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"MODEL UPDATING OF ANCIENT HERITAGE STRUCTURES: THE RIALTO BRIDGE CASE"

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Figure Index

Table Index

Introduction

The subject of this thesis is the evaluation of the dynamic and seismic behaviour of the Rialto Bridge, located in Venice; this structure crosses the Grand Canal and is one of the major attractions of the city and a UNESCO World Heritage Site.
According to Article 9 [see Articles 33, 34] of the Italian Constitution "The Republic promotes the development of culture and scientific and technological research. Protecting the landscape and the historical and artistic heritage of the nation", the Code of Cultural and Landscape Heritage has established the guiding concepts for the activities of knowledge and conservation of structures of historical and artistic importance [1], while the Directive of 9 February 2011 has established the rules for the assessment and reduction of seismic risk of the Italian cultural heritage.
The pursuit of these objectives has led to the development of a permanent monitoring network in Italy, founded in 1995, and managed by the Seismic Observatory of Structures (OSS) with the aim of assessing sudden or progressive damage to structures, both during exceptional events, such as earthquakes, and under normal operating conditions, possibly in order to design effective seismic improvement or structural reinforcement.
In the last few decades a considerable development has been observed in the field of monitoring of civil and infrastructural engineering structures, due to the high seismicity of the Italian territory and to the fact that the Italian architectural heritage consists mainly of historic buildings that are now centuries old, therefore the deterioration of the state of conservation, both of materials and components, has repercussions on their overall performance.
Moreover, the construction techniques used in the past to design and build these buildings are now inadequate, considering the numerous seismic events that have occurred in the last forty years in Italy, which have dramatically highlighted the extreme seismic vulnerability of our building heritage.
In particular, the seismic events of the last twenty years have demonstrated the need to ensure more effective protection of existing buildings in order to mitigate, above all, human losses.
Historical or strategic buildings and infrastructure require costly safety, maintenance or reconstruction, so the availability of a permanent assessment of structural conditions is essential to ensure a consistent level of reliability and safety.

Over the last three decades, in addition to traditional visual inspection methods, new procedures have been developed based on data collection and analysis by the scientific community, professional associations and the Network of University Seismic Engineering Laboratories (ReLUIS), the latter promoted by the Department of Civil Protection (DPC).

These procedures carry out a periodic diagnostic process on the structures measuring the symptomatic characteristics of the phenomena under examination and conduct their analysis to determine the current state of the system. This requires the integrated use of different sensors, devices and auxiliary tools that measure the structural response to environmental stresses.

A database of measurements is then created for the structures studied and the data collected are processed to identify any anomalies and to identify the modal parameters of the structures under operating conditions.

The interest in these parameters derives from the observation that the dynamic behavior of the structure depends on its intrinsic characteristics, such as masses, rigidity and degree of constraint related to boundary conditions.

From an operational point of view, for the case in question, investigations were first carried out to identify the properties of the constituent materials, in fact, in the study of historical masonry buildings, the main difficulty is represented by the lack of knowledge of the mechanical characteristics of the masonry due to the heterogeneity of the quality of the material, the different textures of the walls and the wide variety of construction elements.

The data collected by the accelerometric sensors installed on the bridge were used to evaluate the dynamic response of the bridge to environmental vibrations, so the modal forms and corresponding modal frequencies of the two structural subparts were extracted: the arch of the bridge and the superstructure used to house the shops.

Subsequently, it was necessary to create a finite element numerical model, a phase that involved the use of RhinoCeros and Ansys Mechanical Apdl software. The construction of the model was used to perform the modal analysis of the system and allow the calibration of the parameters, whose initial values were modified and adapted in the second phase of the procedure, called modal update phase, useful to create the best correlation between numerical and measured data.

Chapter 1

1 Structures Monitoring

1.1 Structural damage concept

A structural damage consists in a change in the characteristics of the system that leads to a worsening of its current and future performance. Therefore, the identification of a damage requires a comparison between two different states of the system, one of which represents the initial undamaged state, while the second is indicative of the characteristics and actual behavior.

The possible variations of the system concern the variations of the mechanical characteristics of the materials, the variations of the geometrical properties or of the boundary conditions.

All constructions are subject to a physiological variation of the properties of the constituent materials, as a result of physical and anthropic actions, in particular the latter are often responsible for phenomena of fatigue degradation.

Damage, understood as deviation from the ideal characteristics of materials or structural elements, is always present, albeit at different levels, and is due to intrinsic defects in materials or construction errors, which increase when the system is subjected to operational loads.

However, this term does not necessarily mean a total loss of functionality of the structure but, rather, that the system does not work optimally.

Over time, and therefore as the number of load cycles on the construction increases, the effects of the damage will no longer be acceptable to the users of the structure, and will have to be restored, restored or demolished.

The damage can accumulate over long periods of time, as in cases associated with corrosion or fatigue, or it can occur over short periods of time, as in the case of earthquakes.

The process of identifying damage in aerospace, mechanical and civil engineering is called Structural Health Monitoring (SHM)

1.2 Structural Health Monitoring (SHM)

Structural Health Monitoring (SHM) is the process aimed at identifying structural damage and is applied in various sectors of aerospace, mechanical and civil engineering.
This process is based on the observation of structures through the periodic collection of data that allow to evaluate the variability of the characteristics of the system; these, in fact, should be sensitive to damage and through a statistical analysis the method establishes the current state of health of the structure.
The results collected during the analysis campaigns are periodically updated in order to obtain indications on the remaining time of the structure's ability to perform the established function, considering phenomena of inevitable aging. Structural monitoring is also used to verify the state of the structure in case of exceptional events such as earthquakes or explosions.
This monitoring ensures reliable information on the performance of the system during these extreme events and can be a useful tool to have indications on its integrity even after the event.
A real-time automatic monitoring system is more economical than choosing to carry out periodic surveys. Moreover, in the second case, the visual checks must be carried out by highly specialized personnel with an occurrence that does not depend on the real state of the structure.
In addition, to carry out the inspections, expensive service equipment is required and the complete closure or at least partial limitation of the normal activities that take place in the structure under investigation.
A permanent monitoring system is much more cost-effective if one considers the amortisation of initial costs over a rather long period, costs related to the design, design and implementation of the system.

1.2.1 Experimental Technologies of SHM

Experimental structural monitoring technologies are classified according to the nature of the tests performed, which may be static or dynamic.

Static tests can be performed with non-destructive or destructive tests: in the first case, the response of a limited portion of the structure under controlled load is measured to obtain a mechanical characterization of the materials.
In the latter case, the samples collected are broken in the laboratory or in situ to be characterised. These latter tests are expensive and their results are difficult to generalize: this method is usually limited to the field of scientific research.
Dynamic tests for structural monitoring aim at dynamic characterization of the structure and take the form of non-destructive dynamic tests or permanent monitoring.
In the case of non-destructive dynamic tests, vibration analysis allows the modal properties of the structure to be extracted. The vibrations can be induced by environmental vibrations produced by wind, traffic or microearthquakes, or alternatively by forced excitation. The latter can be supplied impulsively through the impact of a hammer or the fall of a weight, or regulated by an electrodynamic or electro-hydraulic actuator.
The most innovative and ambitious development of dynamic tests is the permanent monitoring: the measuring system is placed on the structure to acquire periodically different quantities related to the structural behavior and the operating and environmental conditions. This allows to study the correlation to provide reliable warnings.

1.2.2 Analytical Technologies of SHM

Analytical technologies are a useful tool for the simulation of structural behavior under operating conditions and with the presence of damage, in other words, the purpose of the analytical investigation is both diagnosis and prediction: in the first case, the causes of damage are sought through the correlation between the signals observed and analyzed, in the second case, their possible evolution over time is traced. The structural modeling can be geometric or numerical, but always deterministic.
A stochastic approach has recently become widespread, particularly in the case of historical buildings, where uncertainties about the mechanical properties of materials, boundary conditions and non-linear effects cannot be overlooked.
From a practical point of view, a stochastic process is a form of representation of a variable that varies randomly over time. Repeated testing of the same process results in

different trends of magnitude over time; observing the different realizations in an instant, it results a random variable X(t) that includes the different values that the process can assume in that instant.

However, in most cases a deterministic approach is preferred, whose uncertainties are managed by varying the parameters within certain limits and according to the results of the sensitivity analyses.

The model must be realized in order to represent in an exhaustive way the geometrical, mechanical and contour characteristics in order to simulate with a certain reliability the possible scenarios of damage to the structure.

Therefore, the experimental data collected are very important to calibrate the model and improve its predictive capabilities on phenomena delayed over time, such as the failure of foundations.

For the evaluation of structural behavior, the first step in analytical modeling is computer-aided geometric design. The geometric conceptualization of structures is obtained through a three-dimensional representation, essential to deal with the complexity especially of historical constructions.

Images are used to create the geometric model and then to precisely define the dimensions and capture the most significant details of the structure.

Photogrammetric techniques aim to obtain a three-dimensional representation of the object of investigation and to calculate through triangulation the coordinates of certain reference points.

Numerical modelling using the finite element method (FEM) is generally the most widely used solution for the availability of multiple commercial packages.

However, FEM modelling is often associated with errors related to the idealization of geometry, constraints and interaction between structural elements, errors that must be minimized through appropriate calibration.

Some of the most common objectives of modeling are:

- provide a basis for the design of the control system;
- Provide a term of comparison for the results of structural identification;

- serve as a tool to assess possible future changes in structural conditions and define some reliable alert threshold to perform non-linear analysis, able to take into account the effects of mechanical and geometric non-linearities.

Non-linear analysis focuses on second-order effects (interactions between internal elements and material discontinuities), while linear analysis provides good results when materials are subjected to high stresses.
The numerous advantages of non-linear analysis require a higher cost at the computational level, and therefore in terms of calculation time, as well as a certain difficulty in interpreting the results, caused by the marked dependence of the precision of the analysis on the choice of parameters.
In fact, the incorrect choice of mechanical parameters during the calibration of the model can significantly modify the results and lead to a complete misunderstanding of the phenomena investigated.

1.3 Monitoring phases

Il processo di monitoraggio può essere suddiviso in quattro fasi [3]
1. Operational assessment;
2. Data acquisition, normalization and cleaning;
3. Extraction of characteristics;
4. Development of statistical models.

1. The first phase identifies the object of the monitoring and the techniques to be used; this evaluation will be decisive in the following phases of identification of the damage in the structure. During this phase it is also necessary to decide the method of excitation to be administered and the number, type and position of the sensors to be installed. Finally, the hardware memory necessary to store the data to be collected and its economic weight influence the decision of the acquisition time interval.
2. In the data acquisition phase, it is necessary to normalise the data in order to obtain unambiguous information on the damage, since the measurements are

3. observed under different conditions. Standardisation is the operation that eliminates changes in the measures not caused by structural damage but byenvironmental and operational changes. If the environmental and operating conditions change during the time interval of observation, the data must be normalized in time bands characterized by operatin cycles and environmental conditions such as to allow a comparison of the measurements made. Since not all sources of variability can be eliminated, sometimes it is necessary to make a statistical study on the influence of these operating and environmental conditions on the responses in the monitoring. Finally, the phase of cleaning the data necessary to extract the characteristics of the structure takes place. If, for example, a sensor is installed by unskilled personnel, it may observe data that are not significant for the monitoring of the structure, so such data can be eliminated during the cleaning phase.

4. The third phase allows to evaluate whether the structure has been damaged or not, by extracting the characteristics: observing some parameters such as the amplitude of the vibrations or the frequency, it is possible to arrive at an initial assessment of the state of degradation.
   Another possible extraction method uses experimentally validated analytical tools, such as finite element models, to introduce engineering defects and perform numerical simulations.
   Alternatively, some structural elements belonging to the construction can be degraded under real load conditions and used to obtain specific characteristics. Usually the results obtained from different types of studies, both analytical and experimental, are compared to obtain even more reliable information.

5. In the fourth phase, algorithms are developed that, through statistical models, extract the characteristics of the system and allow the quantification of the deterioration of the structure. When the data are collected both before and after the damage, the algorithms are called supervised learning, if instead the algorithms work only on the data of the structures in healthy conditions, they are defined unsupervised learning.

The structural damage survey must first of all identify its objectives, therefore the operator must ask himself the following five questions proposed by Rytter [4]
1 Existence: is there damage in the structure?
2 Location: Where is the damage in the structure
3 Type: What type of damage is involved?
4 Extension: how severe is the damage present in the structure?
5 Prognosis: how much useful life does the structure have left?

