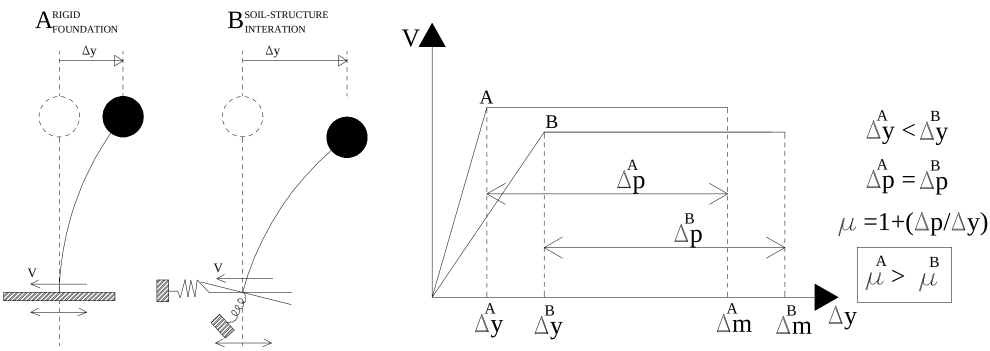


Fig. 5.3 Behavior of system whit interaction soil- foundation: A. System model whit rigid foundation; B. System model whit deformable foundation.

The approach presented, is detailed in following paragraphs.

In the transversal direction piers, or multicolumn bents are considered independent, defining a set of simple SDOF systems (Fig. 5.2). With respect to the longitudinal direction two assumptions are made. The first assume that the entire bridge is single oscillator for which mass is the sum of the masses of the individual piers and stiffness is obtained by the composition of the force-displacement parameters of single systems. This assumption is correct in the case of regular structure and when displacements of single span deck respect to the pier cap is very smalls very small.

This requires the presence of seismic restraints. In the second case, in the absence of the validity of the first hypothesis, the model in longitudinal direction is the same of the transversal direction. In this case, single columns bents and on frame systems are considered individually, each as a simple oscillator, in line with what is shown in Fig. 5.4.

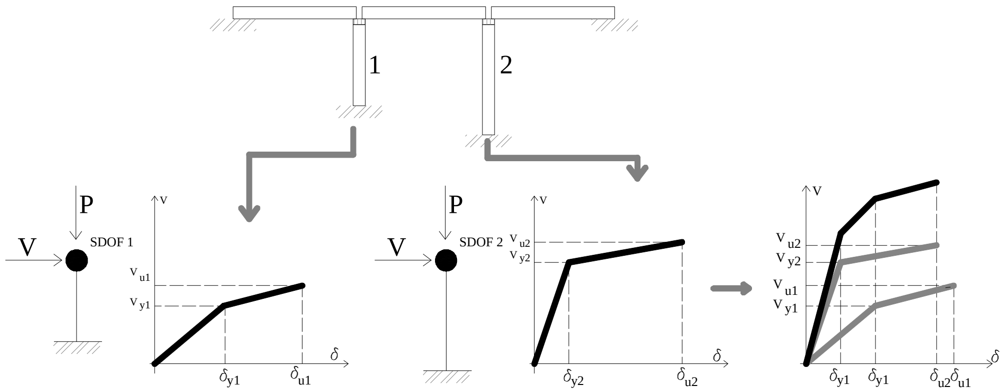


Fig. 5.4 Longitudinal evaluation of structural seismic response, assembly of the response of individual piers the longitudinal direction

5.2.1. *Stress-strain relationship for existing structures*

For the implementation of a nonlinear analysis procedure, it is necessary to define appropriate stress-strain relationship able to represent the behavior of the materials, concrete and steel.

Confinement of the concrete is improved if transverse reinforcement layers are placed relatively close together along the longitudinal axis. There will be some critical spacing of transverse reinforcement layers above which the section midway between the transverse sets will be ineffectively confined. It is generally found that a more significant limitation on longitudinal spacing of confinement reinforcement s is imposed by need to avoid buckling of longitudinal reinforcement under compression load. This condition is not often encountered condition in existing bridges. So, due to the lack of reinforcement details, compression stress-strain relationships for unconfined concrete are used.

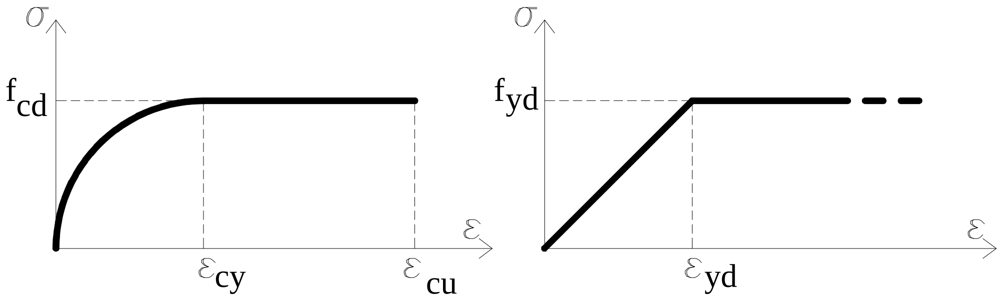


Fig. 5.5 Unconfined concrete stress-strain relationships and elastic-perfectly plastic stress-strain low for steel.

For unconfined concrete stress-strain relationships are referred to the Italian codes [NTC (2008)]for classes of concrete lower then C45/55.

Parabolic–rectangular $\sigma - \varepsilon$ is used, based on design compression strength and f_{cd} ultimate strain ε_{cu}:

$$\varepsilon_{cy} = 2\text{‰} \tag{5.3}$$

$$\varepsilon_{cu} = 3{,}5\text{‰} \tag{5.4}$$

The values employed, although conservative, are justified by the very modest level of structural details of the longitudinal reinforcement and transversal confinement, and the reduced thickness of concrete cover subject to cyclic loads during the structures exercises. With regard to steel, elastic perfectly plastic stress-strain relationships undefined in used. As required in Italian codes [Circolare 617 (2009)] in absence of information in assumed that the ultimate strain of steel is equal to 4%.

5.2.2. *Stiffness and moment rotation relationship*

The restoring force term in general equation of motions

$$m\ddot{u}_s + c\dot{u}_s + ku_s = -m\ddot{u}_g \tag{5.5}$$

depend on the stiffness body system. The translational stiffness for slender bridge can be expressed as:

$$k = \alpha \frac{EI}{H^3} \tag{5.6}$$

where E is the modulus of elasticity, I in the effective moment of inertia of the cross section, H is the effective column height, and the coefficient α represent the boundary conditions.

In the transversal direction, the bridge piers deform as cantilever, and stiffness can expressed as (Fig. 5.6.a):

$$k = 3 \frac{EI}{H^3} \tag{5.7}$$

In longitudinal direction, assuming a stiff of rigid superstructure, the stiffness term in double curvature bending whit both ends fully constrained against rotation can be expressed (Fig. 5.6.b):

$$k = 12 \frac{EI}{H^3} \tag{5.8}$$

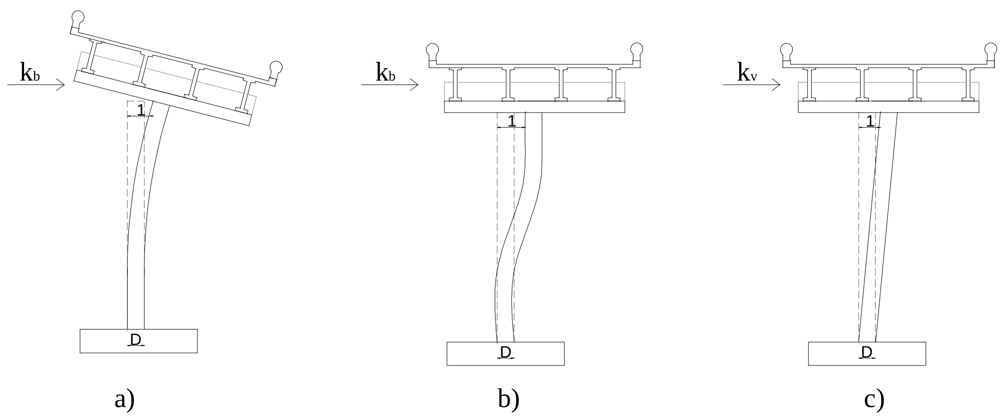


Fig. 5.6 Bridge stiffness terms for lateral displacement.

For squat bridge piers, where the clear column height H is no longer significantly than larger than the column depth D, shear deformation can become significant in comparison whit the flexural deformation. The shear deformation for a unit load or the shear flexibility, can be expressed as (Fig. 5.6.c):

$$f = \frac{H}{A_{ve}G} \tag{5.9}$$

where A_{ve} represent the effective shear area and G the shear modulus of the pier cross section. As a general rule shear deformation can become significant when the significant when the shear span M/V of the pier is less than three times the pier depth D [Priestley et al. (1996)]:

$$f = \frac{H}{A_{ve}G} \tag{5.10}$$

In displacement-based design, the elastic stiffness is required at the start of the design, in order that the elastic periods of the structure can be defined, and also, at a later stage of the design to distribute the total inertia force to members in proportion to their initial stiffness. To reflect the cracked state of a concrete bridge column in the seismic response analysis, and to take in account the level of cracking in the elements and the deformation materials level, in nonlinear plasticity concentrated analysis an effective or cracked- section moment of inertia I_{eff} is used. The effective stiffness $E_c I_{eff}$ does not reflects only the effect of cracking, but also the state of the bridge column determined at first theoretical yield of the reinforcement. Different relationship are available in literature to define the effective stiffness $E_c I_{eff}$ [Pinto et al.(2009), Priestley et al. (1996); Less loss].

More realistically, stiffness can be assessed from the moment-curvature relationship. The reduction of the flexural stiffness is determined by an ad-hoc moment curvature relationship, constructed by a mathematical program, which is also used to estimate the yield moments and curvature.

The value of ultimate moment, along whit the corresponding value of ϕ_y is determined from analysis of the cross-section, on the basis of:

- Plane section hypothesis;
- Elastic perfectly plastic $\sigma - \varepsilon$ low for steel;

- A parabolic $\sigma - \varepsilon$ low for concrete up to the compressive strength f_c at a strain $\varepsilon_{cy} = 2\permil$, followed by a rectangular branch up to $\varepsilon_{cy} = 3{,}5\permil$

PIER CROSS SECTION MOMENT ROTATION RELATIONSHIP

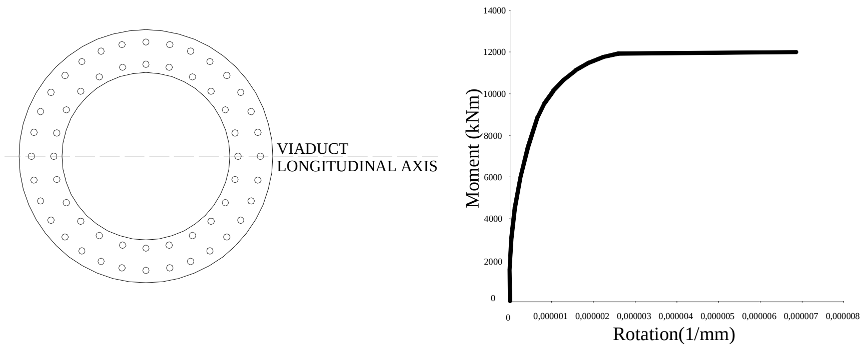


Fig. 5.7 Moment rotation relationship for a typical circular column

A yield criterion consisting of the following, whichever occurs first:

- Yielding of the reinforcement over one-third of the part of the perimeter that falls within the tension zone (for circular section), or yielding of the tension reinforcement (for hollow rectangular piers);
- Attainment of $\varepsilon_{cy} = 2\permil$ at the extreme compression fibers.

5.2.3. ***Geometrical non linear effects***

The effects of geometric nonlinearity are included in the analysis through the column model analysis approach. This method is applicable to isostatic systems, whit constant geometry and reinforcement cross section. The axial force must be constant along the longitudinal axis. However there are not limitation, for transversal loads.