In the case of unsupervised learning algorithms, the procedure allows to answer questions concerning the existence and localization of the damage, if instead supervised learning algorithms coupled with the use of analytical models are used, it is possible to know the type of damage, its extent and the final prognosis.
The control process is aimed at the construction of the control graph in which the characteristics are traced with their upper and lower limits, established according to the number of samples and within which the system is considered unchanged. When unusual variability is found in the characteristics observed and the statistical samples fall outside the control limits, an alarm signal is generated. The monitoring system then sends alarm signals when there is a possibility of damage to the construction, thus allowing the intervention of specialized personnel.

1.4 Type of sensors

The monitoring system is based on the use of a network of sensors that observe the symptomatic characteristics of the structure's behaviour.
Initially, the type of transducers to be used and their positioning must be identified. The complexity and the historical/strategic importance of the structure influences the number of sensors to be installed, while the type depends on the purpose of the monitoring and therefore on the characteristics to be detected. Through the construction of a finite element model it is possible to guess the possible future damage scenarios, which suggest the choice of the position of the sensors.

The reliability of the system is guaranteed when the staff is careful to identify any defective sensors, which may compromise the accuracy of the deductions on the phenomena investigated

Transducers are instruments that allow the transformation of quantities such as displacements, speeds, accelerations, voltages, and deformations, quantities that represent the response of the system, into an electrical signal that is subsequently processed by the data acquisition system.

The transducers can be of different types:

- Analogue: the output signal is an electrical quantity that varies continuously and requires an A/D converter;
- Digital: the output signal is composed of one or more signals that can only assume two voltage levels and the conversion takes place inside the sensor;
- Active: it does not need a power supply to work;
- Passive: in order to be able to function requires power supply.

In general, all sensors must meet the performance characteristics such as: sensitivity, resolution, range, linearity, hysteresis, accuracy, accuracy, isolation, low cost and durability.

1.5 Seismic Risk

Seismicity is the characteristic of the territory that describes the danger of seismic events that occur there, in fact, given the frequency and energy (magnitude) associated with earthquakes that affect a region and given the probability of the occurrence of a seismic event of a certain magnitude, in a certain time interval, we can define its seismic danger. In other words, the higher the probability of an earthquake of a certain magnitude in the same time period considered, the more problematic the territory.

However, the criticality of the event is determined by the characteristics of the buildings' resistance to the action of an earthquake shock.

The ability of a building to be damaged is defined as vulnerability. The more vulnerable the building stock of a territory is (due to inadequate planning, expiration of the quality of materials and construction methods, or poor maintenance), the more frightening the consequences of the earthquake are.

Finally, exposure is defined as the quantity of assets at risk combined with the level of anthropization of the territory and, therefore, the consequent possibility, in a given period of time, to suffer economic damage, in terms of human lives and/or cultural assets.
Seismic risk is determined by the combination of hazard, vulnerability and exposure and the Italian peninsula is associated with a medium-high risk level, for frequency and intensity of earthquakes.
Compared to other countries, such as California or Japan, where the danger is even greater, Italy has a very high vulnerability to the considerable fragility of its building, infrastructural, industrial and productive heritage.
Moreover, the "exposure" factor has a very high value considering the high population density and the presence of an unparalleled artistic, monumental and historical heritage.

Therefore, state government agencies have requested a classification of the territory based on the intensity and frequency of past earthquakes and have imposed the application of new and stricter rules for the construction of buildings and structures in areas classified as seismic.
The anti-seismic legislation requires that a building must withstand the least severe earthquakes without serious damage and without the collapse of the most severe earthquakes, safeguarding first of all human lives. [5]

The most realistic solution to deal with seismic risk is on two fronts: probabilistic forecasting of the occurrence of earthquakes and prevention.
The National Institute of Geophysics and Volcanology (INGV), a research body, starting from the analysis of past earthquakes and based on geological information has classified the Italian territory into four areas of decreasing danger (the first has such a danger that strong earthquakes can occur, the fourth is characterized by a probability of occurrence of a very rare seismic event).
Since 60% of Italian municipalities belong to the first three zones, the design and construction of new buildings must comply with anti-seismic standards, while buildings of old construction must be adequate. [6]

1.6 Seismic Observatory of Structures

In the light of the seismic events that periodically occur in Italy, the investigations on the seismic response of structures assume an increasingly important role. The permanent dynamic monitoring system allows the continuous recording of data during these phenomena, useful information in the evaluation of seismic damage and in the analysis of the behavior of structures.

As part of a seismic prevention strategy, the Seismic Observatory of Structures (OSS) was created in Italy. It represents the national network for the permanent monitoring of the seismic response of public buildings, designed, built and managed by the Civil Protection Department, both for cognitive and control purposes.

The OSS makes it possible to assess the damage caused by an earthquake to the monitored structures and those similar to them that fall in the area concerned, providing useful information for planning the activities of the Civil Protection immediately after an earthquake. It also produces useful data for updating design codes and technical standards for buildings in seismic areas. The data is fed into a computer at the headquarters of the Department of Civil Protection and is processed and disseminated via the Internet.

This helps to increase knowledge about the behaviour of buildings in the event of an earthquake.

The OSS network consists of two subnets:

- A key sample;
- Additional sample.

The Core Sample subnetwork comprises 105 buildings, including schools, hospitals and town halls, plus 10 bridges and a number of dams, which have been carefully designed

and equipped with a complete dynamic monitoring system, based on 16/32 acceleration measurements, [39].
The integrated Sample subnetwork comprises 300 public buildings that are strategic for seismic emergency management, equipped with a simplified monitoring system based on 7 acceleration measuremen

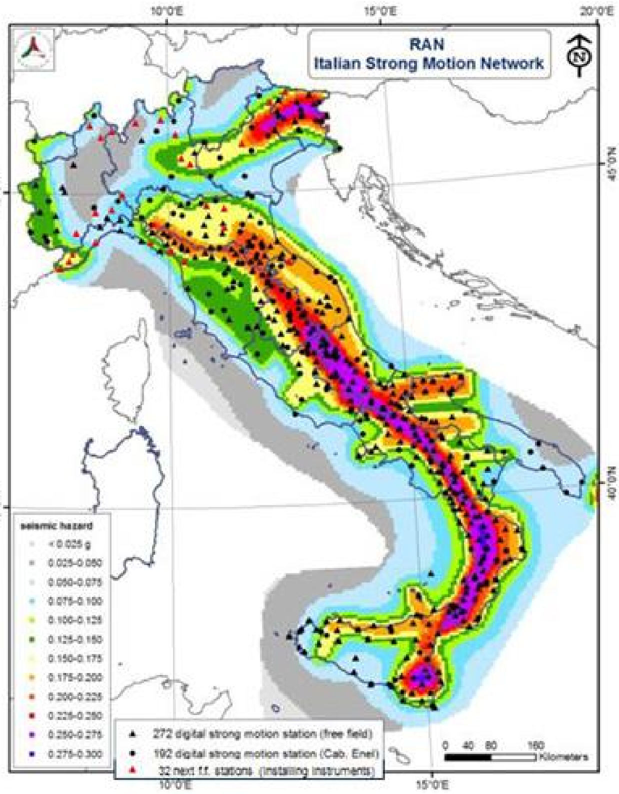
The fundamental sample allows you to:
- evaluate the security of buildings;
- provide data on seismic behavior, useful for theoretical and regulatory developments;
- Validate the rapid results of the integrative sample;
- provide data to support seismic scenarios.

The supplementary sample allows:
- approximate safety assessments;
- extrapolations of the fundamental data of the sample to the typological classes to which it belongs;
- the extension of monitoring to all strategic buildings with the contribution of the owners.

The monitoring systems of the individual structures automatically record the oscillations of significant intensity and send an alarm message to the central computer of the OSS network, which is located at the headquarters of the Department. The computer automatically transfers and processes the data for the first time and produces an event report.
The building that will be analyzed in this thesis work is part of the OSS network and is part of the basic sample.
The relevant use of the data provided by the OSS resides in the experimental identification of the dependence of modal parameter changes occurring during a simically intense earthquake.
This object of investigation has been extensively investigated, as demonstrated by the studies advanced by the Millikan Library of the Caltech campus, which, through the data obtained from the monitoring of structural safety, has shown how the lengthening of the natural periods of the structures is influenced by seasonal effects and seismic loads. Todorovska, on the other hand, analysed the same buildings, explaining the temporary frequency variations as an effect of structural non-linearity.

Of great interest is also the study conducted by Boroschek and Lazcano, on the structure of the Chilean Chamber, focused on the analysis of the variation of modal parameters following moderate and severe earthquakes. [7]

1.7 National Accelerometric Network (RAN)



The National Accelerometric Network (RAN) is the network that monitors and records the response of the Italian territory to the earthquake, in terms of soil accelerations.

The data collected are used to analyze the seismic shaking of the area around the epicenter, and then to assess the expected effects on buildings and infrastructure.

In addition, the definition of seismic action to be applied in structural calculations, obtained from studies of seismology and seismic engineering. The RAN is active and operating throughout Italy, with a higher concentration of measuring stations in areas with high seismic danger. The RAN currently has 561 permanent and temporary digital stations, equipped with an accelerometer, a digitizer, a modem/router with an antenna to transmit the digitized data via GPRS network and a GPS receiver to associate the data to the universal time UTC and to measure the latitude and longitude of the station.

Of all the total stations, 201 are located in Enel Distribuzione's electrical transformer rooms and 360 are located on public land (data updated in July 2017). The data flow to the central server of the RAN at the headquarters of the Department of Civil Protection, where they are acquired and processed automatically to obtain an estimate of the main descriptive parameters of the seismic event. Data from other publicly owned accelerometric networks flow into the RAN database in near real time, based on programmatic agreements and conventions. Parameters and waveforms are automatically stored in the central database and then made available online. [8]Structural Damage Evaluation

The structural damage caused by a seismic event can be identified through different methods of analysis: [6]

- Comparison between the peak acceleration measured at the base of the building and that used in the design of the work itself and any observation of overcoming the threshold. This methodology is rather simple and uses a trio of uniaxial accelerometers arranged in three directions perpendicular to the level of the foundations, but this method is sometimes useless because it can happen that the structural damage occurs at high frequencies, so the peak value of acceleration is not very correlated with it.
- Execution of the time-history analysis of acceleration during the earthquake and non-linear dynamic analysis of the structure: the verification of seismic damage is carried out by observing the deformations and stresses in the various structural elements induced by an earthquake with the ability to resist design. Also in this case, a trio of uniaxial accelerometers is used, arranged in the three perpendicular directions at the level of the foundations.

  The main critical point of the method is linked to its own hypotheses, since the real strength and rigidity of the structural elements can differ significantly from those of the design.
- Observation of modal characteristics before, during and after the earthquake: the period of the first ways of vibrating the structure is determined from the study of the time history of accelerations and from the analysis of the vibration frequencies of the same. As the damage progresses, the period of vibration is higher, as a consequence of the progressive degradation of stiffness. The limit of this method is due to its ability to provide information on damage but not on its location of the defect.
- Observation of the interplane displacement, obtained by linearly combining the absolute displacements of the planes of the structure. This method requires that the sensors are placed uniformly on the structure.

The USGS, the scientific agency of the United States Government, first suggested the choice of monitoring the inter-storey displacement and its experimental evidence has shown that this is the main indicator of the state of damage in a building.

In the literature there are also known correlations between the value of drift and the damage suffered by the structure.

However, some simplifications, such as the hypothesis of infinitely rigid plane, allow to reduce to a double triplet (to take into account also the effects of the earthquake in the vertical direction) of uniaxial accelerometers per plane the number of total sensors. This monitoring strategy has the advantage of providing information on the damage not only regarding its existence, but also on its location, when instead in the case of monitoring based on the study of the modal characteristics of the structure, the location of the damage is particularly difficult and uncertain. The method consists in assessing whether during the seismic event the values of the drift have exceeded the predetermined threshold values, which characterize different levels of damage.

From the above, it is clear that if we are interested in the location of the damage and not only to highlight its presence, the most appropriate strategy is to monitor the absolute and relative movements of particular points of the structure. The problem arises of measuring absolute displacements. In traditional (non-seismic) monitoring, the displacements between two points are generally determined by means of special transducers (e.g. LVDT displacement transducers) that are equipped with two parts attached respectively to the two points. This approach is not possible for seismic monitoring as there is no possibility of having a fixed point against which to report the measurement. The technologies that today allow the absolute measurement of displacements are those based on the use of GPS or those based on the use of accelerometers, in which the displacements are calculated by means of double integration in the time of the accelerations. GPS technology guarantees displacement measurement accuracy in the order of 1 mm and it is possible to acquire displacement time histories with a sampling frequency of up to 20 Hz. With such a limited sampling frequency, oscillations in the structure at frequencies above 2 Hz are lost and measurement errors are significantly increased. In addition, the GPS receiver must be placed on the roof of the building in order to be in line of sight with the satellites; therefore, diffused monitoring of the building is not possible. For this reason, the use of GPS is valid for structures characterized by a high period of oscillation and a single prevailing mode of vibration. In fact, in the literature are available experiences of GPS monitoring

Chapter 2

2 Case Study: The Rialto Bridge

In order to offer a detailed description of the object of study, this chapter will describe the following characteristics of the bridge: starting from the geographical data, a historical, architectural and geometric framework will be carried out, and finally the last restoration operations carried out on the bridge will be illustrated.