The curvature ϕ of the section at the bottom of the structure should be considered as a parameter model. To connect the horizontal displacement f and the curvature ϕ sinusoidal deformation is assumed:

$$v(z) = f\left(1 - \cos\frac{\pi z}{2L}\right) \tag{5.11}$$

that satisfies the boundary conditions:

$v(z = 0) = 0$; $v'(z = 0) = 0$; $v(z = L) = f$

The bottom curvature is given by v'' (z = 0), so

$$v''(z) = f\,\frac{\pi^2}{(2L)^2}\cos\frac{\pi z}{2L} \Rightarrow \phi = v''(z=0) = f\,\frac{\pi^2}{(2L)^2} \tag{5.12}$$

Since unsupported length of the element it is equal to 2L, can be written:

$$f = \phi\cdot\frac{L_o^2}{\pi^2} \cong \phi\cdot\frac{L_o^2}{10} \tag{5.13}$$

The second order moment is equal to:

$$M_{II} = P\cdot f \cong P\cdot\phi\cdot\frac{L_o^2}{10} \tag{5.14}$$

And the total moment is equal to:

$$M_I + M_{II} = M_I + \phi\cdot\frac{PL_o^2}{10} \tag{5.15}$$

The moment – rotation at the base of the element relationship can be represented by linear equation, whit inclination equal to $PL_0^2/10$ dependent by the applied axial load [Cosenza et a. (2008)], as shown in Fig. 5.8.

Fig. 5.9 show some compress element whit different constraints at the ends. [CEN (2004)].

Bridge structures in this study, are present in case of Fig.5.9.g, in which in case of framed piers is further introduced bending stiffness for taking to account for the bending stiffness.

The unsupported length of column, be used to define the stress-strain relationships of the cross sections, after detracting second order effects, is given by:

$$L_0 = L \cdot \max\left\{\sqrt{1+\frac{10\cdot k_1 k_2}{k_1+k_2}};\left(1+\frac{k_1}{k_1+1}\right)\cdot\left(1+\frac{k_2}{k_2+1}\right)\right\} \tag{5.16}$$

The relationship depends on the deformability of constraints at the end of the element.

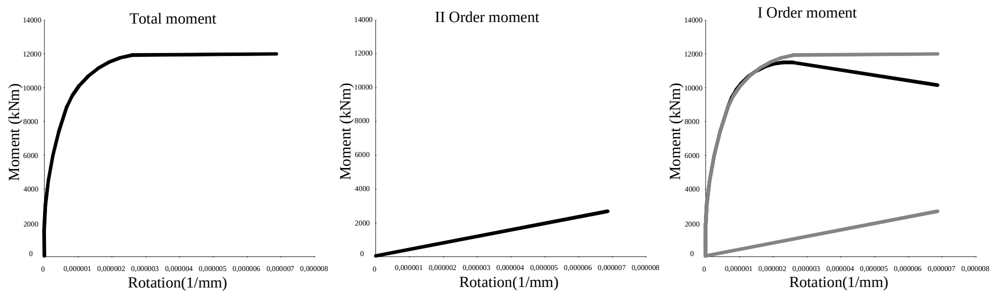


Fig. 5.8 Moment-rotation relationship: I and II Order effects

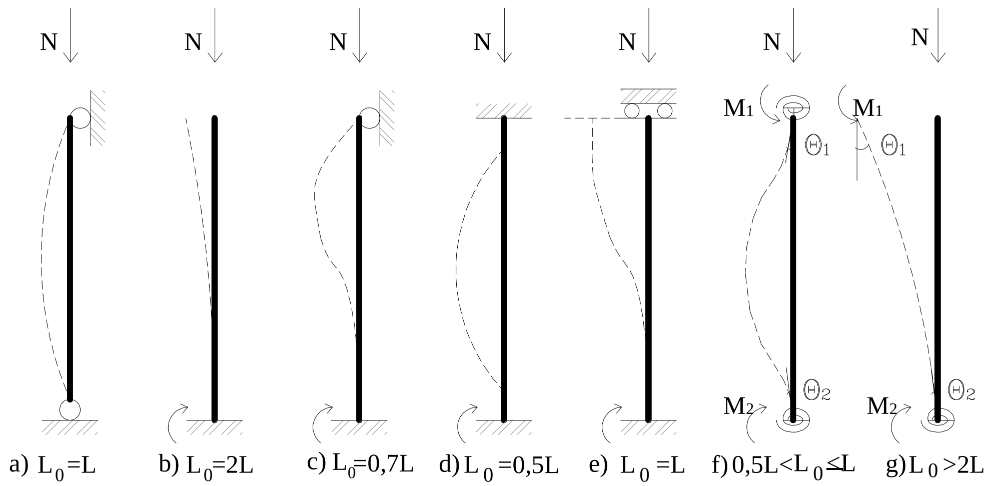


Fig. 5.9 Deformed and corresponding unsupported length of column.

For each of the deformation constraint the following parameter is introduced:

$$k_i = \frac{\vartheta_i}{M_i}\cdot\frac{EI}{L} \tag{5.17}$$

The equation (5.17), represent the relationship between rotation of the end sections ($i = 1{,}2$) and moment at the same section, appropriately normalized by the flexural stiffness EI and the distance between the ends of constraints.

The procedure for purifying the critical section of the second order effects and assess only the effects of lateral bending induced by the seismic action is well summarized in Fig. 5.8, where we observe the total moment-rotation relationship $(M-\phi)$, the contribute of second order effects evaluated using the method described in the column model, and the resulting element relationship characterized by a descending branch.

5.2.4. *Bi- linear moment rotation relationship for degrading performance of pier*

A bilinear approximations to the moment curvature relationship for critical section is required. In cases where the response of the SDOF system is degrading, for the evaluation of the seismic force-displacement relationship a bi-linear response that is able to reproduce the degrading elements response is required.

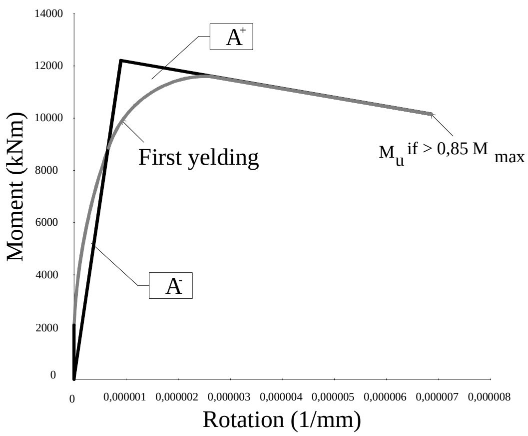


Fig. 5.10 Piecewise linear of moment rotation relationship for degrading performance of pier

Elements necessary to determine the piecewise linear moment rotation relationship are the first yielding moment and the decay of the bearing capacity due to non linear geometric phenomena.

The first order moment-rotation relationship, the maximum resisting moment of the structure M_{max} and the yielding moment M_y are computed. The ultimate moment M_u and rotation ϕ_u are computed by considering a reference value equal to M_u if M_u>0,85M_{max}or 0,85 M_{max} otherwise. The elastic branch is obtained from the yielding point and the yielding moment is obtained by imposing that the bilinear and the real curves have the same underlying area, with the ultimate rotation ϕ_u corresponding to the above mentioned value for M_u.

The plastic curvature capacity is the difference between ultimate curvature, corresponding to the limit compression strain ε_{cu} and the yield curvature:

$$\phi_p = \phi_u - \phi_y \tag{5.18}$$

This plastic curvature is assumed to be constant over the equivalent plastic hinge length l_{pl} which is calibrated to give the same the same plastic rotation as occurs in real structures. Plastic hinge length is estimate as [Circolare 617 (2009)]:

$$l_{pl} = 0{,}1 \cdot L_V + 0{,}17 \cdot H + 0{,}24 \frac{d_{bL} \cdot f_y}{\sqrt{f_c}} \tag{5.19}$$

where $L_V = M / V$ is the ratio moment/shear at the end section and d_{bl} is the diameter of longitudinal reinforcement.

5.2.5. *Effective seismic mass and control point height*

The mass of a bridge system, which contribute to the seismic response in the form of inertia forces, cab be characterized by the weight if the moving portion of the bridge divide by the gravitational constant g .

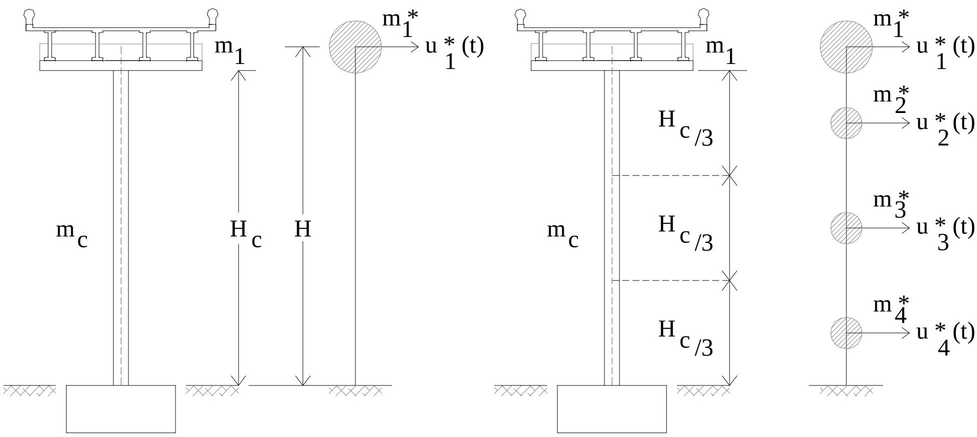
The simplest case of mass model used in bridge design assume that the entire mass is concentrated in superstructure, and the mass of the pier is negligible. However, the mass of the pier is large, so a percentage of mass of the pier height H_c can be added to superstructure mass at the height H.

Assumed uniformly distributed mass m_c along the column height, the generalized mass $m*$ which characterizes contributions from the distributed column mass to generalize displacement $u*$ can be expressed for the mass components m as [Priestley, M.J.N. et al. (1996)]:

$$m* = \frac{m_c H_c}{3} \tag{5.20}$$

So, mass has to be considered at the head of each pier is mass of the deck, pier cap, and 30% of the pier mass.

$$m = \frac{0{,}3W_{pier} + W_{piercap} + W_{deck}}{g} \tag{5.21}$$



a) Monolithic connection a) Higher modes in pier

Fig. 5.11 Idealized inelastic column response model

The height at which this mass is located is given by:

$$H = \frac{\left(m_{piercap} + 0{,}3m_{pier}\right)H_p + m_{deck}H_{deck}}{m} \tag{5.22}$$

where:

- H_p is the mass center of the pier cap from the extrados of the foundation;
- H_{deck} is the mass center of the deck from the extrados of the foundation.

In Fig. 5.12 the distribution of the first order bending moment along the shaft of the pile in shown.

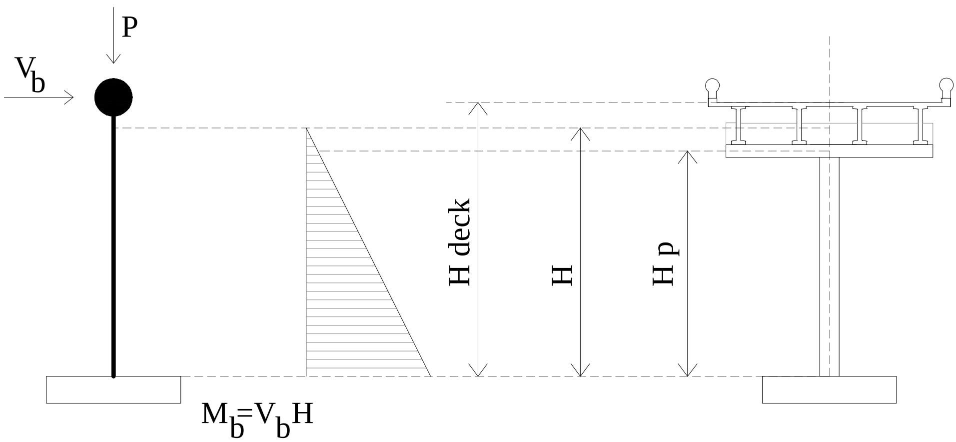


Fig. 5.12 Geometry of the SDOF system

5.2.6. *Determination of the idealized force – displacement relationship*

To obtain force-displacement diagram, required for vulnerability analysis, the model used is shown in Fig.5.13.

Considering pier-foundation system, modeling by one degree of freedom system, whit mass and stiffness definite values, for each horizontal force F assigned, total displacement δ is given by four components:

$$\delta = \delta_h + \delta_\varphi + \delta_f + \delta_v \tag{5.23}$$

where:

- δ_h rigid displacement due to the foundation translational deformation, represented by the spring K_h;
- δ_φ displacement due to the foundation rotational deformation by the rigid rotation, represented by the spring K_φ;

- δ_f displacement due to the bending pier deformation;
- δ_v displacement due to the shear pier deformation.

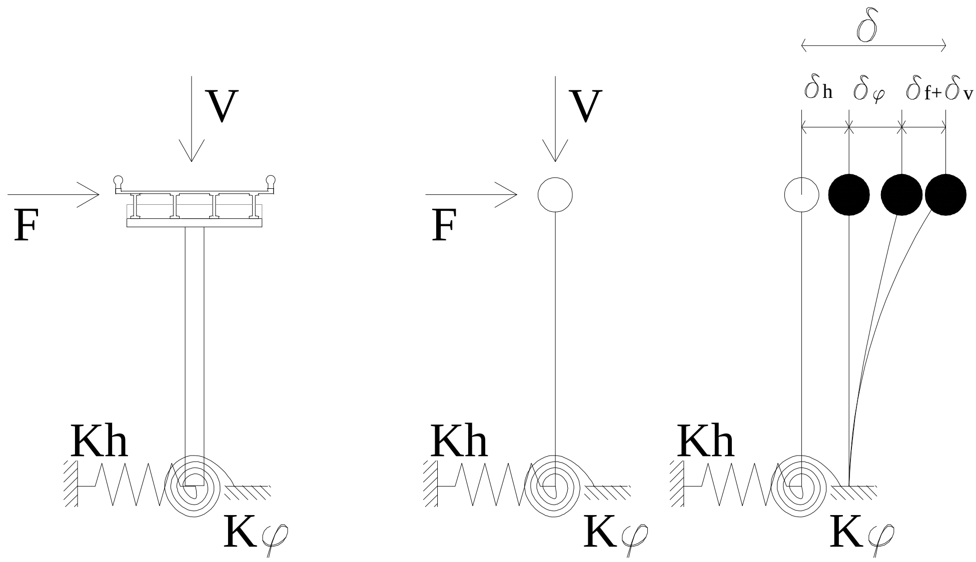


Fig. 5.13 Pier isolated model

Assuming for piers a moment of inertia, we can evaluate the displacement of the effective mass due to the reaching of yield strength at the base by the expressions:

$$\delta_{fy} = \theta_y H \tag{5.24}$$

where:

$$\theta_y = \frac{H\phi_y}{3\nu} \tag{5.25}$$

and $\nu = 1{,}2$ is a factor which takes into account the increased stiffness of the part the pier who is not cracked.

$$\delta_{fu} = \delta_y + (\phi_u - \phi_y) l_p (H - l_p / 2) \tag{5.26}$$

About the foundations deformation contribute, the displacement due to horizontal stiffness of is equal to:

$$\delta_{y,h} = \frac{F_y}{k_h} = \frac{M_y}{H}\frac{1}{k_{HH}} \tag{5.27}$$

$$\delta_{u,h} = \frac{F_y}{k_h} = \frac{M_u}{H}\frac{1}{k_{HH}} \tag{5.28}$$

The displacement due to the roking stiffness of is equal to:

$$\delta_{y,\varphi} = \frac{M_{ytot} \cdot H}{k_{MM}} \tag{5.29}$$

$$\delta_{u,\varphi} = \frac{M_{utot} \cdot H}{k_{MM}} \tag{5.30}$$

Where M_{ytot} ed M_{utot} are total and yielding and ultimate moment comprehensive of the second order effects.

The displacement due to the shear stiffness is equal to:

$$\delta_{y,v} = \frac{M_y}{GA_{ve}} \tag{5.31}$$

By combinations of the bi-linear diagrams, the seismic force-displacement relationship obtained is also bilinear.

5.2.7. *Cyclic shear resistance after flexural yielding*

Before the individuation of characteristics of single degree of freedom systems, it is required to control the possibility of shear dominated failure after flexural yielding, owing to reduction of the shear resistance, V_R, of the plastic hinge zone due to inelastic cyclic deformations. In design of new structures, a shear failure after flexural yielding, though if it is no so brittle as a shear failure before flexural yielding, is still to be avoided because, if it happens, it take place at a pier deformation less than the flexure- controlled ultimate deformation and hence limits the deformation capacity of the pier.

In the plastic hinge zone the shear resistance V_R decrease after flexural yielding whit increasing cyclic inelastic deformation

$$\mu_{\Delta}^{pl} = \mu_{\Delta} - 1 \tag{5.32}$$

For this purpose μ_{Δ}^{pl} may be calculated as the ratio the plastic part of the chord rotation θ, normalized to the chord rotation at yielding θ_y.

Shear strength is also calculated for each column, according to the model proposed by Biskinis [(Biskinis et al., 2004)], also adopted in EC8 [CEN 2005 (A.15)]. In this model, a degradation of the shear strength with the ductility demand is modeled (units in MN and meters) as the following:

$$V_R = \frac{1}{\gamma_{el}} \left[\begin{array}{l} \frac{h-x}{2 \cdot L_V} \min(N; 0{,}55 \cdot A_c \cdot f_c) + (1 - 0{,}05 \min(5; \mu_{\Delta}^{pl})) \cdot \\ \left[0{,}16 \max(0{,}5; 100\rho_{tot}) \cdot (1 - 0{,}16 \min(5; \frac{L_V}{h})) \sqrt{f_c} A_c + V_W \right] \end{array} \right] \tag{5.33}$$

where:

- γ_{el} in equal to 1,15 for primary seismic elements and 1,0 for secondary seismic elements ;
- h depth cross section (D for circular section);
- x compression zone depth;
- N compressive axial force;
- A_c is the cross-section area, taken as being equal to $b_w d$ for a cross-section with a rectangular web of width (thickness) b_w and structural depth d, or to $\pi_c^2 / 4$ (where $D_c = D - 2c - 2d_{bw}$, is the diameter of the concrete core to the inside of the hoops and d_{bw} the diameter of the transverse reinforcement) for circular sections;
- f_c is the concrete compressive strength;
- ρ_{tot} is the total longitudinal reinforcement ratio;

- V_W is the contribution of transverse reinforcement to shear resistance.

The contribution of transverse reinforcement to shear resistance for circular cross section is equal to:

$$V_W = \frac{\pi}{2}\frac{A_{sw}}{s} f_{yw}(D-2c) \tag{5.34}$$

where:

- D is the diameter of the section;
- A_{sw} Is the cross-sectional area of a circular stirrup;
- s is the centerline spacing for stirrups;
- c is the concrete cover;

For cross sections whit rectangular web of width (thickness) b_w the contribution of transverse reinforcement to shear resistance is equal to:

$$V_W = \rho_w b_w z f_{yw} \tag{5.35}$$

where:

- ρ_w is the transverse reinforcement ratio;
- z is the length of the internal level arm.

For a concrete wall, the shear strength V_R, may not be taken greater than the value corresponding to failure by web crushing, $V_{R,\max}$, which under cyclic loading may be calculated from the following expression (with units: MN and meters) [CEN 2005 (A.15)]

$$\begin{gathered} V_{R,\max} = \frac{0.85\left(1-0.06\min\left(5;\mu_\Delta^{pl}\right)\right)}{\gamma_{el}}\left(1+1.8\min\left(0.15;\frac{N}{A_c f_c}\right)\right)\cdot\ldots \\ \ldots\left(1+0.25\max\left(1.75;100\rho_{tot}\right)\right)\left(1-0.2\min\left(2;\frac{L_V}{h}\right)\right)\sqrt{f_c}\,b_w z \end{gathered} \tag{5.36}$$

Where γ_{el} is equal to 1,15 for primary seismic elements and 1,0 for secondary seismic ones, f_c is in MPa, b_w and z are in meters and $V_{R,\max}$ in MN, and all other variables are as previously defined.

The proposed formulations, relate the shear strength of reinforced concrete elements to the ductility request. This circumstance allows to define the curve who represents the shear strength of the element as a function of the displacement of the control point. It can make a direct verification of the brittle mechanisms by graphical. Is possible to compare the force-displacement relationship with the shear strength under cyclic action loads. This comparison allows to identify the real ductility available. Fig. 5.14 shows the three possible mechanisms of failure:

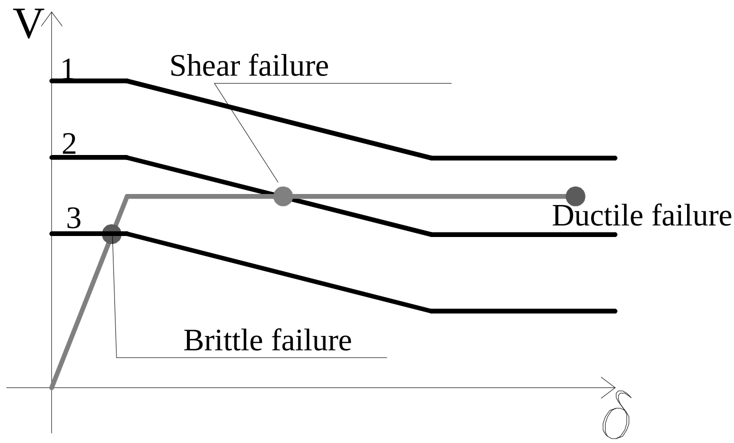


Fig. 5.14 Ductility at failure of columns whit different longitudinal reinforcement ratio

The form of Eq. 5.33 -5.36 are suitable for incorporation in a plastic collapse analysis, since the total shear strength may be expressed in terms of displacement, and compared with the flexural strength-displacement relationship. Fig. 5.14 shows this comparison, the three shear strength-displacement relationship are compared with flexural strength-displacement relationship. For convenience, the flexural moment-curvature relationship are expressed as equivalent shear force-curvature relationship.

Relationship 1, develops a maximum shear force corresponding to full ductile response that is inside the shear strength envelope, and hence shear failure does not occur, and the column fails when the flexural ductility capacity.

Relationship 2, has a shear force corresponding to ideal flexural strength lower than the shear strength envelope. Limited ductile shear failure thus occurs less than the limit to flexural ductility.

Relationship 3 develops a shear force less then at flexural strength and hence a brittle shear failure results

5.2.8. *SDOF system characteristics*

After control of premature brittle failure, the bi-linear force-displacement relationship obtained is used to estimate single degree of freedom characteristics.

The fundamental mode of vibration characteristics can be found for a simple system SDOF after lumped mass and stiffness are known.

In order to take into account different levels of damage related to different limit states, and the resulting difference in terms of dissipative capacity of the structure of the value of the conventional viscous damping is assumed to be equal to 2% for the SLD and 5% for the SLV and the SLC.

Stiffness system can be defined as:

$$k = \frac{V_y}{\delta_y} \tag{5.37}$$

The results of motion equation (Eq. 5.5) represent the circular frequency:

$$\omega = \sqrt{\frac{k}{m}} \tag{5.38}$$

and the fundamental period of vibration of SDOF system can be expressed as:

$$T_i = 2\pi\sqrt{\frac{m_i}{k_i}} \tag{5.39}$$

As already described above for the longitudinal direction two structural models are considered The first consider each pier as independent single simple oscillator as for the transverse direction. The second consider the entire viaduct with mass as the sum of the masses related to the individual piers and stiffness as the sum of the stiffness.

As several times earlier mentioned, in the first case the piers are considered as independent SDOF, so the procedure is completely analogous to that described for the transverse direction.

In the second case, before the bi-linear force-displacement diagram construction, is required to calculate the overall force-displacement relationship. That is the sum of the individual piers force displacement piers.

The bond strength thus obtained is then bi-linear as described in Circolare 617 (2009).

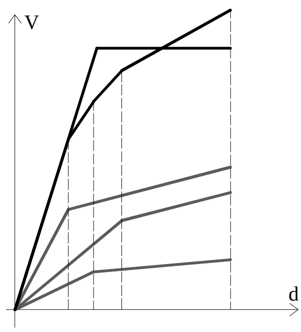


Fig. 5.15 Force displacement bi-linear relationship displacement relative to the longitudinal direction in the overall model

Is possible to characterize oscillator representative of the viaduct in the longitudinal direction.

$$k_{tot} = \frac{V_{ytot}}{\delta_{ytot}} \tag{5.40}$$

$$T = 2\pi\sqrt{\frac{m_{tot}}{k_{tot}}} \tag{5.41}$$

5.3 CAPACITY AND LIMIT STATES

Limit states were introduced in paragraph 2.2.3 were it was suggested that four limit states could be considered for new constructions design. For existing bridges, Limit State on Collapse Prevention, Ultimate State, and Damage are evaluated.