2.1 Geographical Positioning

The Rialto Bridge, in Italian "Ponte di Rialto, is located in the city of Venice.

Figure 2-1 Geographical Positioning

The following are the geographical positioning data [9]:

- Latitude: 45° 26' 23" N;
- Longitude: 12° 19' 55" 13 E;
- Height above average sea level: 2.56 m.
- Seismic class: zone IV (very low seismicity).

From a geographical point of view, the municipality of Venice is divided into two parts, its island area and its mainland area.

The Rialto Bridge is one of the four bridges, with the Accademia Bridge, the Scalzi Bridge and the Constitution Bridge, that cross the Grand Canal, in the island part of Venice. Of the four he Rialto Bridge is the oldest and certainly is one of the most popular attraction of the city in terms of tourism.

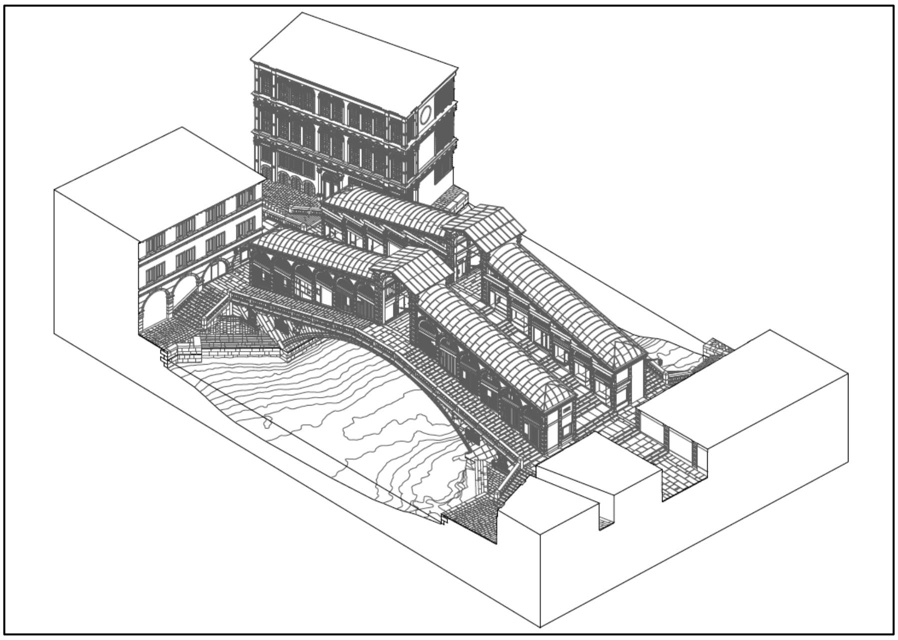


Figure 2-2 Urbanistic context of the Bridge in 3D

It is a strategic infrastructure for the pedestrian viability because if connects the districts of San Marco and San Paolo, and the communication necessity between the boroughs has led to reconstructions and adjustments several times also in the past.

2.2 Historical Description

The first Bridge over the Grand Canal was built in 1181: it was a boat bridge and assumed the name "Ponte della Moneta", presumably because of the coin mint presence at its eastern entrance. [10]
In 1225 the old Bridge was substituted by a new one made of structural wood, because of the eastern bank market expansion, so the increasing pedestrian traffic needed a stability that the old floating one could not guarantee.
The structure had two inclined ramps and a removable central section, that could be lifted in case of the passage of massive ships. Given the close collaboration with the market, the bridge changed its name and became "Ponte di Rialto".

Between 1400 and 1440 two rows of shops were built along the sides of the bridge, and the markets rent contributed for the maintenance of the wooden bridge. It was partly burnt in a local revolt in 1444 and again in 1524 it ceded under the weight of a great crowd watching the procession for the wife of Marquis of Ferrara. [11]
In 1503 for the first time a designer prosed to rebuild the bridge with stones; in the following years other ideas were considered but only in 1551 the request for the renewal of the bridge became pressing, so some famous architects, such as Palladio, Vignola and also Michelangelo showed their projects, but all of them proposed a solution with several arches according to the classical approach, which was actually inappropriate for the boat traffic.
Antonio da Ponte was chosen for his proposal because his project involved a stone bridge with a single span and the entire construction lasted from 1588 and 1591. [12]
The actual shape seems similar to the old wooden bridge, with two inclined ramps, a central portico and rows of covered shops on both sides.
The Rialto Bridge is even now the biggest one in Venice between those which span the Grand Canal, and because of its eye-catching design it is one of the most celebrated bridges of the Italian Renaissance in addition to be one of the architectural icons of the city.

2.3 The Bridge Construction

The construction method used to build the Rialto Bridge has not been documented with precision, so we can refer to the techniques used to build other river bridges of the same age.
When a bridge crosses a river it is required that the water channel is temporarily diverted and drained from the site, and for the case in question, it seems to be the choice adopted, considering the fact that the Grand Canal was born as an artificial canal. Therefore, the water was drained to allow the construction of foundations and abutments.
The designer of the bridge, Antonio da Ponte, seems to have been one of the first to recognize that the choice of arrange the stones of the abutments in the perpendicular direction to the thrust of the arch could significantly increase the strength of the abutment, in fact this construction choice dates back to this period.

The Istrian stones forming the arch of the soffit were placed using a wooden scaffold and to provide the necessary energy to lift them it is probable that pairs of horses were used. These stones, as it is understood by the name, come from the region Istria which is located 100 km from Venice, in Croatia, at the time under the control of the Venetian Republic.. The abutments are sufficiently rigid and the line of the arch is so important to avoid that the line of the thrust comes out beyond the limit of the profile of the bridge, eventuality that would have led to the expulsion or the crumbling of the stones of which it is compost, even if there had been the mortar to unite them.

After completing the construction of the arch, the side walls were built with the purpose of containment to avoid excessive deformations of the deck during the placement of different layers of stones and bricks. After completing this operation and after completing the creation of the ornamental motifs on the side walls, the lateral scaffolding has been removed.

2.4 Description of the superstructure

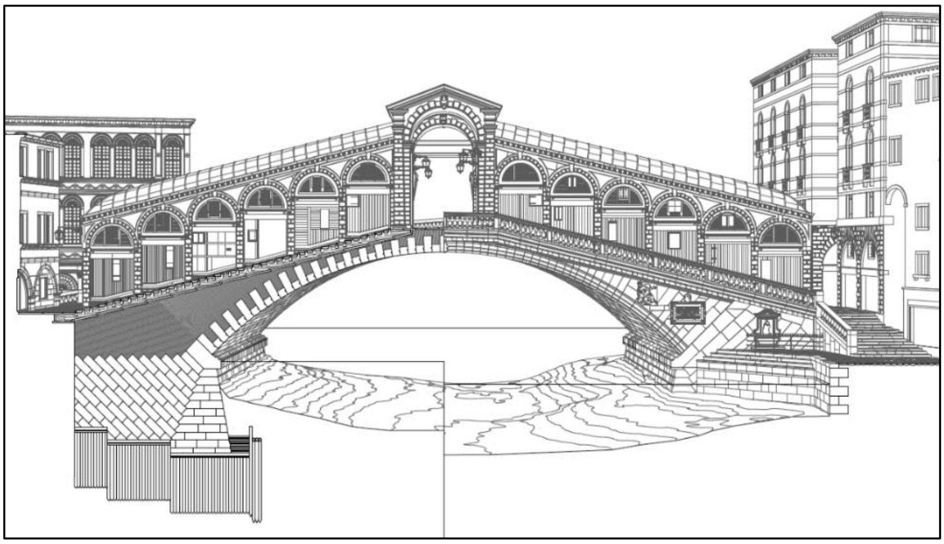


Figure 2-3 Front view of the Rialto Bridge

The Bridge has a 47 meters long span and it’s 22 meters wide, it is composed by two macro-structures with different typological features: the masonry arc and the shop buildings overlying.

The masonry arch is lowered shaped, it has a 28.8 meters span and it reaches 6.40 meters in the keystone, giving a span/rise ratio of 4.5, and this shape generates large lateral thrust which is transferred into the abutments. A smaller span/rise would be less onerous in terms of lateral
actions on the adjacent elements, but it would not have provided the necessary space under the arch permitting the passage of vessels, also considering the high and variable level of the water in Venice [13]
The lower part is made in Istrian stone with a uniform thickness of 70 cm clamped to the upper masonry layer, which has a variable thickness from 70 cm in key to 400 cm near the shoulders, and this masonry layer constitutes the body of the substructure up to the level of the paving.
The shop buildings are composed by masonry walls, they have wooden floors for the shop lofts and wooden barrel roofing with an external coating in lead. The reason of the change of materials, between the stone and masonry galleries and the inner part of the shops, could be seemingly associated to a reduction of the mass gravitating on the deck.
The superstructure that houses the side shops is composed of two parallel rows of masonry portico structures divided by a central pedestrian corridor, while externally, on the north and south sides, there are two side ramps with stone floors that allow visitors to see the Gran Canal, protected by a stone balustrade that runs longitudinally all the bridge.

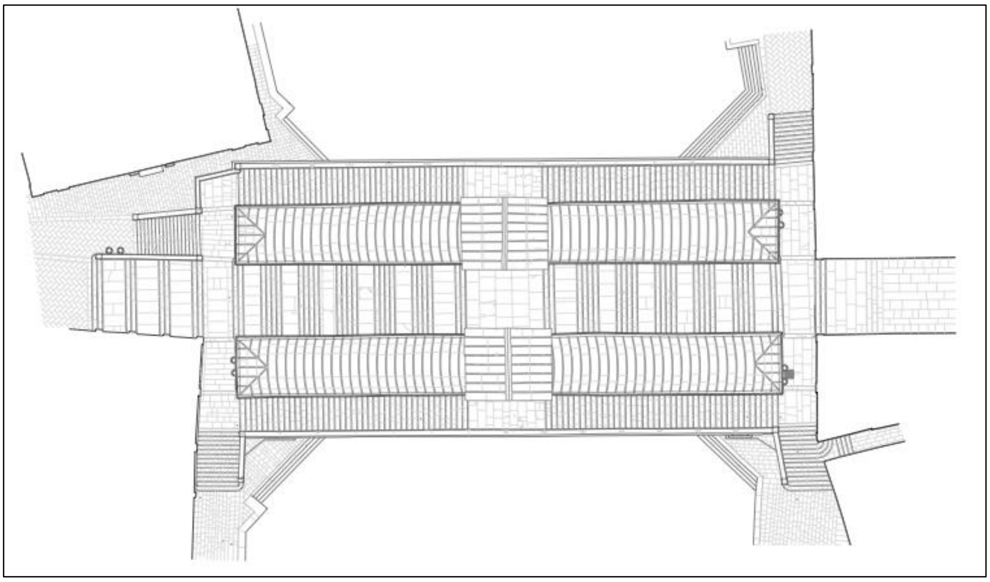


Figure 2-4 Top view of Rialto Bridg

Each of the two rows houses twelve commercial premises, six on the right and six on the left looking at the bridge from the Canal, and at the top there is a large corridor that allows pedestrians to cross the bridge transversely. This corridor is covered by a cylindrical dome vault, aimed at protecting visitors and Venetians from atmospheric agents.

he galleries just described are semi-circular and also the central dome vault has the same shape, although of greater proportions, and this aesthetic choice gives the visual continuity feeling across the structure.

The void below the arch and the one provided by the central portico corridor provides an almost uninterrupted view through the structure and make it appear more lightweight. The abutments are disproportionately large, but their dimension is necessary to stability due to presence of poor ground condition. This particular however is not so much visible because it's hidden below the water level, as it can be understood by the previous pictur

2.5 Description of soil and foundations

The geognostic investigations for the study of the stratigraphic successions that characterize the Venetian soil are conducted periodically because the lagoon area of Venice is subject to the phenomenon of sedimentation of river deposits. It is known in fact that the geographical conformation of the city in recent centuries has changed considerably, so as to lead the authorities to the decision to deviate the course of some rivers emissaries to contain the phenomenon. Below is reported an illustration that shows the superimposition of the current profile of the city with the of the fifteenth century attributed to De Barberi. [14]

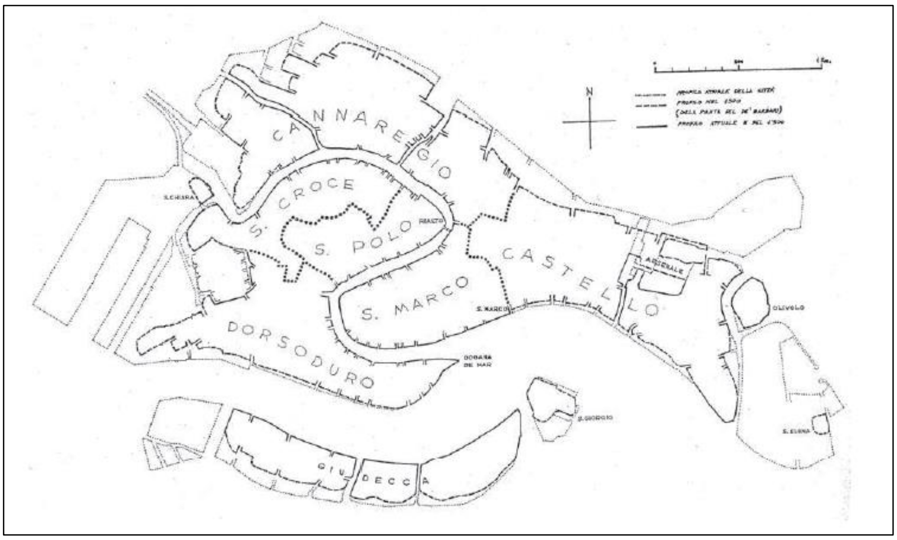


Figure 2-5 Comparison between today and the XV century city conformati

In order to reconstruct the stratigraphic profile, undisturbed samples were taken and then processed in the laboratory. They are representative of the characteristics of density, compressibility and resistance to cutting of the soil. SPT (Standard Penetration Test) tests were also performed during drilling.

From the examinations it results that the Venetian lagoon is characterized by a type of land called "caranto", particularly present in the lagoon of Venice, but also found in alluvial sediments of the low Venetian-Friulian plain. [15]

The level of sediment is located at the base of the lagoon deposits, between the underlying alluvial series. This range, averagely 1-2 m thick, consists of clayey silt and clay that are remarkably compact. Below is a stratigraphic typological section of the soil forming the Venetian seabed.

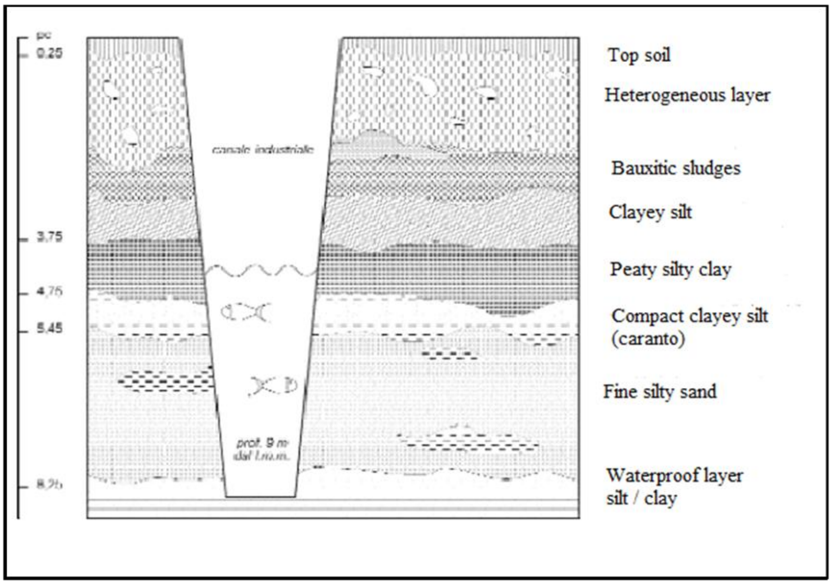
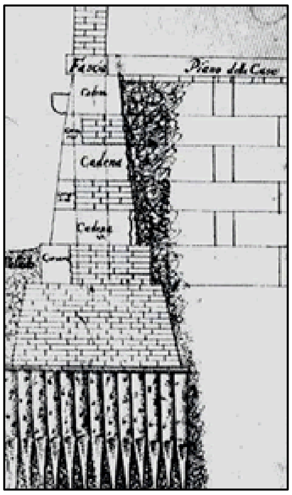


Figure 2-6 Soil stratigraphy under Venice

For all the structures of the city, the area to be built was first solidified by planting pointed wooden poles (larch or oak), short and knotty until reaching the layer of particularly hard and compact clay soil, or the caranto, of particular consistency, which is located about ten meters below the surface soil layer of the lagoon.



If the layer of caranto is too deep, the subfoundation is made by compaction. In this case, the poles are planted over the entire surface above which the building will rest, first closing the perimeter with a thick pile and then proceeding inside with a spiral pattern towards the center of the area.

This procedure is done if the building to be supported is very heavy, for the construction of the Rialto Bridge where 11,000 poles were implanted to support the weight of the stone bridge. Above the heads of the poles were then fixed two layers, crossed with each other, of thick larch wood planks. Above this special raft is raised the actual foundation, consisting of a plinth wall, with the walls slightly inclined, in fairly regular layers of blocks of Istrian stone until the average tide level is exceeded. [16] In the following image is visible a section to understand the foundation scheme:

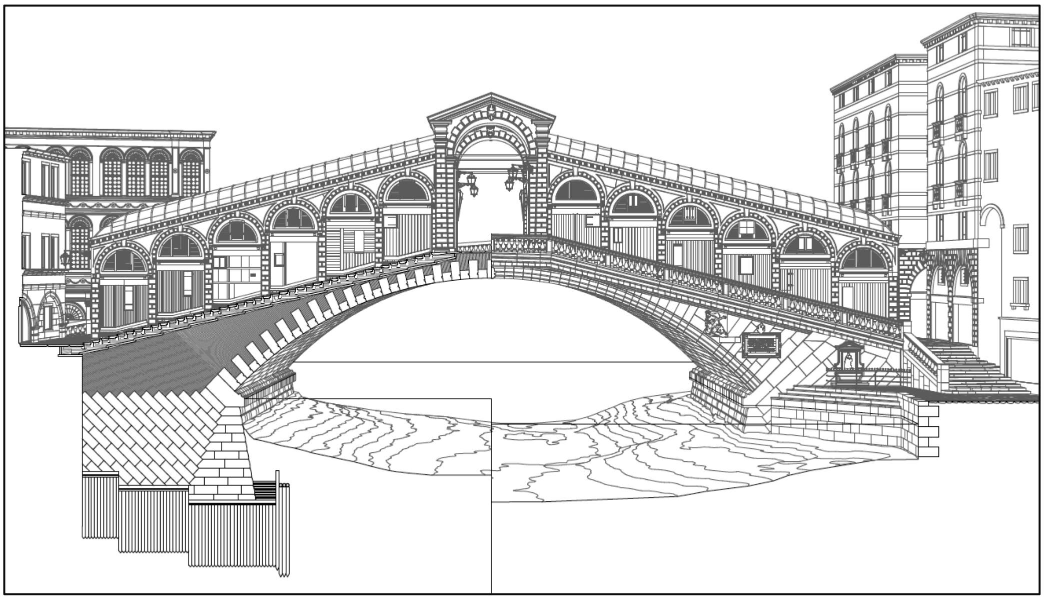


Figure 2-7 The bridge foundations

With this type of foundation, only the part made of Istrian stone remains in contact with salt water and air, while the wooden parts remain stuck in the mud or sediment, undergoing a process of mineralization over time that instead of rotting make them increasingly resistant. Due to its compact and particularly impermeable nature, or in any case not subject to modification even when in contact with erosive elements, Pietra d'Istria was the only solution to support the immense weight of Venetian factories, all this without being exfoliated or sizzled.
To get an idea of the weight, just think that the two supporting pillars are made of Rovinj Istrian stone, using more than 5200 blocks weighing about 13 quintals each, for a total of almost 6800 tons. [17]

2.6 Architectural Description

Crossing slowly the walkways you can appreciate the ornaments of the bridge: the balustrades have an aesthetically attractive profile, while in the superstructure, the shape

of the stones in the arches is clearly visible. The shops space under the circular archs are painted in a dark colour
so that the sight is attracted to the white stone structure, making the look of the bridge lighter. The pitched roof above the central arch, has been decorated with channels excavated into the stone. Travelling slowly on the gondolas under the arch the details on the outside of the Rialto Bridge are clearly visible. On each side of the bridge there are two significant representations of Venice. On the south side (in direction of San Marco) it is evident the Annunciation with the Gabriel Angel, Mary and the dove, which indicates the date of foundation of Venice, March 25, 421; on the north side (in direction of Piazzale Roma) St. Mark and St. Theodore, the patron saints of the city.
During the construction of the bridge it seems that many people were afraid to cross it and religious figures exhorted the pedestrians to place their trust in God to ensure safe passage.
The columns that mark the balustrade are alternated with a squared column every five, which continues below the balustrade to create a vertical continuity; here in fact regular stone cubes, small and close, protrude from the side walls. These elements are not structural, but they add interest, structure and appearance to the aesthetics of the bridge.

Figure 2-8 Lateral view of the Bridge

2.7 Last Restoration Operations

In previous years, several diagnostic campaigns were conducted in order to accurately identify the characteristics of the structure studied. The campaign were made through instrumental and non-destructive experiments, with samplings and laboratory test, and all these operations enlarged the available data in the archives and bibliographies. Over the centuries, the bridge has been subject to modifications and restoration works of a local and fragmented nature (such as the placement of tubes between the intrados and the floor), but it has never shown the need for static restoration or work for the entire structure.

The structural critical analysis, the safety assessment and the subsequent static intervention project are part of the complex of operations carried out by the Municipality of Venice for the overall restoration of the Rialto Bridge.

The analysis started from the critical investigation of the deformations of the Bridge, acquired through a precision topographic survey. A simplified model was built on the profile of the main elements and the analysis highlighted the excellent conservation of the vault structure, despite the relative foundationa displacements that occurred between the shoulders, and therefore the restoration does not involve static consolidation of the foundations and the vault in masonry and stone, with the exception of local interventions to repair injuries.

However the monitoring campaign conducted on the Bridge revealed different types of local damages on materials and on external surfaces: they concern the strain and unbalance, the erosion and pulverizing of mortar joints, the spalling, the brick gaps, the disintegration and the crackings. [18]

To repair these local damages the intervention techniques conducted were:

- grouting with resin and stone aggregates (or with mortar based on lime and stone dust) for the crackings and brick lack;
- masonry integrations and bedding mortars for the mortar joints erosion;
- consolidating microinjections of resin and plaster for the disintegration of the stone.

In spite of these local damages, preliminary investigations have shown that the structure and foundations, contrary to what was thought, are still in good conditions.

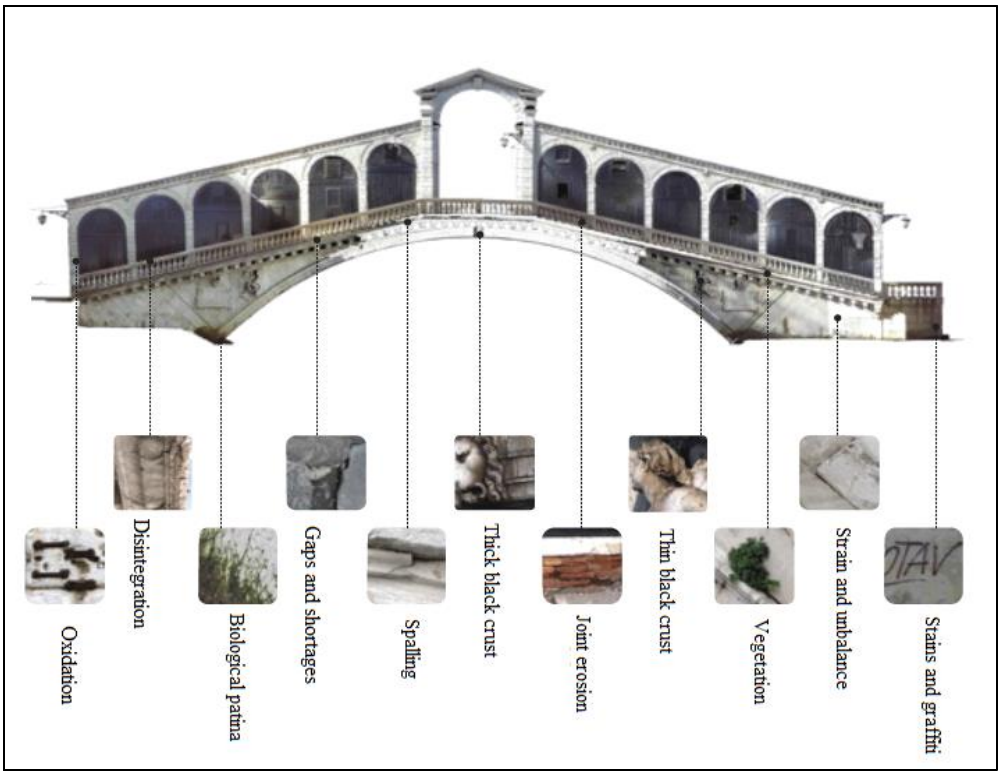


Figure 2-9 Llocal damages on materials and on external surfaces

Surprisingly, however, the balustrade parapet running on either side of the bridge was in a condition-safety limit because of an injury that ran along it longitudinally, with the risk of tipping over. The walls of the superstructure for the shops, which presented detachments between the stone masonry on the external facade and internal bricks that once collaborated, were also consolidated because the resistant section of the masonry resulted reduced. [19]

Two structural interventions were also carried out: one on the balustrades and one on the wall vestments of the workshops.