First step in individuation of critical mechanism in structure seismic evolution, is the check to define if structure achieves collapse whit elastic failure, or evolve in plastic range. For Limit state on Collapse Prevention and Limit State of Ultimate State, capacities shall be based on appropriately defined ultimate deformations for ductile elements and on ultimate strengths for brittle ones. For Limit State of Damage, capacity shall be based on yield strengths. The Damage limit state and are calculated only for piers do not exhibit brittle failure in the elastic range. To these piers only limit state of collapse is considered, since a brittle-type fracture.

In this phase of work, the seismic vulnerability analysis capacity thresholds were evaluated locating the plastic hinges at the base of the piers and considering abutment as vertical supports, consistent with the design practice of the time.

The piers have considered as moving independent, and the deck, considered to remain elastic, could move rigidly sliding in the spans. The deck could move in longitudinal direction between physically limits constituted by slab and piercaps. In the transversal direction, deck could move until the stop, if same dispositive exists, or until the loss of support.

All information required to vulnerability check, is deduced by original design, if present, or on results of simulated design. In both case, there are integrated whit Bridge management System results, who can confirm the real structural conditions.

For each Limit State, capacity levels is determined by the characteristics of the SDOF system to vary if the return period of seismic action. It characterizes the seismic input in terms of return period T_{r}, and peak ground acceleration a_g ,which determines the achievement of the limit state considered.

Defined T_r, the return period for which the displacement in the head of the oscillator has to be calculated, is immediately possible to define the parameters of the seismic characteristics for the site in question with reference to the specific return period who determine the capacity threshold achievement.

Displacement at the head of the oscillator evaluation to vary the return period of the seismic action is performed in accordance with what is specified in codes [CEN (2005); Circolare 617 (2009)].

Defined T, m and V, period, mass and yielding strength of the SDOF system, the inelastic displacement demand $d_{\max,i}$ related to the return period $T_{r,i}$ is calculated by different expressions for structures in short period and in the medium-long period ranges. The corner period between short and medium and long period is T_C. The expression should be used are indicated below [CEN (2005)]:

- for $T > T_{C,i}$ the inelastic displacement demand $d_{\max,i}$ is equal to the demand of elastic system of equal period:

$$d_{\max,i} = d_{e,\max,i} = S_{De,i}(T) \tag{5.42}$$

- for $T < T_{C,i}$ the inelastic displacement demand $d_{\max,i}$ is greater than to the demand of elastic system of equal period and is calculated as:

$$d_{\max,i} = \frac{d_{e,\max,i}}{q}\left[1 + (q - 1)\frac{T_{C,i}}{T}\right] \geq d_{e,\max,i} \tag{5.43}$$

$$q = S_e(T) \cdot m / V \tag{5.44}$$

In case where $q \leq 1$ also in this case, the system inelastic demand is assumed to be equal to that of an elastic system of the same period.

The assessment of piers capacity thresholds is performed as previously described for transversal direction for longitudinal direction, in both modeling assumptions. A third case for evaluation of the threshold piers capacity assumed is described above. It performs capacity by combining the both displacement components as follows:

$$\sqrt{\left(\frac{\delta_{T,i}(T_r)}{\delta_{SL,T,i}}\right)^2+\left(\frac{\delta_{L,i}(T_r)}{\delta_{SL,L,i}}\right)^2}\leq 1 \tag{5.45}$$

where:

- $\delta_{T,i}(T_r)$ is the transversal displacement of SDOF system representative of the behavior of the i-th pier in the transverse direction to vary the return period of seismic;
- $\delta_{L,i}(T_r)$ is the longitudinal displacement of SDOF system representative of the behavior of the i-th pier in the transverse direction to vary the return period of seismic;
- $\delta_{SL,T,i}$ is the limit displacement for the particular Limit State in the transversal direction;
- $\delta_{SL,L,i}$ is the limit displacement for the particular Limit State in the longitudinal direction;

So, thresholds capacity are evacuate in following configurations:

- Transversal independent SDOF systems;
- Longitudinal independent SDOF systems;
- Longitudinal total viaduct;
- Longitudinal and transversal combination.

By varying the return period T_r possible to derive demand curves in terms of inelastic displacement of the head of the piers. In this curve, as showed in Fig. 5.16 thresholds capacity for a SDOF system are indicated (black line for damping equal to 5%, black dashed line for damping equal to 2%)

The results thus obtained are summarized in tables which shows the minimum capacity for each configuration analyzed. For each threshold capacity performance is reported the structural element identification for which the threshold is first and the relatives seismic input parameters for which the capacity is achieved.

Following are presented and discussed, all the stress and deformation mechanisms associated with thresholds capacity studied, related to the Limit States considered.

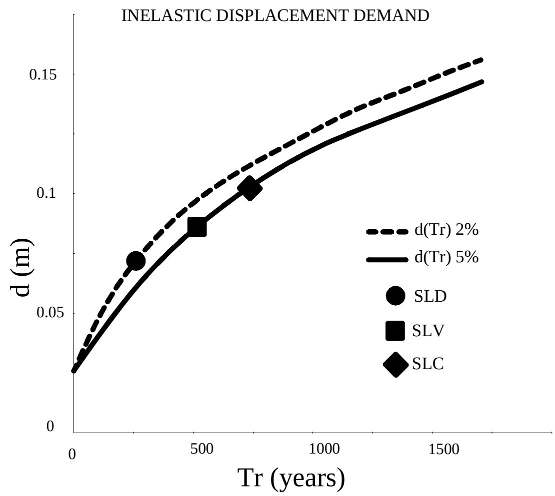


Fig. 5.16 Displacement demand and thresholds capacity for a SDOF system at vary of return period.

5.3.1. ***Limit state of prompt use or Damage (SLD)***

For response to this limit state, the bridge would be expected to be serviceable immediately following an earthquake and should not need repair. Member flexural strengths could be achieved, and some limited ductility developed, provided that concrete spalling in plastic hinges did not occur and that residual crack widths remain sufficiently small so that remedial activity, perhaps in the form of epoxy injection of cracks, was not needed.

Deck performance capacity about the sliding

With reference to the single deck , capacity was evaluated about the sliding. This evaluation was performed both transversal direction, in longitudinal direction, and in longitudinal-transversal combinations. Each evaluation was done by identifying the seismic input, that would result sufficient to overcome the frictional force on the bearings.

$$F_{fric} = W_{deck}\,\mathrm{A}_{\mathrm{bearings}}\,\mu \tag{5.46}$$

$$S_a(T) > F_{fric} \tag{5.47}$$

where F_{fric}, W_{deck} is the weight of deck, beams, and diaphragms, $\mathrm{A}_{\mathrm{bearings}}$ is the surface contact area between bearings and beams, and μ is the frictional coefficient depending of the kind of bearings.

The achievement of this threshold capacity could be associated whit a Operability Limit State (SLO): after an earthquake, it is not subject to significant interruptions in functioning.

Deck performance capacity about the hammering

With reference to the single-deck, capacity was evaluated about the hammering. This evaluation was performed in longitudinal direction. Each evaluation was done by identifying the seismic input, that would result sufficient to produce a elastic displacement who can overcome the distance between consecutive slabs .

$$S_D(T) > D_{slab} \tag{5.48}$$

Achievement of yielding displacement δ_y

With reference to the piers, capacity was evaluated by the achievement of yielding displacement at the top of the piers. This evaluation was performed in longitudinal direction in both cases, in transversal direction, and in longitudinal-transversal combination. Each evaluation was done by identifying the seismic input, that would result sufficient to produce a displacement who can overcome the yielding displacement at the head of the piers.

$$S_D(T) > \delta_y \tag{5.49}$$

5.3.2. ***Limit State for safeguard of human life (SLV)***

The Limit State For Safeguard of Human Life represents the extreme level of seismic response, beyond which it would not be economically and technically feasible to repair the bridge. It is probably the most important in terms of seismic assessment. It is taken to be the limit state beyond which lateral resistance decrease with increasing displacement. in assessing existing bridges, an increased probability of attaining the Limit State for Safeguard of human life, compared with that for new bridges, may be acceptable.

Achievement of displacement δ

With reference to the piers, capacity was evaluated by the achievement of a fixed displacement at the top of the piers. This evaluation was performed in longitudinal direction in both cases, in transversal direction, and in longitudinal-transversal combination. Each evaluation was done by identifying the seismic input, that would result sufficient to produce a rotation who can overcome a 3/4 of ultimate rotation ϑ_u [Circolare 617 (2009)]

$$\delta = \delta_y + \frac{3}{4}\left(\delta_u - \delta_y\right) \tag{5.50}$$

$$S_D(T) > \delta \tag{5.51}$$

5.3.3. ***Survival Limit State for Collapse Prevention (SLC)***

Response to the survival limit state collapse represents the extreme level of seismic response, beyond which collapse would occur. A higher probability of collapse under extreme earthquake intensity should not be accepted for existing bridges compared with new bridges.

Achievement of ultimate displacement δ_u

With reference to the piers, capacity was evaluated by the achievement of ultimate displacement at the top of the piers. This evaluation was performed in longitudinal direction in both cases, in transversal direction, and in longitudinal-transversal combination. Each evaluation was done by identifying the seismic input, that would result sufficient to produce a displacement who can overcome the ultimate displacement at the head of the piers.

$$S_D(T) > \delta_u \tag{5.52}$$

Deck performance capacity about the loss of support L_{supp}

With reference to the individual support, the capacity of the loss of support has been evaluated. This evaluation was performed in longitudinal direction in both cases, in transversal direction, and in longitudinal-transversal combination. The evaluation of the threshold performance was carried out by identifying the seismic input in terms of return period, result sufficient to produce a displacement who can overcome the available length of support.

$$S_D(T) > L_{\text{sup}} \tag{5.53}$$

Assigned a return period of, the maximum displacement undergone by the support was assessed as follows:

$$\delta(T_r) = \delta_{rel,g}(T_r) + \delta_{rel,s}(T_r) \tag{5.54}$$

$\delta_{rel,s}(T_r)$ is the displacement (transversal/longitudinal) relative to each adjacent vertical elements considered to vary the return period of seismic action, evaluated as follows:

$$\delta_{rel,s} = \sqrt{\delta_i^2 - \delta_j^2} \tag{5.55}$$

where:

δ_i is the displacement(transversal/longitudinal) i-th element vertical at the head

δ_j is the displacement (transversal/longitudinal) j-th element vertically adjacent at the element j-th.

$\delta_{rel,g}(T_r)$ is the displacement (transversal/longitudinal) on ground adjacent to the base of the vertical element considered to vary the return period of seismic action.

This component of displacement is measured in accordance with the code [NTC (2008)]:

$$\delta_{rel,g}(T_r) = \delta_{ij}(x) = \delta_{ij0} + \left(\delta_{ij\max} - \delta_{ij0}\right)\left[1 - e^{-1.25\left(x/V_s\right)^{0.7}}\right] \quad (5.56)$$

δ_{ij0} and $\delta_{ij\max}$ are the relative displacement between two points at a small distance (less than 20 m) and maximum displacement displacement between two points i and j characterized by the properties of the respective stratigraphic layers, equal to:

$$\delta_{ij0} = 1.25\left|\delta_{gi} - \delta_{gj}\right| \quad (5.57)$$

$$\delta_{ij\max} = 1.25\sqrt{\delta_{gi}^2 - \delta_{gj}^2} \quad (5.58)$$

and $\delta_{gi}(x)$ and $\delta_{ig}(x)$ are referred at the local characteristics of the soil calculated in accordance at:

$$d_g = 0.0025 a_g S_s S_T T_C T_D \quad (5.59)$$

<u>*Capacity about foundations*</u>

Collapse of foundation is evaluated with reference to the single piers. This evaluation is performed in the transverse direction, in the longitudinal direction and in longitudinal and transversal combination.

Each evaluation was done by identifying the seismic input in terms of return period, that achieve the exceeding of the maximum load capacity of the foundation. The assessment of capacity of the foundation was carried out by assuming two different models of on degree oscillator for piers:

- Elastic SDOF system
- Elasto-plastic SDOF system

In the first case force-displacement relationship indefinitely elastic, whit stiffness equal to the elastic stiffness of the bilinear relationship, is assumed for the piers.