To consolidate the balustrades of the bridge and the breakage of the supporting shelves, the last of which emerged only during the opening phases of the external ramps during site preparation, led to the design of damage repair interventions with the simultaneous static improvement in relation to the horizontal thrust, consisting in the installation of stainless steel shelves with high reverse T strength.

The columns of the balustrades, all removed and numbered, after the restoration were fixed to the base with new lead drippings, first hot, then cold rebated, while previously they had been fixed with very rigid cement castings that had caused the breakage.

Figure 2-10 Consolidation of the bridge balustrades

The manifestations of instability on the buildings of the shops showed the mutual disconnection of the vestments that, as a result of the progressive subsidence of the buildings, have separated becoming unstable. The works on the wall vestments, aimed at reconnecting the stone wall with the brick wall, was instead carried out with basalt strands. It is important to underline that the interventions made with composite materials do not damage the historical value of the bridge thanks to both traditional and innovative restoration techniques. [20]

It was necessary also the restoration or replacement of damaged wooden elements.

Another improvement consists in waterproofing the top surface of the bridge with an elastic sheath in order to allow small physiological movements. The sheath was not laid in direct contact with the extrados masonry of the arch but saw the interposition of a layer of "sacrifice" lime, so that this intervention was also reversible.

The chemical and biological deterioration instead concerns the oxidation processes, the vegetation presence under the arch, widespread stains and superficial patina (in some point it results thicker).

The actions carried for contrast the chemical and biological damages, of a conservative nature, were aimed at cleaning the stones of the intrados of the arch, the sides, the parapets and the chews that make up the flooring. The operations used laser and chemical compresses for the superficial patina and chemical disinfestation for vegetation and algae for the arch intrados.

The latter, very large and heavy stone slabs, individually disassembled and numbered, were cleaned on a site set up at the foot of the wall and in an adjoining field.

Finally the cover of the lead-plate shops has been dismantled and relocated; it has been completely preserved, but always repaired on the inside, so as not to create chromatic differences between one plate and another

2.8 Description of deformation behaviour

The displacements of the Rialto Bridge structure have been subjected to an accurate study through a topographic monitoring system, with a campaign of surveys conducted between April 2013 and May 2014.

The topographic reliefs were commissioned by the Venice Public Works Department to know the state of the building and to understand if there was a need to intervene on it to preserve its integrity. The data were collected on a monthly basis and processed through diagrams sent to the Administration together with interpretative reports on the progressive trend of monitoring. [21]

The temperature variations occurring during the calendar year are in fact a kind of load test and the acquisition of data provides useful information for the identification of any settling of foundations and abutments due to the alternation of the seasons.

The "targets" were installed for their reliability, precision and camouflage, as required in the Automatic Monitoring Systems with Robotic Stations installed in Fixed Stations and/or in pre-arranged Stations for Periodic measurements for structural control. They have been placed long:

- the representative line of the arc close to the intrados;
- the intrados of the arch along two lines placed at a third of the width of the deck;
- on the four tables of the shops, in correspondence of the taxes of the arches and in key of the central newsstands;
- on the elevations of the buildings adjacent to the Bridge.

The monitoring champagne has been conducted in such a way that each target can be observed from two LEICA TRCA 1201 measuring stations, which operate using an internal camera that undivides the mini prisms and they automatically collimate them without the need for operator intervention, which would obviously be less accurate.
The theoretical instrumental error in the design phase of the monitoring system is of the order of 1 mm but during monitoring the results provided higher accuracy than the theoretical one.
Some targets due to accidental hits have lost their precision, but predicting this possibility, the system was designed with an overabundant number of control points and the tampered targets have benn reported and neglected from time to time in the interpretation of data.
The measurements were always conducted between 3 and 7 in the morning in order to avoid interference with the population, and in the minimum daily temperature range avoiding the effect of direct sunlight.

Figure 2-11 Summer displacement of the Arch

Figure 2-12 Winter displacement of the Arch

It is visible in the first row of figures that during the hot months it occurs an extension of the arch, and due to the resistance exerted by the shoulders it raises and is subject to a transversal expansion, while in the cold months opposite displacements take place, so the the vault lowers and contracts.
The figures show that the south side of the bridge extends in the hot season and shortens in the cold one, while the north side deforms in the opposite. This phenomenon is due to the difference of temperature between the two sides of the bridge, and because of the direct sunlight to which the south side is subject, which causes deformations of a higher entity on the south side than on the north.
As regards the superstructure hosting the shops it has been observed that the movements are governed by seasonal variations and by the vault on which they stand.

Figure 2-13 Seasonal dispacements of the superstructure

2.9 Evaluation of the properties of ancient masonry

The need to understand the urgency of the start of a major restoration and possible improvements from the seismic point of view led to investigations on the Rialto bridge, with consequent tests for the mechanical characterization of the constituent materials.

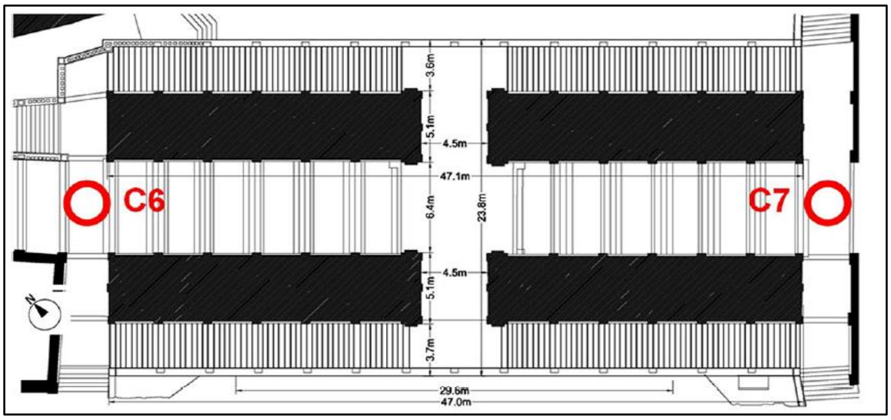


Figure 2-14 Location of the extraction points of the test samples

From the lower part of the bridge, at the shoulders, core drills were carried out to take samples of masonry and samples of Istrian stone, constituting the intrados of the arch.
The samples were taken undisturbed and checks were carried out on them to prove the absence of deterioration. In the following image the extraction points of the samples are visible:
In detail, the tests were carried out with the Galdabini Sun universal machine 20 (maximum load 200 kN) and the Gadalbini press 5000 kN (maximum load 5000 kN). [22]

Figure 2-15 Test samples

Despite the different dimensional configurations, the limited dispersion of the results obtained
show a good quality of the analysed samples.
Moreover, considering the irregularity of the samples, due to the work of extraction of the carrots, the physical and mechanical quantities obtained are still reliable compared to

the values recommended by the technical standard and the values available in the literature. The following table show the test results:

Samples	Typology	Depth (m)	Length (cm)	Diameter (cm)	Weight (kg)	Maximum load (N)	Tensile strength (MPa)	Modulus of elasticity (MPa)
1	Masonry	0.8	18	7.8	1.462	25,000	5.23	4350
2	Masonry	1.8	19	7.8	1.5875	26,500	5.55	4600
3	Masonry	2	17.5	7.8	1.276	23,000	4.82	3900

Figure 2-16 Extracted C6 samples results

Samples	Typology	Depth (m)	Length (cm)	Diameter (cm)	Weight (kg)	Maximum load (N)	Tensile strength (MPa)	Modulus of elasticity (MPa)
1	Stone	2	18.8	7.8	2.394	425,000	88.99	34,650
2	Stone	2.8	17	7.8	2.179	365,000	76.42	31,200
3	Masonry	3.8	18.8	7.8	1.662	32,000	6.70	4750
4	Stone	2.3	17.3	7.8	2.1685	405,000	84.80	33,800

Figure 2-17Extracted C7 samples results

The investigations carried out allowed a group of experts to study the correlation between the results obtained by integrating the dynamic results with both ND (non-destructive) and MD (microdestructive) tests.

- The first method consists in the realization of a dynamic monitoring for the identification of dynamic parameters of damping, frequency and modal forms. These parameters are searched starting from environmental vibrations, in fact, due to the fragility of the context, the use of environmental vibrations typical of Venice has become the natural method of excitation of the Bridge. This method also makes a distinction between global and local studies.

  Dynamic monitoring has used pedestrian traffic, wind and wave shaking caused by boats in the Grand Canal as natural sources.

- The second method involves the integration of the results of the MD tests, i.e. breaking load tests on samples extracted from the structure under analysis in order to determine the modulus of elasticity of the constituent materials and compare them with those obtained from the ND approach.

In the case of historical constructions, as in the case in question, both approaches play an important role since their application is non-invasive.

For the Rialto bridge an ND test based on the GPR technique was conducted, which exploits the propagation of electromagnetic waves through the studied material.
To understand the nature of the GPR technique, the explanation of another similar technique can be useful, namely the sound type ND technique, which uses an instrumented hammer as an excitation point, and sensor accelerometers as an acquisition point. This technology is relatively old, but has been considered reliable since the beginning of the 1970s, especially to assess the level of compactness in masonry structures.
The excitation induced on the surface starts a vibration, which spreads within the structure itself in the form of longitudinal (compression) and transverse (shear) elastic waves.
The properties of the signal recorded by the sensors depends on the nature of the layer of material crossed, in fact the signal is attenuated, deflected or absorbed when it encounters discontinuity, vacuum or significantly different materials.
The GPR technique obtains the same results that can be obtained from the study of materials by means of sound tests, but it is more precise in detecting discontinuities and different materials within the masonry; moreover, this technique is used especially in the presence of large surfaces.
The different resistivities and dielectric constants of materials, generate changes in electromagnetic waves that testify to the presence of discontinuity.
Dynamic monitoring, through the identification of the main modal frequencies and the corresponding modal forms, has served to study the interaction between the arc and the upper structures.
The results have shown that the structural system appears as the result of a sum of two distinct contributions, in particular those of the arched structure and the system of the upper structures hosting the shops.The significant difference between the frequency values measured for the arch and for the individual upper structures, as can be seen in the table, confirms the actual distinct behaviour of the two structural types.

Frequency [Hz]	Damping [%]	Localization
5.27	2.23	Shops- North side
5.45	2.7	Shops- South side
7.08	2.64	Shops- North side
7.76	0.91	Shops- South side
19.75	2.23	Arch
48.2	0.02	Bridge Shutters

In the case of the Rialto bridge, in addition to the sound tests carried out on site, the GPR test was also carried out on a sample of the bridge masonry; this allowed a comparison to be made between the elastic modulus determined by its breaking by compression and the value of the elastic modulus obtained by sound tests.
In this regard, the figure
shows the agreement between the MD (in figure DT or destructive tests) and ND (in figure NDT or non-destructive tests) tests relating to the evaluation of the modulus of elasticity in the compression of the masonry, also in consideration of other similar investigations. The value relative to the Rialto Bridge is indicated by the blue symbol, and the distance of the point from the diagonal line represents an index of reliability.
For the Rialto Bridge, the comparison shows that the ND test results underestimate the results obtained with the MD results.