In the second case, however, elastic-perfectly plastic force-displacement relationship is assumed. For the calculation of risk indices, both in terms of PGA and in terms of T_R, we used the elastic-plastic oscillator model, generally consistent with the crisis of the system. The introduction of the elastic model was designed to evaluate thresholds performances of the foundations, whit the aim of increase the yield point whiteout changes stiffness, for piers whit collapse whit shear failure. The results of the model whit elastic SDOF, are useful, in seismic retrofitting point of view.

The ultimate capacity of foundation was evaluated differently for shallow and deep foundation. The horizontal capacity of surface foundation was considered as a limit force due to the friction between soil and foundation. According with the Mohr-Coulomb description, the limit shear strength was expressed as (Viggiani, 1999):

$$T_{\lim} = W \tan \delta \tag{5.60}$$

in which N was the total value of the vertical force and δ is the soil/foundation friction; for the concrete the value of δ was considered equal to the friction angle of the soil ϕ. Due to the massive shape of the bridge foundation, in the evaluation of the lateral resistance the passive earth pressure was considered as not negligible.

For the pile foundation, the horizontal bearing capacity was evaluated using the Broms formulation [Broms (1996a,b)] for a single pile in the hypothesis of piles rigidly jointed in the foundation block. In most of the cases, the horizontal capacity was evaluated for the case of long piles, considering, in failure condition, the formation of two plastic hinges along the pile. The horizontal capacity of the piles group was accounted from the sum of the bearing capacity of each pile, multiplied for an efficiency coefficient of the group $E < 1$.

For the caisson foundation, the horizontal bearing capacity was evaluated similarly to the shallow foundation, accounting both horizontal base resistance and lateral passive resistance.

Each check was done by identifying the seismic input, that would result sufficient to overcome the horizontal capacity, for example:

$$S_a(T) > T_{\lim} \tag{5.61}$$

5.4 RISK INDICATOR EVALUATION

As previously explained the risk indicators have been evaluated in relation of the following limits:

- *Limit state of prompt use or Damage* (α_{SLD});
- *Limit State for safeguard of human life* (α_{SLV});
- *Survival Limit State for Collapse Prevention* (α_{SLC}).

The indices are evaluated by following expressions:

$$\alpha_{SL} = \frac{PGA_{CL}}{PGA_{DL}} \tag{5.62}$$

$$\alpha_{SL} = \left(\frac{T_{R,CL}}{T_{R,DL}}\right)^a \tag{5.63}$$

where:

- PGA_{CL} is the peak ground acceleration at the site in relation to the action who achieve the Limit State;
- PGA_{DL} is the peak ground acceleration expected in the site in relation to the Limit State;
- TR_{CL} is the return time at the site in relation to the action who achieve the Limit State;
- TR_{DL} is the return time expected in the site in relation to the Limit State.

The relationship relative to the return period (Eq. 5.63) returns a scale of risk different to the scale due to the (Eq. 5.64). The difference is due to the shape of the hazard curves. In order to obtain similar scale of risk, we computed return period ratio to the power a high for a coefficient alpha, obtained from statistical hazard curves analysis at the national level.

$$a = \left(\frac{1}{2{,}43} \right) = 0{,}4115 \tag{5.64}$$

5.5 CASE OF STUDY

An example of a seismically vulnerability analysis, conducted for one of the viaduct of the stock, is following presented. For this viaduct, tables of the database for knowledge structure, as presented in Chapter 2, are omitted to ensure the anonymity of the viaduct. However, all the features necessary for work methodology understanding and for performing of final results are summarized and showed in Tab. 5.1. As regards information relative to the mechanical and seismically classification of site, refer to the table showed in paragraph 4.4.2.

Below are listed the summary tables for both the horizontal site acceleration that determine the achievement of the Limit States analyzed, and the indices of risk as defined in Eq. 5.63 and 5.64. They reflect the ratings in the context of this document and they are a guide for the analysis of vulnerability of the stock.

They are presented in order to provide a guide of the work performed and results obtained, and to complete the exposition of the Bridge Management System implemented for this work, regarding sections inherent the vulnerability assessment. For this aims a simple geometry viaduct is choose, whit circular hollow section. Because simple circular sections are used, response in term of force- resisting characteristic is independent of the direction and obtained results are the same in longitudinal and transversal direction.

Results of the entire stock are only briefly discussed in the conclusions, in terms of Risk Index. Single structure are not detailed presented, because they belong to the networks owner [StreGa (2011)].

5.5.1. ***Description of sample: geometry, structural details and mechanical properties of materials***

The structural type is beams simply supported viaduct. Beams are simply supported on cast–in situ R.C. piercap. The deck is reinforced concrete on pre-cast slab. The viaduct, 204 m long and 11.60 m width, consists of three longitudinal pre-cast I beams whit asymmetric current section with asymmetric bulbs. In each of the 6 bays are 4 RC diaphragms, two at the head and two in the span, placed at a constant distance. Piers are single columns whit hollow circular cross section. All bearings are in neoprene pads. Pier cap is devoid of seismically restraint. Foundations are homogeneous, consisting in on piles foundations. Regarding the geometry information, it can be considered complete because original design were founded.

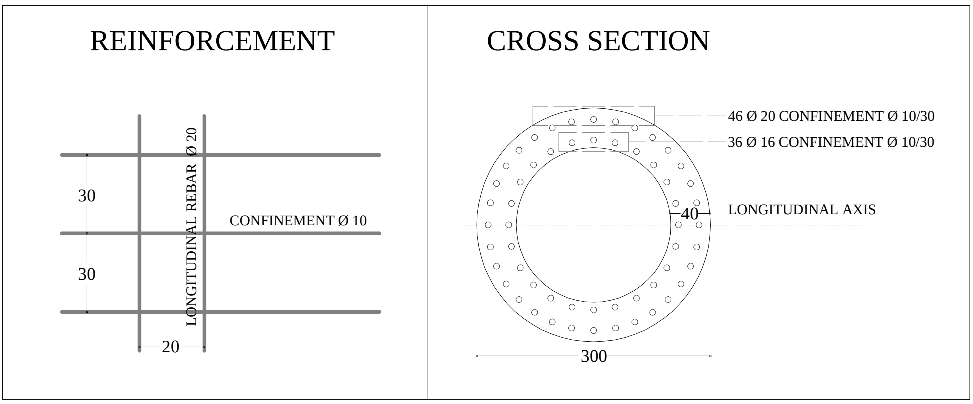


Fig. 5.17 Cross section and structural detail for base piers. Dimensions are in centimeter and millimeter

However, confirm of geometry and construction details shown in the drawings are made in piers where, concrete cores drilling and extraction of rebars are made. After original documentations analysis, simulated design were made to support the in situ surveys planning. The on situ surveys carried out, have confirmed as detailed in the original design, both in terms of longitudinal bars then of the stirrups. Diameter and centerline spacing of bars are confirmed.

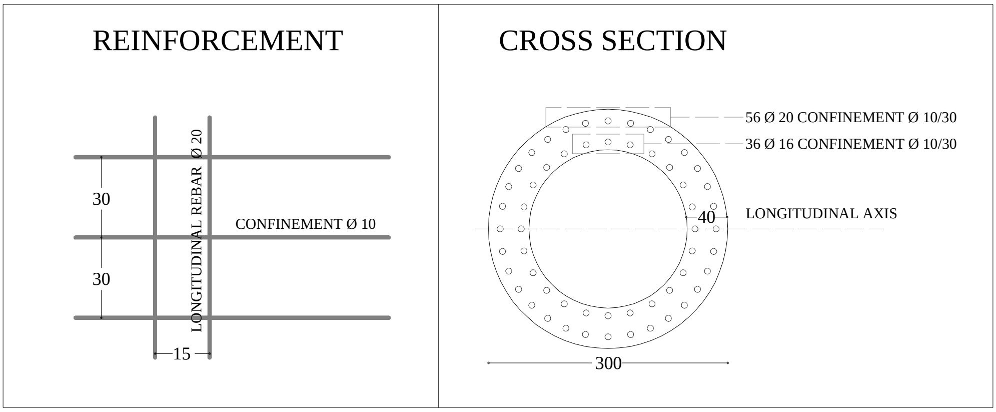


Fig. 5.18 Cross section and structural detail for in river bed piers. Dimensions are in centimeter and millimeter

Having found during the investigation phase the complete reliability of original drawings, it was decided to use information about details for piers in the river bed, not investigable. Structural details are shown in Fig. 5.17 and 5.18.

5.5.2. *Material's mechanical properties.*

Regarding materials originals test certificates for materials are not available. This circumstance require number of concrete and steel samples compatible whit extended in-situ tests. Results of compression tests on concrete specimens and tensile tests on steel bars are shown in Table 5.3 and 5.3.

	HEIGHT	H_p	H_{deck}	R	S	c	f_{cis}	f_y	BEARING
	m	m	m	m	m	m	Mpa	Mpa	
Ab 1									NEOP
Pier 1	9,95	0,9	1,85	1,5	0,4	0,05	30	416	NEOP
Pier 2	12,74	0,9	1,85	1,5	0,4	0,05	35	396	NEOP
Pier 3	9,53	0,9	1,85	1,5	0,4	0,05	30	488	NEOP
Pier 4	7,82	0,9	1,85	1,5	0,4	0,05	30	400	NEOP
Pier 5	5,11	0,9	1,85	1,5	0,4	0,05	30	400	NEOP
Ab 2									NEOP
	EXT. REINF.		INT. REINF.		CONFINEMENT		SEISMICALLY LOADS		
	n	Φ	n	Φ	Φ	s	$N_{basepier}$	N_{deck}	$N_{piercap}$
		mm		mm	mm	m	Kn	Kn	Kn
Pier 1	46	20	36	16	10	0,3	7414	5291	1067
Pier 2	46	20	36	16	10	0,3	7642	5291	1067
Pier 3	46	20	36	16	10	0,3	7380	5291	1067
Pier 4	46	20	36	16	10	0,3	7240	5291	1067
Pier 5	46	20	36	16	10	0,3	7019	5291	1067
BEARINGS						SPANS LENGHT			
	Ldx	Ldx	A	A					
	m	m	mq	mq			m		
Ab 1	0,8	0,71	0,675	0,675			33		
Pier 1	0,8	0,71	0,675	0,675			34		
Pier 2	0,8	0,71	0,675	0,675			34		
Pier 3	0,8	0,71	0,675	0,675			34		
Pier 4	0,8	0,71	0,675	0,675			34		
Pier 5	0,8	0,71	0,675	0,675			34		
Ab 2	0,8	0,71	0,675	0,675			33		

Tab. 5.1 Information's required for seismically vulnerability analysis.

ELEMENT	ID	$f_{car,i}$	Ch/D	C_{dia}	C_a	C_d	f_{cis}
		N/mm2					N/mm2
AB	**S2**	10,71	1,00	1	1	1,2	12,9
PIER 1	**C1**	27,16	1,00	1	1	1,1	30,0
PIER2	**C2**	31,83	1,01	1	1	1,1	35,2

Tab. 5.2 Compressive strength values for concrete cores drilled

ELEMENT	N	EQ. REBAR	LINEAR MASS	fy	fu
		mm	kg/m	N/mm^2	N/mm^2
AB	1	12,3	0,93	419,39	611,29
PIER 1	2	20,1	2,48	415,81	644,60
PIER2	3	20,1	2,48	395,68	613,68
PIER 3	4	20,4	2,56	488,46	753,92

Tab. 5.3 Tensile strength values for steel rebar

5.5.3. ***Moment rotation relationships***

For each pier moment-rotation relationship is presented. In this case, diagrams is the same for both direction, because of the symmetry of the section. Diagrams show the moment-curvature relationships for the section (blue line), the second order effects (green line), the moment-curvature relationship corrected for the effects of second-order (red line) and bi-linear moment-curvature relationship (black dashed line).