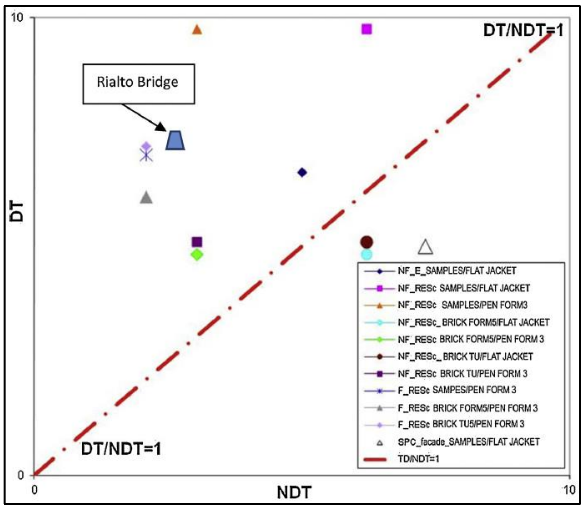


Figure 2-18 Comparison between Non destructive and Destructive test results.

Chapter 3

3 Finite Element Modelling

The most congenial geometric modelling method to physically and mathematically represent the reality of the structure under examination is the finite element FEM method. It maintains a dominant position in the panorama of numerical approximation techniques and represents the core of most of the codes of automatic analysis available on the market, in fact, for the work of thesis in question, the finite element modeling has been supported by the software Ansys Mechanical Apdl, developed since 1970 in Pennsylvania, USA.

The FEM method is based on a numerical technique for the search of approximate solutions to the problems described by the partial differential equations, reducing the latter to a system of algebraic equations. The construction of the model was articulated in the following phases:

Construction of the geometry;

- Definition of the type of element (solid and shell);
- Attribution of the mechanical properties of the material to the element;
- Definition of the mesh;
- Definition of boundary conditions;
- Analysis;
- Post Processing.

3.1 Geometrical Model

The geometric model has been appropriately reconstructed from a photogrammetric survey conducted with Laser Scanner Survey, which was divided into the following phases:

- Acquisition of scans by means of 2 scanners at the same time as the georeferencing of markers;
- Acquisition of RGB images (acronym of Red-Green-Blu, i.e. in colour) aimed at defining the chromatic aspects of the cloud derived from the laser scanner, aimed at generating orthophotos.

During the above mentioned campaign two FARO FOCUS3DS laser scanners were used, while for the geo-refencing of the markers necessary for the relative positioning of the different scans a TOPCON GPT9003A robotized Total Station was used, with an accuracy of 1" (1/1296000 round angle).The combined use of these tools has allowed the acquisition of the information necessary for the subsequent phases of creation of the geometric model.

Below there are some of the tables obtained:

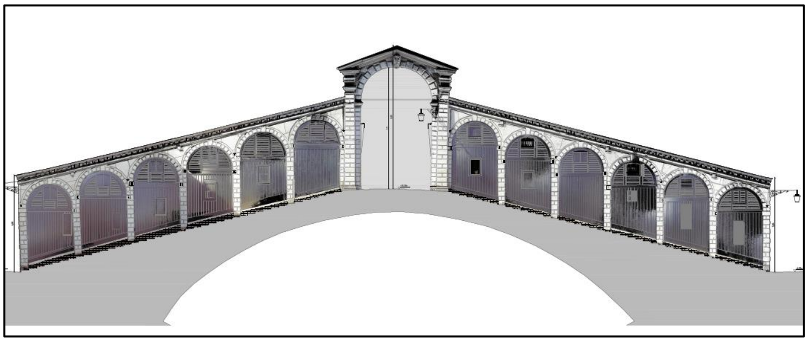


Figure 3-1 Front Chart, North Side

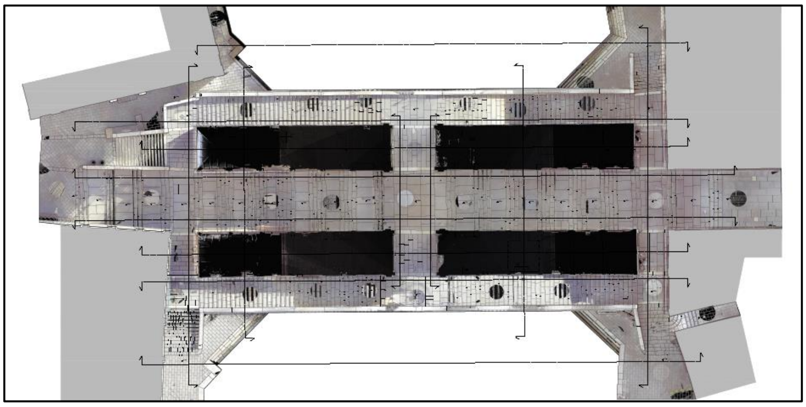


Figure 3-2 Planimetric Chart

Figure 3-3 Planimetric Section Chart

This first phase of geometric relief was followed by a second phase of geometric modelling, in which the bridge was schematically sketched, simplified and broken down into 41,164 brick elements (Wedge6 and Hexa8) for a total of 59,136 knots, as can be seen in the following image:

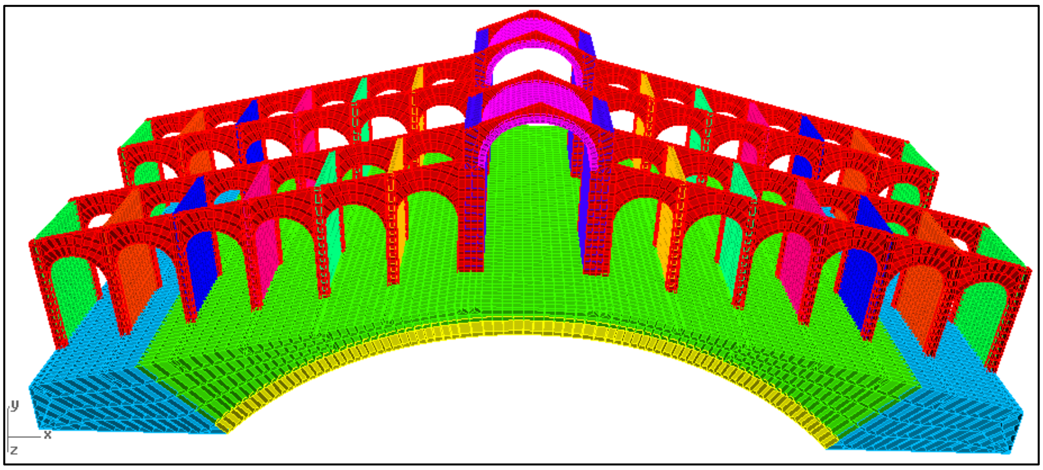


Figure 3-4 Architectural 3D Model

This last product of the geometric modelling activity was of fundamental importance for the formulation of a second scheme, born from the need to import the features of the bridge into the working environment of the Ansys software.

The geometry of the structure has been further simplified and regularized to allow the transposition of the model and the structural behavior has been represented by two-dimensional and solid elements.

The bridge was therefore divided into two macrostructures: for the arch below it was decided to use three-dimensional volumetric elements (Solid185), while the masonry superstructure housing the shops was modelled through shell elements (Shell181), being characterized simply by walls.

- Shell181 element: is suitable for analysing both thin and moderately thick shell structures, characterised by four knots each with six degrees of freedom, three of which are associated with translations and three with rotations. Both quadrangular and triangular elements can be used as filling elements in mesh generation.

  These considerations are valid in the hypotheses in which the element considered performs the function of "shell", but in the hypotheses of membrane behavior then the rotational components are neglected and only the translational ones are present.

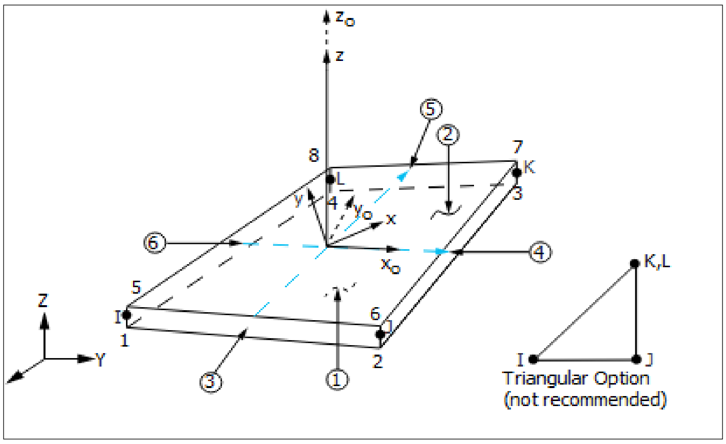


Figure 3-5- Shell181 Element

- SOLID185 element: used for the three-dimensional modeling of solid structures, it is defined by eight nodes with three degrees of freedom in each node (translations in the x, y and z directions of the node.

  The element has plasticity, ability to represent great deflections and great deformation capacity.

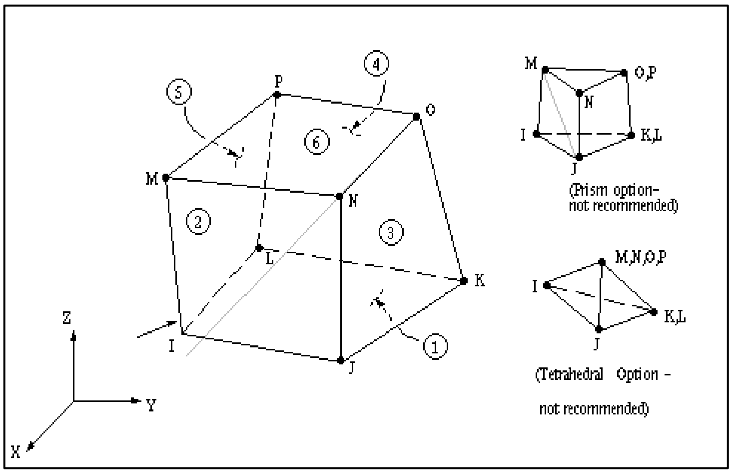


Figure 3-6- Solid185 Element

The volumetric elements making up the arch have been designed and built in their geometric features in such a way as to communicate coherently with the Shell surfaces representing the walls of the superstructure.

Therefore, observing the Figure 3.7 it is possible to see that some of them have a prismatic shape, others have a tetrahedral shape and, finally, in correspondence with the actual arch, it is possible to observe volumes with a curved profile.

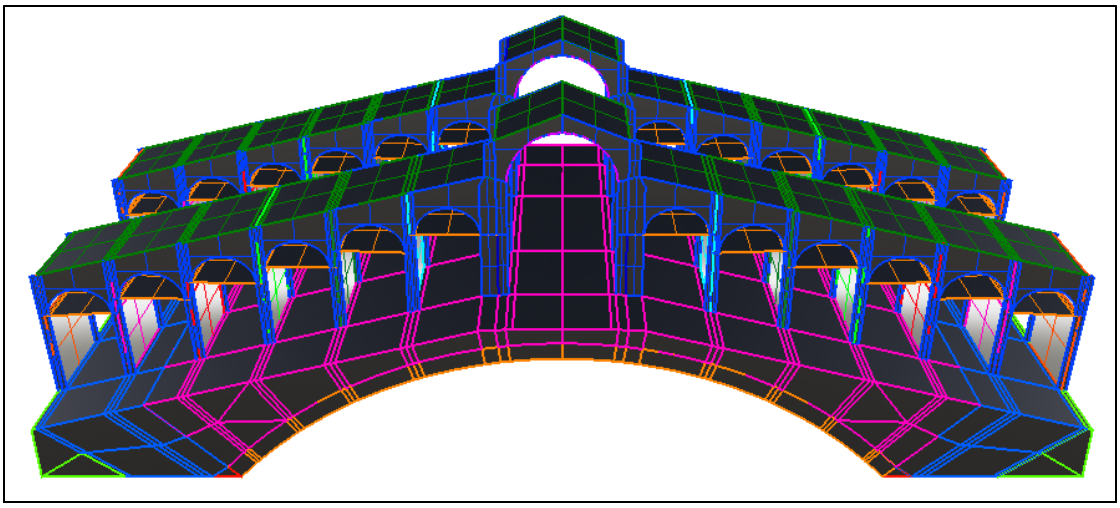


Figure 3-7 Geometric Model in Rhino5 Drawing Software

Finally, by way of example, there is a detailed image that illustrates the way of operating in the process of delineation of volumes:

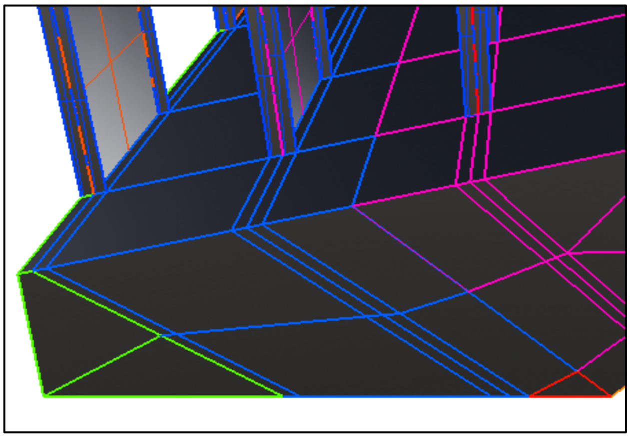


Figure 3-8 Detail on the construction of the Volumes

The upper structure that houses the shops cannot be considered simply as an added load, but rather as a structural component of the arch itself.