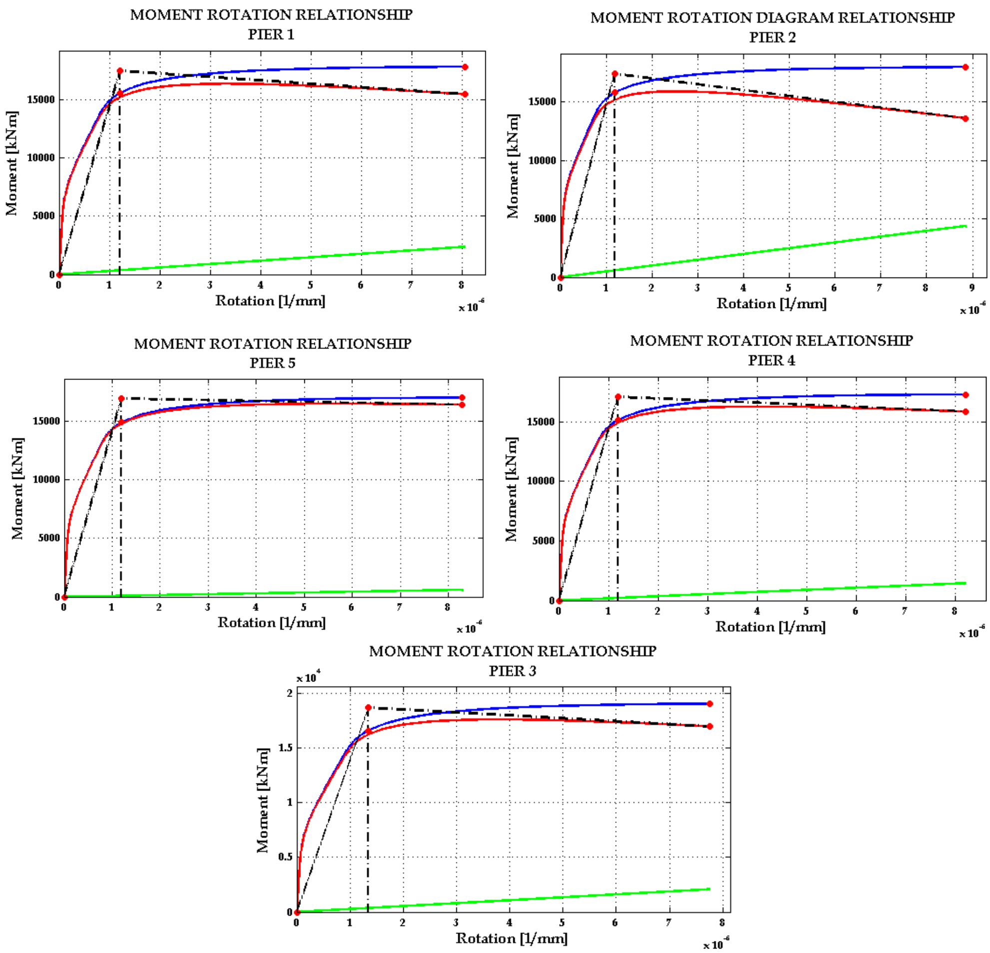


Fig. 5.19 Moment rotation relationship for the bridge piers for longitudinal and transversal directions

5.5.4. *Force-displacement relationships*

For each pier force-displacement relationship is represented. For following pictures, referred to both directions the diagram shows the force displacement relationship for the section (black line) and the shear strength under cyclic action loads expressed in terms of displacement relationship (blue line). In Fig. 5.21 relationship for entire bridge in longitudinal direction is shown.

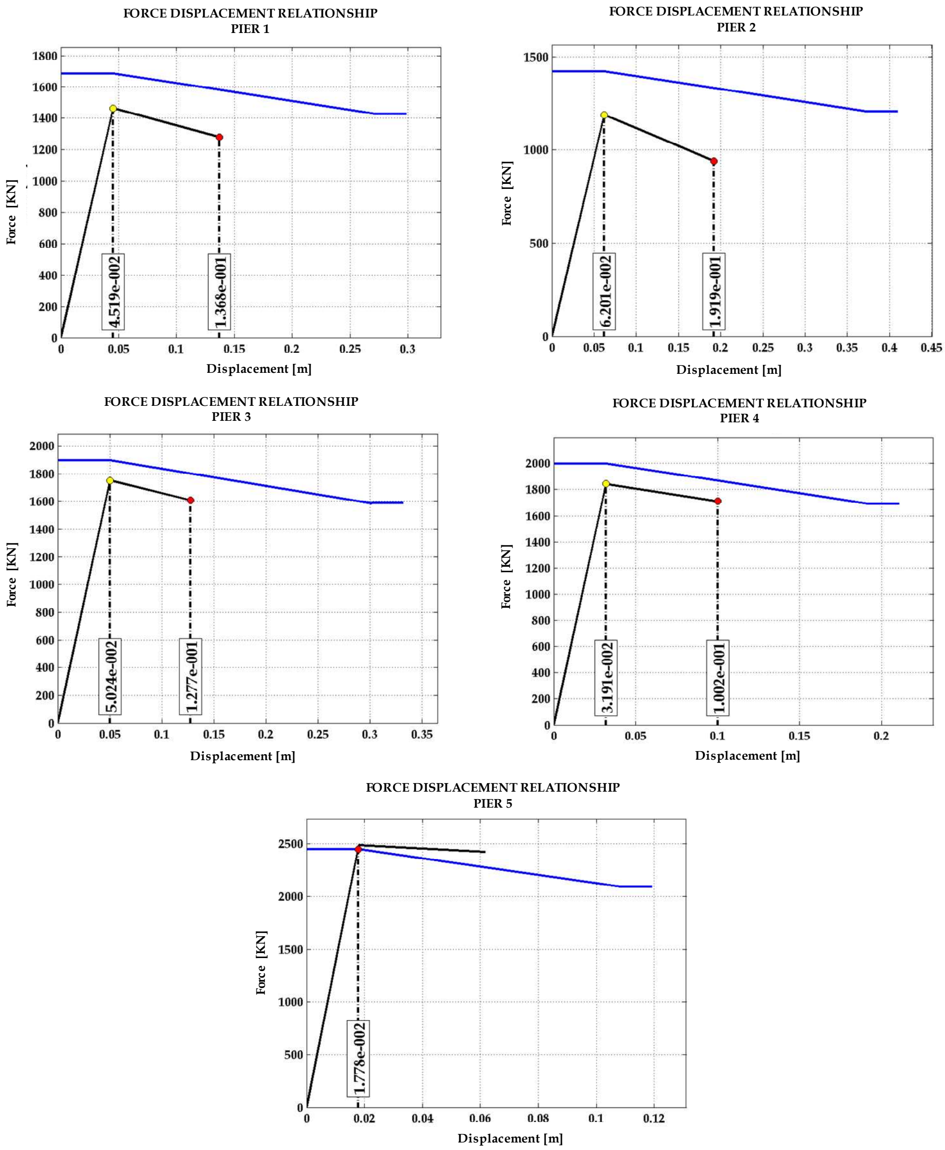


Fig. 5.20 Force displacement relationship for the bridge piers for longitudinal and transversal directions

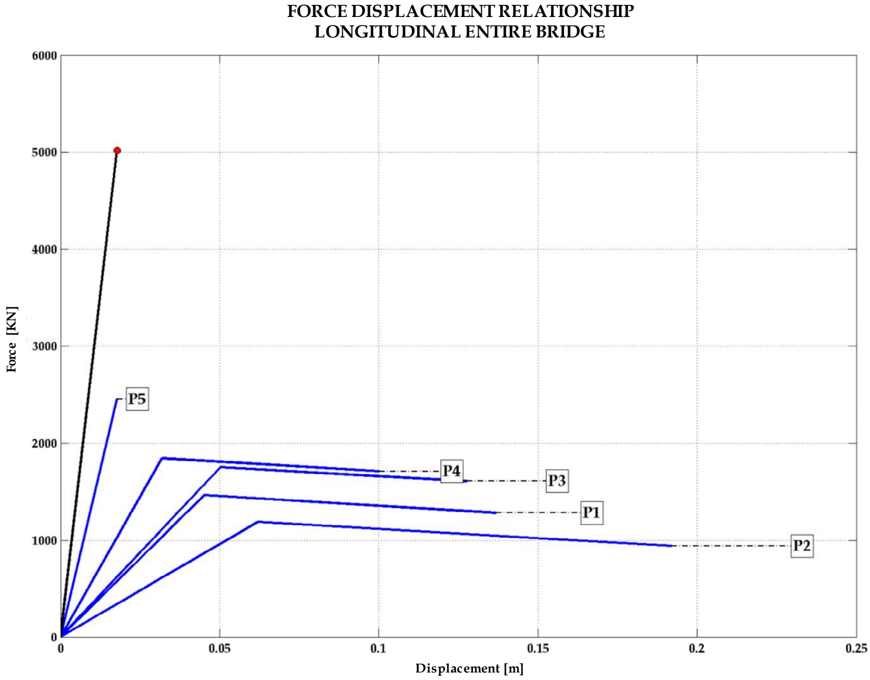


Fig. 5.21 Force displacement relationship for the entire bridge in longitudinal direction

5.5.5. *Capacity threshold*

Following table (Tab. 5.4) show the analysis result for each mechanism of failure analyzed, for all limit states. Values of return time for which the limit state is achieved are associated to each pier. The minimum value, who correspond at the achievement of the limit state for entire bridge, is highlighted by exposition of the peak ground acceleration (a_g), local amplification factor (F_0) and control period (T_C). Curves of capacities are shown in Fig. 5.22-5.24. For best comprehensions, capacity results are summarized in Tables 5.5-5.6.

LONGITUDINAL CAPACITY DECK SLIDING					
Pier	Tr _sn	Tr _dx	a_g	F_0	T_C*
	[years]	[years]	[g]		[s]
A1	-	81			
P1	42	41			
P2	65	65			
P3	39	39			
P4	26	26			
P5	13	13	0,0364	2,4198	0,2457
A2	-	81			

Tab. 5.4 Return period values for threshold capacity related to the deck sliding failure in longitudinal directions

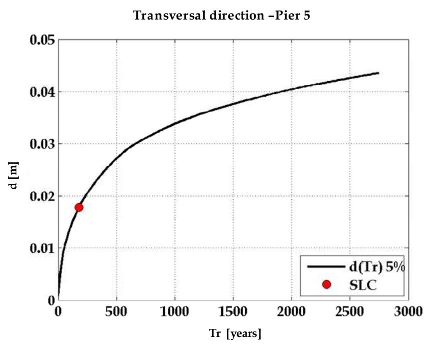


Fig. 5.22 Displacement demand and thresholds capacity for longitudinal direction for entire bridge.

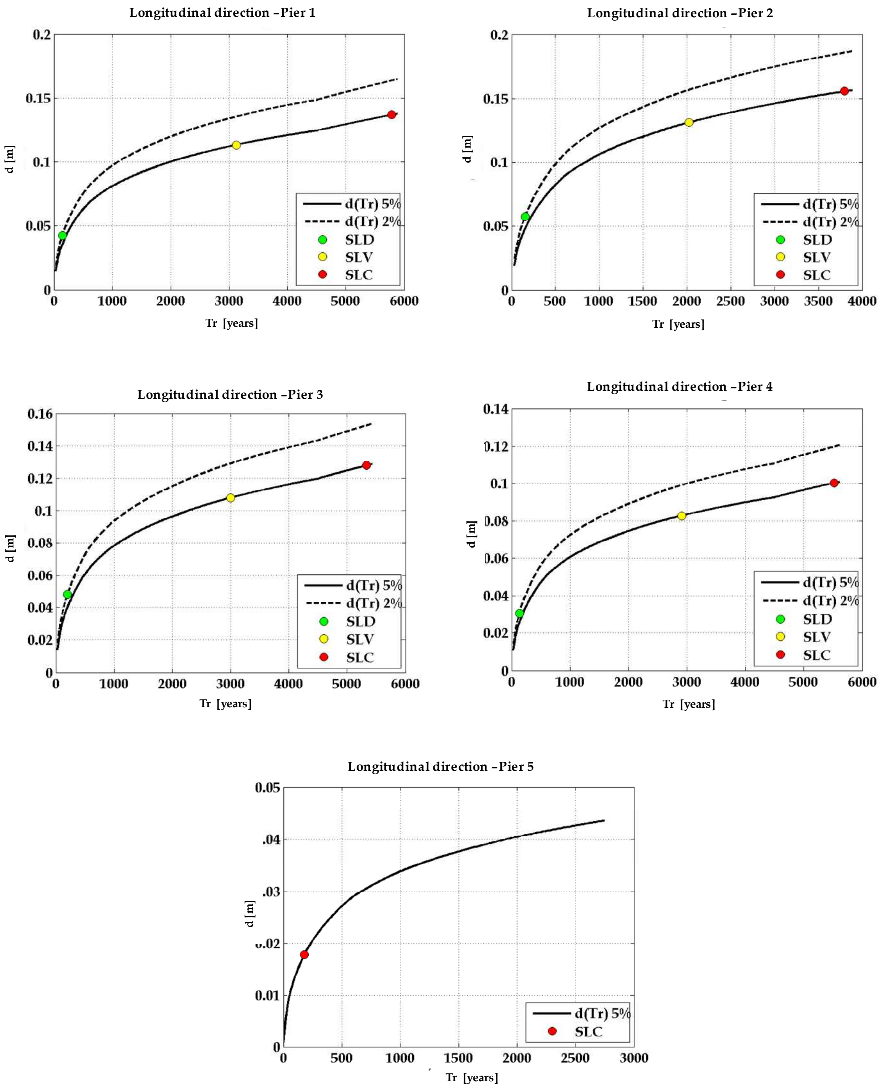


Fig. 5.23 Displacement demand and thresholds capacity for a each piers, in longitudinal direction.

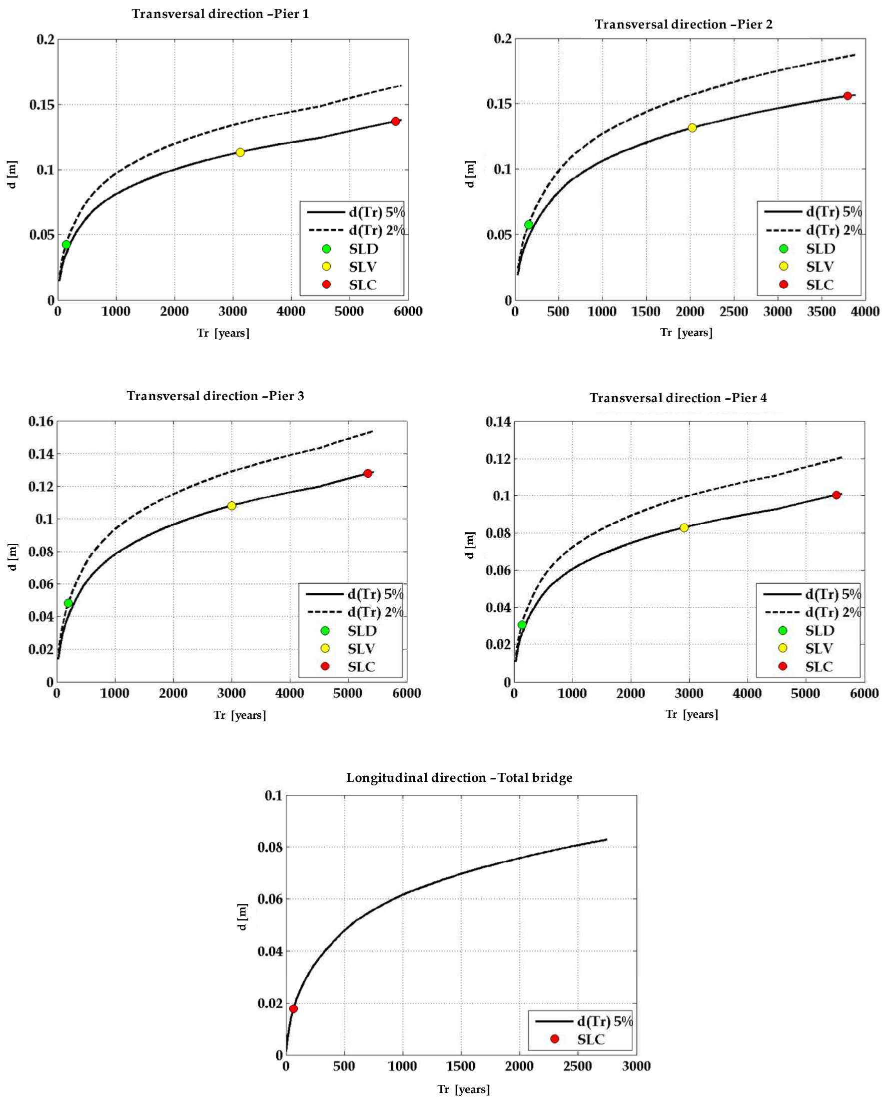


Fig. 5.24 Displacement demand and thresholds capacity for a each piers, in transversal directions.

LONGITUDINAL CAPACITY					
Pier	TrSLD	TrSLV	TrSLC		
	[years]	[years]	[years]		
P1	143	3121	5786		
P2	154	2024	3799		
P3	200	2997	5333		
P4	135	2908	5517		
P5	-	-	179		
	Pier	Tr	a_g	F_0	T_C*
	[years]	[years]	[g]		[s]
SLD	P4	135	0,1084	2,4639	0,3462
SLV	P2	2024	0,3066	2,4816	0,3979
SLC	P5	179	0,1225	2,4883	0,3497

TRANSVERSAL CAPACITY					
Pier	TrSLD	TrSLV	TrSLC		
	[years]	[years]	[years]		
P1	143	3121	5786		
P2	154	2024	3799		
P3	200	2997	5333		
P4	135	2908	5517		
P5	-	-	179		
	Pier	Tr	a_g	F_0	T_C*
	[years]	[years]	[g]		[s]
SLD	P4	135	0,1084	2,4639	0,3462
SLV	P2	2024	0,3066	2,4816	0,3979
SLC	P5	179	0,1225	2,4883	0,3497

LONGITUDINAL CAPACITY TOTAL VIADUCT					
	TrSLD	TrSLV	TrSLC		
	[years]	[years]	[years]		
	-	-	65		
	Pier	Tr	a_g	F_0	T_C*
	[years]	[years]	[g]		[s]
SLD		-			
SLV		-			
SLC		65	0,0765	2,4444	0,3346

Tab. 5.5 Capacities for longitudinal, transversal and longitudinal total viaduct directions

LONGITUDINAL-TRANSVERSAL COMBINATION CAPACITY					
Pier	TrSLD	TrSLV	TrSLC		
	[years]	[years]	[years]		
P1	73	955	1781		
P2	78	674	1126		
P3	99	925	1603		
P4	69	903	1674		
P5	-	-	87		
	Pier	Tr	a_g	F_0	T_C*
	[years]	[years]	[g]		[s]
SLD	P4	69	0.0787	2,449	0,3374
SLV	P2	674	0.2065	2,5136	0,3745
SLC	P5	87	0.0878	2,4548	0,3426

Tab. 5.6 Capacities for longitudinal-transversal combination direction

5.5.6. ***Index risk***

Following tables presents the final results, in terms of Index Risk, expresses for return period and PGA.

LONGITUDINAL DECK SLIDING INDEX RISK						
	Pier	Tr	a_g	F_0	T_C*	PGA
		[years]	[g]		[s]	[m/s²]
Capacity	P5	13	0,0364	2,4198	0,2457	0,429
Domand	P5	120	0,1025	2,4613	0,3459	1,2065
Index		0,3976				0,3555
TRANSVERSAL DECK SLIDING INDEX RISK						
	Pier	Tr	a_g	F_0	T_C*	PGA
		[years]	[g]		[s]	[m/s²]
Capacity	P5	13	0,0364	2,4198	0,2457	0,429
Domand	P5	120	0,1025	2,4613	0,3459	1,2065
Index		0,3976				0,3555
LONGITUDINAL-TRANSVERSAL COMBINATION DECK SLIDING INDEX RISK						
	Tr (Mcr)	Tr	a_g	F_0	T_C*	PGA
		[years]	[g]		[s]	[m/s²]
Capacity	P5	7	0,0283	2,4171	0,2196	0,3327
Domand	P5	120	0,1025	2,4613	0,3459	1,2065
Index		0,3076				0,2757
LONGITUDINAL YIELDING DISPLACEMENT INDEX RISK						
	Pier	Tr	a_g	F_0	T_C*	PGA
		[years]	[g]		[s]	[m/s²]
Capacity	P4	135	0,1084	2,4639	0,3462	1,276
Domand	P4	201	0,1285	2,4992	0,3513	1,5131
Index		0,8478				0,8433

TRANSVERSAL YIELDING DISPLACEMENT INDEX RISK						
	Pier	Tr	a_g	F_0	T_C*	PGA
		[years]	[g]		[s]	[m/s²]
Capacity	P4	135	0,1084	2,4639	0,3462	1,276
Domand	P4	201	0,1285	2,4992	0,3513	1,5131
Index		0,8478				0,8433
LONGITUDINAL- TRANSVERSAL COMBINATION YIELDING DISPLACEMENT INDEX RISK						
	Pier	Tr	a_g	F_0	T_C*	PGA
		[years]	[g]		[s]	[m/s²]
Capacity	P4	69	0,0787	2,449	0,3374	0,9268
Domand	P4	201	0,1285	2,4992	0,3513	1,5131
Index		0,6417				0,6126
LONGITUDINAL DECK HAMMERING INDEX RISK						
	Pier	Tr	a_g	F_0	T_C*	PGA
		[years]	[g]		[s]	[m/s²]
Capacity	P3	86	0,0875	2,4547	0,3424	1,0305
Domand	P3	201	0,1285	2,4992	0,3513	1,5131
Index		0,7031				0,6811
LONGITUDINAL PIER DISPLACEMENT INDEX RISK						
	Pier	Tr	a_g	F_0	T_C*	PGA
		[years]	[g]		[s]	[m/s²]
Capacity	P2	2024	0,3066	2,4816	0,3979	3,2955
Domand	P2	1898	0,2998	2,4834	0,3966	3,2412
Index		1,027				1,0167
TRANSVERSAL PIER DISPLACEMENT INDEX RISK						
	Pier	Tr	a_g	F_0	T_C*	PGA
		[years]	[g]		[s]	[m/s²]
Capacity	P2	2024	0,3066	2,4816	0,3979	3,2955
Domand	P2	1898	0,2998	2,4834	0,3966	3,2412
Index		1,027				1,0167
LONGITUDINAL-TRANSVERSAL COMBINATION PIER DISPLACEMENT INDEX RISK						
	Pier	Tr	a_g	F_0	T_C*	PGA
		[years]	[g]		[s]	[m/s²]
Capacity	P2	674	0,2065	2,5136	0,3745	2,4152
Domand	P2	1898	0,2998	2,4834	0,3966	3,2412
Index		0,6508				0,7451
LONGITUDINAL PIER ULTIMATE DISPLACEMENT INDEX RISK						
	Pier	Tr	a_g	F_0	T_C*	PGA
		[years]	[g]		[s]	[m/s²]
Capacity	P5	179	0,1225	2,4883	0,3497	1,442
Domand	P5	2475	0,329	2,4761	0,4021	3,467
Index		0,3363				0,4159
TRANSVERSAL PIER ULTIMATE DISPLACEMENT INDEX RISK						
	Pier	Tr	a_g	F_0	T_C*	PGA
		[years]	[g]		[s]	[m/s²]
Capacity	P5	179	0,1225	2,4883	0,3497	1,442
Domand	P5	2475	0,329	2,4761	0,4021	3,467
Index		0,3363				0,4159