Figure 3.8 shows how the choice of the division of the arch was dictated by the desire to create a communication between each node of the Shell surfaces and the extremities of the volumetric elements, both for the purpose of congruence in the field of displacements, and to promote a continuity' geometric in the next phase of definition of the mesh.

In this phase it was decided to divide the intrados of the bridge from the rest, since in reality it is made of a different material, namely Istria stone, while the part resting on this arch is simply made of masonry and bedding sand with poor mechanical properties. In other words, the influence of the different properties of the material has been decisive in the geometric division of the volumes.

The wall partitions arranged along the z-axis, i.e. orthogonally to the longitudinal development of the bridge, have been outlined by means of surfaces passing through the middle line of the bricks, i.e. the volumetric sub-elements visible in Fig 3.4, and the thickness of each Shell element, subsequently requested by the Ansys software, has been calculated as a qualitative average of the thicknesses of the various brick elements. There is also an element that acts as a link between the transverse walls and the central vaults of the buildings used to house the shops, and whose thickness, density and elastic modulus were decided in a fictitious way.

Element	Thickness [m]
1	0.265
2	0.257
3	0.238
4	0.252
5	0.252
6	0.263
7	0.897
8	0.642
9	0.285
10	0.1
11	0.185
12	0.25
13	0.35

Table 3-1

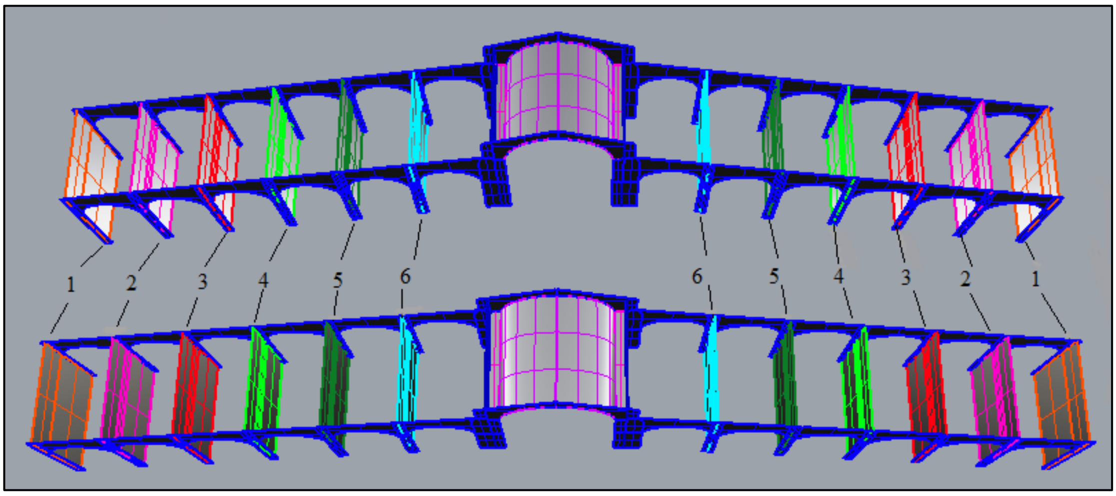


Figure 3-9 Shells numeration (1)

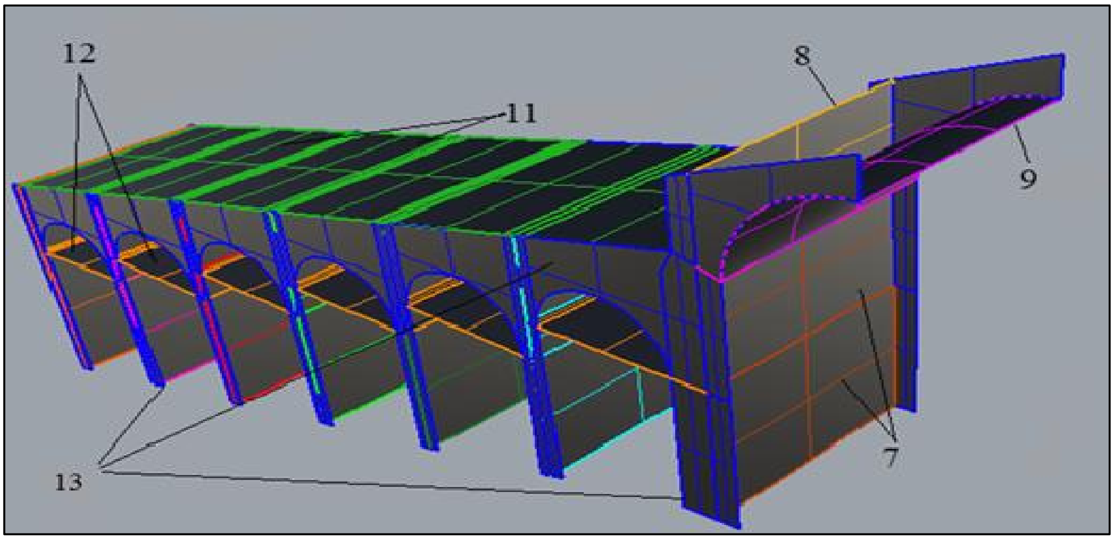


Figure 3-10 Shells numeration (2)

3.2 Material Properties

In the construction of the model some simplifying hypotheses have been adopted, in particular the masonry has been considered a continuous, homogeneous and isotropic material. A further simplification, inherent in the concept of homogenization, consists in the fact that it does not take into account also the precise variation of the properties of the masonry, found in historical masonry due to deterioration, degradation, voids, cracks: these aspects are summarized in a single value that governs the behavior of the macroelementThe masonry constituting the abutments of the bridge and the Istrian stone that forms the intrados of the arch were subjected to mechanical characterization tests, as already documented in the previous chapter (section 2.9).

With regard to the various samples extracted during core drilling, a good correlation between the values can be seen in general. It should be noted that the sample sizes do not meet the requirements of the Italian standards, so the results can be conditioned by their geometric configurations. [23]

The physical and mechanical data of the uniaxial compression tests carried out on the samples are given in Table 3.3 and 3.4, respectively one relating to the masonry constituting the shoulders of the bridge and the other to the Istrian stone.

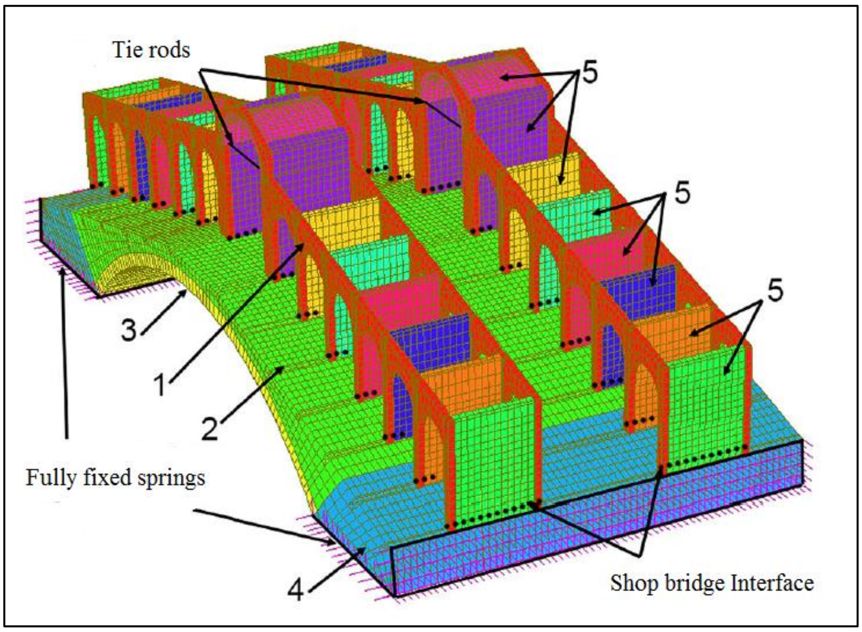
Abutment masonry						
Samples	Depth [m]	Diameter [m]	Volum [m^3]	Weight [kg]	Density [kg/m^3]	Average Density [kg/m^3]
1	0.180	0.078	0.00086	1.462	1699.79	1700
2	0.190	0.078	0.00091	1.588	1748.56	
3	0.175	0.078	0.00084	1.276	1525.93	
4	0.188	0.078	0.00090	1.662	1850.10	

Istrian Stone						
Samples	Depth [m]	Diameter [m]	Volum [m^3]	Weight [kg]	Density [kg/m^3]	Average Density [kg/m^3]
1	0.188	0.078	0.00090	2.394	2664.94	2650
2	0.170	0.078	0.00081	2.179	2682.44	
3	0.173	0.078	0.00083	2.169	2623.22	

Table 3-2

The mechanical parameters relative to the other structural elements visible in Fig 3.14 have been listed in table 3.4, and are the elastic modulus E, the shear modulus G and the Poisson modulus, respectively, in the different directions.

It was decided to use a model in which the Mechanical properties have been hypothesized as isotropic, also because experimentally the dynamic characteristics of a system vary in an almost negligible way, considering the structural elements that constitute them with orthotropic properties.



	E [MPa]	ν [-]	Density [kg/m^3]
1	33200	0.3	2400
2	4400	0.3	26
3	33200	0.3	2650
4	6000	0.3	1700
5	2500	0.3	1800

Table 3-3

The mechanical parameters assumed in the initial phase of creation of the model are shown in the Table 3.3, and the next phase of "modal updating" has begun from them to search for the optimal characteristics of the bridge for the calibration of the model (the hypothesis of orthotropic materials would be introduced later and only if the procedure of Modal Updating had not returned satisfactory results converging to the real physical model).

:A separate speech was made for the coverage of the superstructure hosting the shops and for the internal mezzanines. Below is the table containing the physical data for both constituents

	Load [kg/m^2]	Thickness [m]	Density [kg/m^3]
Covering	43	0.185	232
Loft	118	0.25	472

Table 3-4

The data used in the analysis of the loads were obtained from the technical report for the assessment of seismic risk [24], and are summarised in Table 3.6:

		[kg/m^2]
Covering	Lead Coating (1.5 mm)	17
	Wooden Planking (2 cm)	26
	Total	43

Loft	Wooden Planking (3 cm)	24
	Wooden Beams	64
	Suspended ceiling	30
	Total	118

Table 3-5

During the calibration phase of the model it was decided to consider these elements as non-structural, since due to their low rigidity a large number of local modal forms were born; therefore, to obtain the identification of the ways corresponding to the experimental ones and to avoid a huge computational burden, the roof and the loft of the shops were considered as concentrated masses at the top of the walls arranged along the x-axis.

3.3 Mesh

For the definition of the mesh it was necessary to first attribute to all the structural elements of the bridge the physical-mechanical characteristics, i.e. the thicknesses, the properties of the materials (density, elastic modulus and Poisson modulus) and finally the type of element (Shell in the case of the wall partitions or otherwise three-dimensional volume for the arch).

Subsequently, a distinction was made for the declaration of the mesh relative to the Shell elements characterizing the superstructure and the volumes constituting the arch: the first were meshed with the sole specification of the number of divisions on each side of the surfaces, while for the volumes it was decided to use a free mesh with triangular mesh. The Ansys calculation program offers both the possibility to construct the grid in a free way, in the way that it computationally considers most appropriate, and to construct a mapped grid.

Once the criteria for both structural elements had been established, a discourse that is usually defined in the literature as "mesh sensitivity" was addressed: Ansys software was asked to perform the modal analysis for the restitution of modal forms with the respective modal frequencies, and this analysis served to analyze the variation of results as the mesh size varied.

In Figure 3.13 below, the values of the modal frequencies for the first ten ways of vibrating the structure as a whole have been diagrammed; in this diagram, the modal frequencies are shown on the axis of the ordinates, while on the axis of the abscissae, the variable is the number of divisions of one linear metre of elements during the construction of the mesh, starting from the unit value up to a decidedly reduced value (i.e. 0.4).