LONGITUDINAL ENTIRE BRIDGE ULTIMATE DISPLACEMENT INDEX RISK						
	Pier	Tr	a_g	F_0	T_C*	PGA
		[years]	[g]		[s]	[m/s²]
Capacity	-	65	0,0765	2,4444	0,3346	0,9001
Domand	-	2475	0,329	2,4761	0,4021	3,467
Index		0,2209				0,2596
LONGITUDINAL-TRANSVERSAL COMBIANTION ULTIMATE DISPLACEMENT INDEX RISK						
	Pier	Tr	a_g	F_0	T_C*	PGA
		[years]	[g]		[s]	[m/s²]
Capacity	P5	87	0,0878	2,4548	0,3426	1,0336
Domand	P5	2475	0,329	2,4761	0,4021	3,467
Index		0,2493				0,2981
LONGITUDINAL LOSS SUPPORT INDEX RISK						
	Pier	Tr	a_g	F_0	T_C*	PGA
		[years]	[g]		[s]	[m/s²]
Capacity	P3	28450	0,7755	2,4106	0,4565	7,6078
Domand	P3	2475	0,329	2,4761	0,4021	3,467
Index		2,7545				2,1944
TRANSVERSAL LOSS SUPPORT INDEX RISK						
	Pier	Tr	a_g	F_0	T_C*	PGA
		[years]	[g]		[s]	[m/s²]
Capacity	P3	49726	0,9435	2,3959	0,4699	9,2557
Domand	P3	2475	0,329	2,4761	0,4021	3,467
Index		3,4727				2,6696
LONGITUDINAL-TRANSVERSAL COMBINATION LOSS SUPPORT INDEX RISK						
	Pier	Tr	a_g	F_0	T_C*	PGA
		[years]	[g]		[s]	[m/s²]
Capacity	P3	16805	0,6446	2,4246	0,4442	6,3239
Domand	P3	2475	0,329	2,4761	0,4021	3,467
Index		2,214				1,824
LONGITUDINAL FOUNDATIONS INDEX (ELASTIC SDOF)						
	Pier	Tr	a_g	F_0	T_C*	PGA
		[years]	[g]		[s]	[m/s²]
Capacity	P5	257	0,1416	2,5065	0,3555	1,6673
Domand	P5	2475	0,329	2,4761	0,4021	3,467
Index		0,3907				0,4809
LONGITUDINAL FOUNDATIONS INDEX (ELASTO-PLASTIC SDOF)						
	Pier	Tr	a_g	F_0	T_C*	PGA
		[years]	[g]		[s]	[m/s²]
Capacity	P5	257	0,1416	2,5065	0,3555	1,6673
Domand	P5	2475	0,329	2,4761	0,4021	3,467
Index		0,3907				0,4809
LONGITUDINAL-TRANSVERSAL COMBINATION FOUNDATIONS INDEX (ELASTIC SDOF)						
	Pier	Tr	a_g	F_0	T_C*	PGA
		[years]	[g]		[s]	[m/s²]
Capacity	P5	118	0,1017	2,4609	0,3459	1,1976
Domand	P5	2475	0,329	2,4761	0,4021	3,467
Index		0,2829				0,3454

LONGITUDINAL-TRANSVERSAL COMBINATION FOUNDATIONS INDEX (ELASTO-PLASTIC SDOF)						
	Pier	Tr	a_g	F_0	T_C*	PGA
		[years]	[g]		[s]	[m/s²]
Capacity	P5	118	0,1017	2,4609	0,3459	1,1976
Domand	P5	2475	0,329	2,4761	0,4021	3,467
Index		0,2829				0,3454

5.6 CONCLUSIONS

Surveys and assessments carried out in seismic perspective have outlined a setting of significant seismic vulnerability of exanimate infrastructures. A large number of shear failure is founded, who don't allow the evolution of plastic hinge. Considering the Index Risk in terms of PGA (Eq. 5.63), about the premature shear failures, Collapse Prevention Limit State is achieved approximately in the range between 0,1 and 0,3. Always in relation to the Collapse Prevention Limit State, lower Index Risk are referred to the foundations, in a range between 0,15 and 0,4, while the indices due to the cord rotation, related to the displacement ductility, are significantly higher, between 0,5 and 0,7. Only one viaduct achieve the collapse for the loss of support (Fig. 5.25).

Similar results are founded with regard to the Ultimate Limit State.

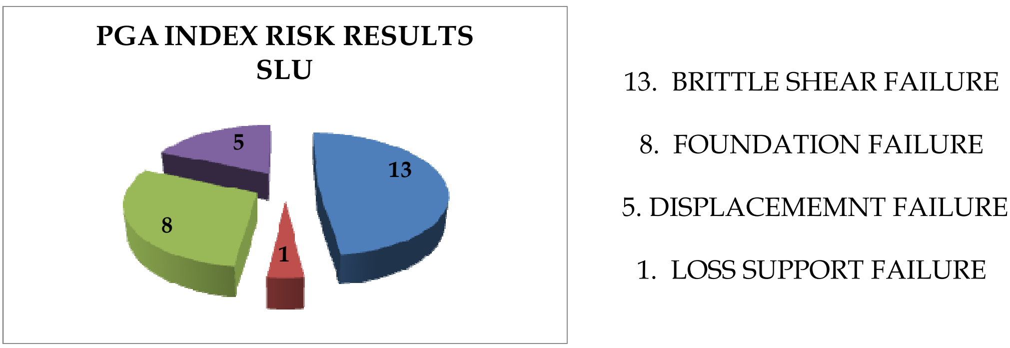


Fig. 5.25 Index risk results for Limit State of Collapse prevention

For Damage Limit State, threshold capacities are achieved, equal both for the deck hammering, and for the yielding displacement. Risk indices, expressed in terms of PGA, are amounted to 0,5 and 0,6. For exception of a few cases, for which, because wrong technological details, Damage Limit State is reaches by the deck hammering for which Index Risk attested between 0,1 and 0,3.

This scenario of results is due to the codes of original design of structure, obsolete or in complete absence of specific seismic requirements. The lack of attention to construction details, the absence of the performance based design indications, inadequate ductility capacity of columns, outlined scenarios of performance lower that required by current regulations. They also highlighted significant deficiencies in mechanical materials properties, in particular for strength concrete, and in structural details, as concrete cover and centerline spacing rebars. Deficiencies are amplified by the state of preservation of the structures, who is not optimal for many structural elements, as piercaps, bearings, beams heads, slabs. Particular attention deserves the mechanisms of brittle failures, which are certainly enhanced by spread of poor or incorrect construction details.

This conditions, together with the level of seismic hazard of the territory on which the infrastructural assets are located, could suggests the opportunity to modulate the increase of the resistance and ductility of the critical components also by using external dampers.

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Chapter 6

CONLUSIONS

The work carried out has shown the complexity and interdisciplinary related to the analysis of seismic vulnerability of distributed infrastructures. The interconnection between different topics, such as geology, seismology engineering, emerges in the first stage of work, the knowledge. The level of knowledge, not only closely related to the structure features, [NTC (2008)], but also to the site characteristics, for viaducts located outside the Urban Areas in complex geological conditions, and to the conditions of maintenance due to the continuous expositions at atmospheric agents and cyclic loading is the base for the choice of modeling.

The accuracy of analysis is closely related to the knowledge background. Complex mathematically models, could be invalidate by unknown boundary conditions, geometric data and material properties that can only be roughly estimated, as well as seismic forces are not known with precision. In reality, the unknown nature of the seismic event, uncertainties in material properties, and unknown boundary conditions, among other imponderables, do not support such an approach but suggest instead that a design process which deals iteratively with all these uncertainties rather than deterministic mathematical models and analyses needs to be the driver.

Assessment the knowledge of the structure is required. If the structure was built before the Seventies original design drawings can be hardly found. This is even more difficult in the case of infrastructures managed by a few administrations at the regional scale, where the organization of the archives is usually very complex. A previously presented, also in-situ survey could be difficult, and in some case not possible.

Moreover structural characterization in view of quantitative assessment of real performances has be known. A careful analysis of conservation state of and degradation of the structure, allows two goals. The first is to ensure that they are not workings special phenomena that may compromise the structural safety of the work, and for which the seismic capacity of the structure can be greatly cut down, the second is related to the observation of the conservation status structure, and quantification of degradation/corrosion due to natural structure working.

A procedure able to provide quantitative comparative data for networks at regional scale has been presented.

The result of the work, is the implementation of global Bridge Management System, useful for global assessment at single structure level, and sufficiently detailed for administration of medium-sized populations, such for example a regional scale. The different sections of BMS, allows the collection and cataloguing of background data and knowledge data, and constitute, the base for performance evaluations in structural perspective, linked to the road network exercise, and in seismic perspective, for seismic vulnerability assessment. A guide for the maintenance program and seismic retrofit is provided, useful for by responsible agencies.

In seismic perspective, the analysis used is pushover analysis, carried out on independent stand-alone frames considered to be completely separated from adjacent frames at the movement joints. In these analyses the superstructures is considered to be effectively rigid in the horizontal plane.

The force-based design approach, is used, considering the plastic hinges localized at the base of the piers, and the abutment as vertical supports. For each pier, modeling as a SDOF system, the displacement at the top is calculated. Displacement is incremented, tracking the formation of plastic hinges, materials non linearity, and deformation of foundations. As explained, geometrical considerations, including the effects of foundation flexibility, influence the relationship between structural displacement ductility factor and member ductility factor, which may be expressed in curvature, rotation, or displacement units.

Serviceability and ultimate limit states are related to inelastic rotations of plastic hinges, and other resistance criteria.

Different threshold criteria for reaching Limit States, at single structure level, are the expression of the vulnerability of the system. At the network level, however, provide a guide for maintenance, structural reinforcement, and seismic retrofitting planning. In terms of road network, the first goal who as to be achieved is the overall improvement of the road performances, by the leveling of Index Risk for each viaduct, at the average of the Indices of the entire network.

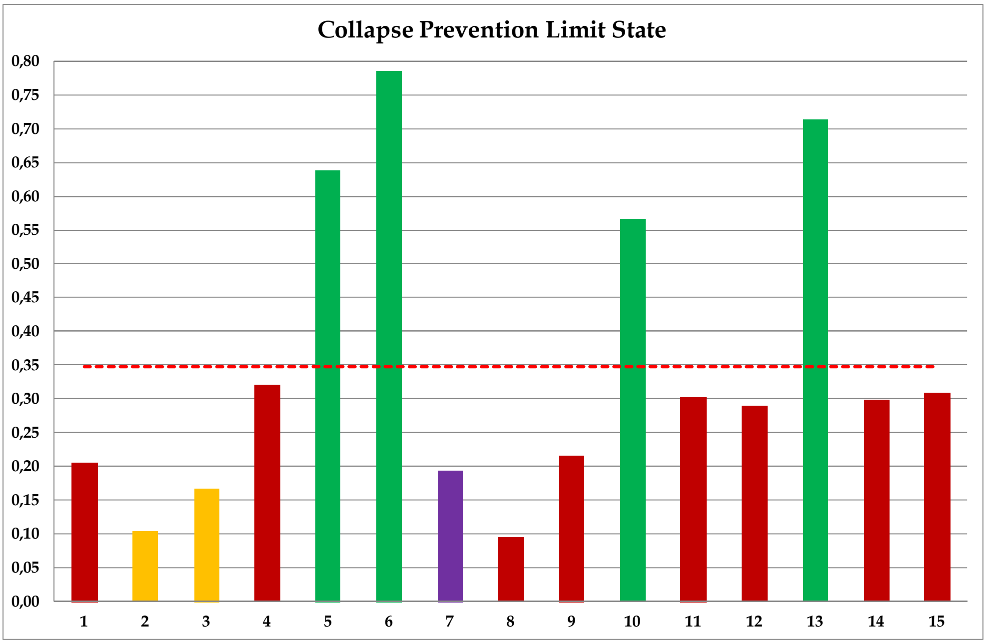

Fig. 6.1 Index risk results for Prevention Collapse Limit State for bridges in one network analyzed. Red index are referred to brittle shear failure, yellow index to a collapse of foundations, green index to a achievement of the ultimate displacement, violet index to a loss of support.

In terms of road network, the first goal who as to be achieved is the overall improvement of the road performances, by the leveling of Index Risk for each viaduct, at the average of the Indices of the entire network (Fig. 6.1).

In this regard, knowledge of performance for each failure mechanisms, supported by results of surveys for the status of preservation assessment, allowing a detailed and rational planning of preventions measures. The controlled increasing of the threshold level of capacities, possible by knowledge of each failure level, allow the management on time, the delay of economical resources, and the knowledge continue of the status of degree as well as the static and seismically performance conditions. Results about the entire stock are not detailed presented, because they are resumed in a private communication. [StreGa (2011)].

6.1.1. *Perspectives*

As future perspective two objective could be in progress pursuit. At single structure level, model of complete bridge systems can be performed, and analyze the dynamic time- history response to incoherent input ground motions along the length of the bridge, considering both material and geometric nonlinear effects. In fact, for long bridges with several movement joints, for which pier are located in different ground conditions, it is improbable that the excitation at all soil-structure boundaries will be coherent and synchronous [Ioannis F. et al. (2011)]. Problems still needs best investigated in non linear time histories analysis are, among others: movement joint characterization, dynamic allowance for soil structure interactions, fully cyclic (hysteretic) characteristics and damping, deformations in joints and connection regions.

Sensitivity analysis for estimate the influence of mechanical materials properties in fragility curves [Pinto et al. (2004)], particularly referred to the concrete strength could be useful, for in situ survey planning, for determinations of percentage of element to be checked.

At network level, catalogue of possible retrofitting, specifics for controlled increasing threshold capacities, could be provided and, whit the aim to intersection of structural conditions and seismic performance, the definition of decision makers useful to reduce performance levels of the structure, related of the effective status of preservation, could be obtained.

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