The Mesh Sensitivity discourse consists in identifying the mesh size such that the response of the structural model (in this case the value of the modal frequencies) is not significantly influenced by a further decrease in the mesh. The diagram shows that the functions representative of the modal frequencies are qualitatively convergent for a mesh size value of 0.4 and experimentally it has been verified that for still inferior dimensions the benefits related to a more refined mesh are not so evident as to justify a further computational burden of the model, which in operational terms means longer calculation times required to obtain the results of the modal analysis.

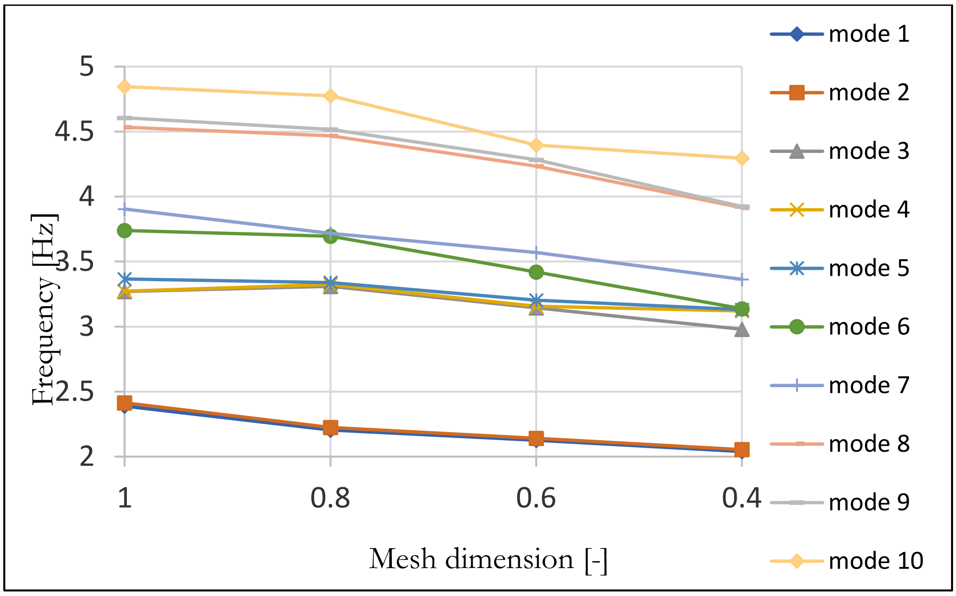


Figure 3-11 Mesh sensitivity Diagram

The result of the attribution of the mesh to the Fem model of the Rialto Bridge graphically returns the following image:

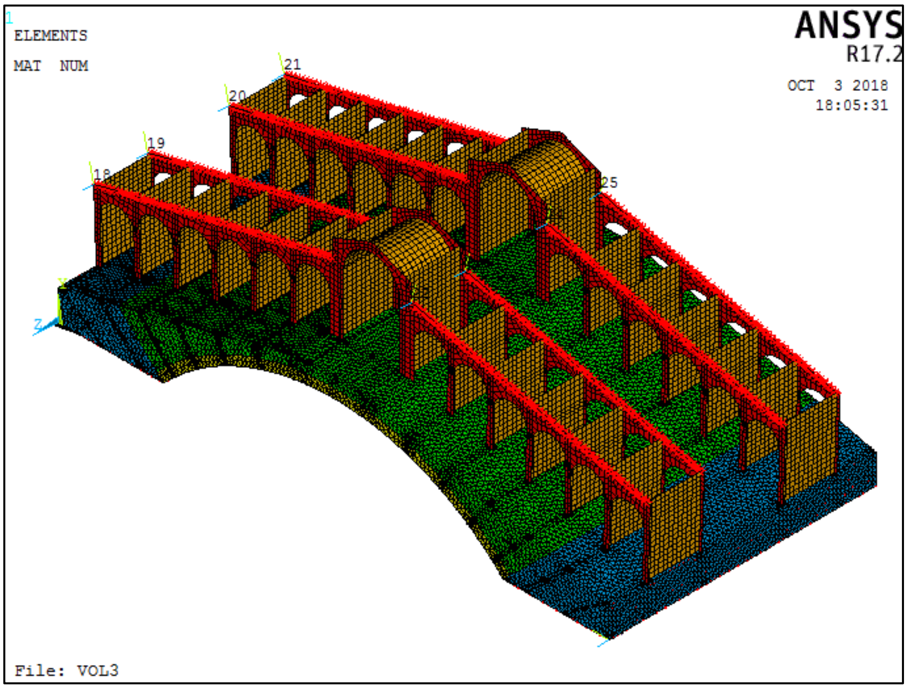


Figure 3-12 Meshed model

3.4 Creation of surface loads

The arch of the Rialto bridge carries permanent loads associated with the presence of the paving, consisting of a layer of sandstone slabs and a thickness of bedding sand and heterogeneous aggregates. These permanent loads were modelled through the introduction of concentrated masses, evenly distributed over the extrados surface of the bridge, since the modal analysis carried out with the help of the Ansys Mechanical APDL calculation software does not recognize the gravitational surface loads during the calculation of the modal forms and corresponding frequencies; the input elements of the analysis are in fact the matrix of the masses [M] and the matrices of the rigidity [K].
In addition to the permanent loads associated with the paving, the weight of the Istrian stone balustrade, which runs along the north and south ends of the bridge throughout its longitudinal development, was also taken into account.
Therefore, these loads, treated separately as they derive from the presence of materials with different specific weights and distribution, were positioned by dividing the total weight into many concentrated masses at the knots of the mesh created in the previous phase.

3.5 Definition of boundary conditions

A description of the high complexity of the foundations of the structure has been introduced in the previous chapter: the bridge rests on wooden poles connected to each other by a sturdy wooden planking, and on this rest the pylons made of Istrian stone.
It was therefore decided to simply avoid modelling the boundary conditions, and to simulate an interlocking condition both at the level of the foundations and at the sides of the shoulders of the bridge, springs with a very high stiffness were introduced into the Ansys software.
To understand the influence of the boundary condition on the results of the model, it was decided to study the evolution of the modal frequencies for the first modes of the structure calculated in the Ansys environment, which basically involve the buildings containing the shops.

These springs are schematically described as COMBIN14 elements: the longitudinal spring-shock absorber is an element able to undergo mono-axial compression and traction, and it is possible to apply these more elements of this type for each node, up to a maximum of three degrees of freedom: translations in the x, y and z directions of the node. No bending or torsion is considered and this spring-shock element has no mass.

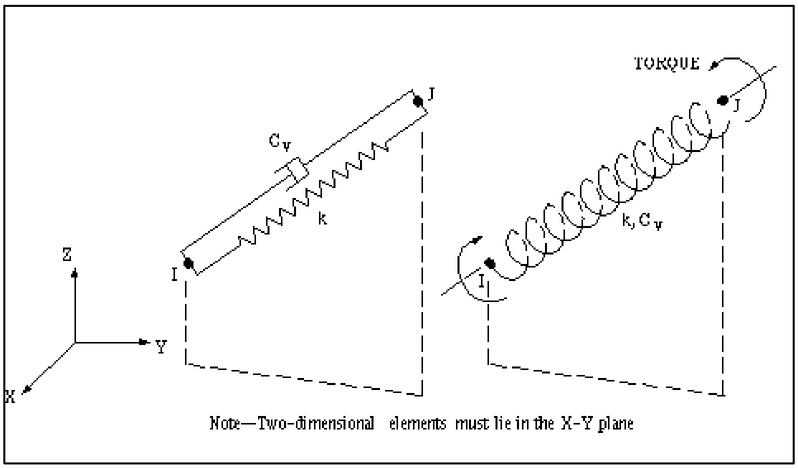


Figure 3-13 COMB14, Spring element

3.6 Modal Analys

Operationally, all the phases described, from the introduction of the geometry to the definition of mesh, boundary conditions and load-carrying capacities, have taken place through the writing of a file.txt, which once inserted into the toolbar of the finite element software allows to perform the modal analysis for the structure in question.
Modal analysis is a technique used to determine the dynamic vibration characteristics of a structure; the dynamic behaviour is identified by the frequency of vibration and by the value of the modal shifts at points of interest (in particular at the points of installation of the sensors on the Rialto Bridge).
In order to determine the modal forms, a problem is solved by means of eigenvalues and motor vehicles. Starting, in fact, from the fundamental equation of motion:

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = \{F(t)\} \tag{1}$$

That in the absence of external forcing (free oscillations) and zero damping becomes:

$$[M]\{\ddot{u}\} + [K]\{u\} = \{0\} \tag{2}$$

And searching for a solution of harmonic motion, that is, by placing:

$$\{u\} = \{U\}\, sin(\omega t) \tag{3}$$

You get:

$$([K] - \omega^2[M])\{u\} = \{0\} \tag{4}$$

The roots of this equation are the eigenvalues ω^2, whose number is equal to the degrees of freedom of the system and represent the frequencies at less than a factor equal to 2π, while the vector {u} is constituted by the autoceptors that identify the modal forms.
Therefore, modal analysis means the resolution of a problem with the car carriers. It is also necessary to clarify that in the case study the number of degrees of freedom of the system coincides with the number of points where the experimental observation channels have been installed.
For the extraction of the cars we chose the method of Block Lanczos, recommended for its versatility in different applications, and that lends itself in a satisfactory way to model works with shell, beam and solid components.
To correctly evaluate the modal forms of the work under examination, it would be necessary to extract a number equal to the degrees of freedom; this condition would imply analyzing a large number of modal forms, a procedure not feasible in practice.
To view the results, you switch to the ANSYS graphic interface.
Here you can choose from several options including the display of the movements combined with the "contour", which are nothing more than the colors that highlight the areas with minimum displacement, in blue, from the areas with maximum displacement, in red. For the intermediate zones there are a whole series of shades.
In addition to a static visualization it is possible to animate the solution and this clearly represents a great help to distinguish the type of modal deformata.
We now move on to a representation of the modal deforms with regard to the first four global modes of the superstructure, and the mode related to the arc.

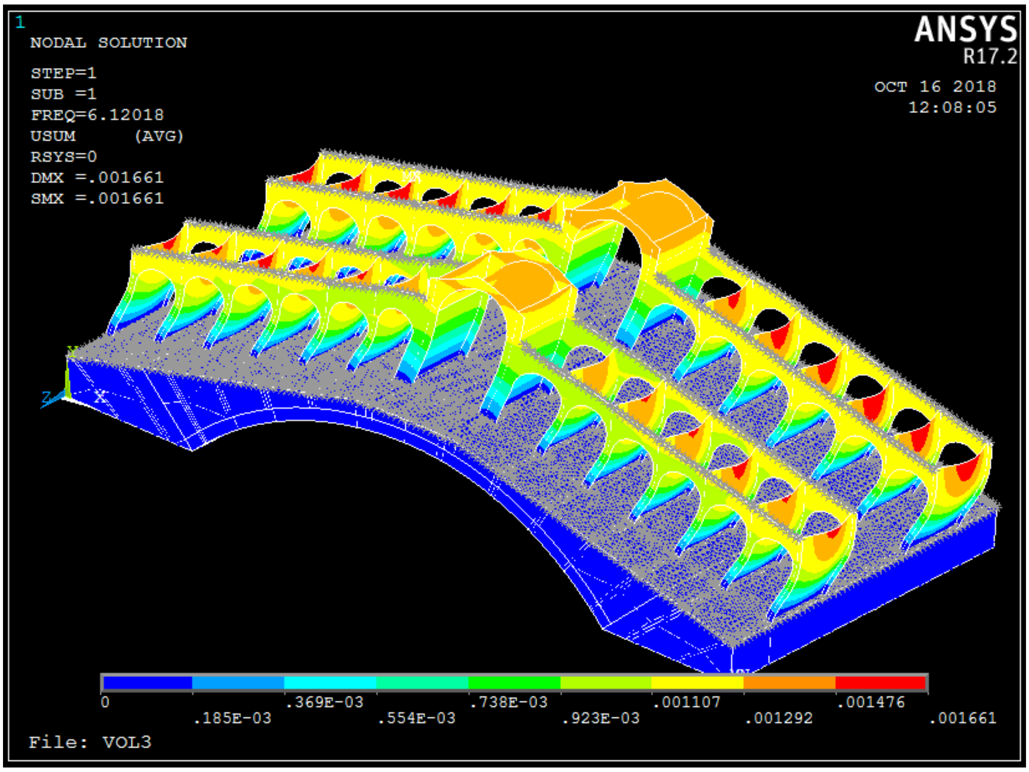


Figure 3-14 First mode

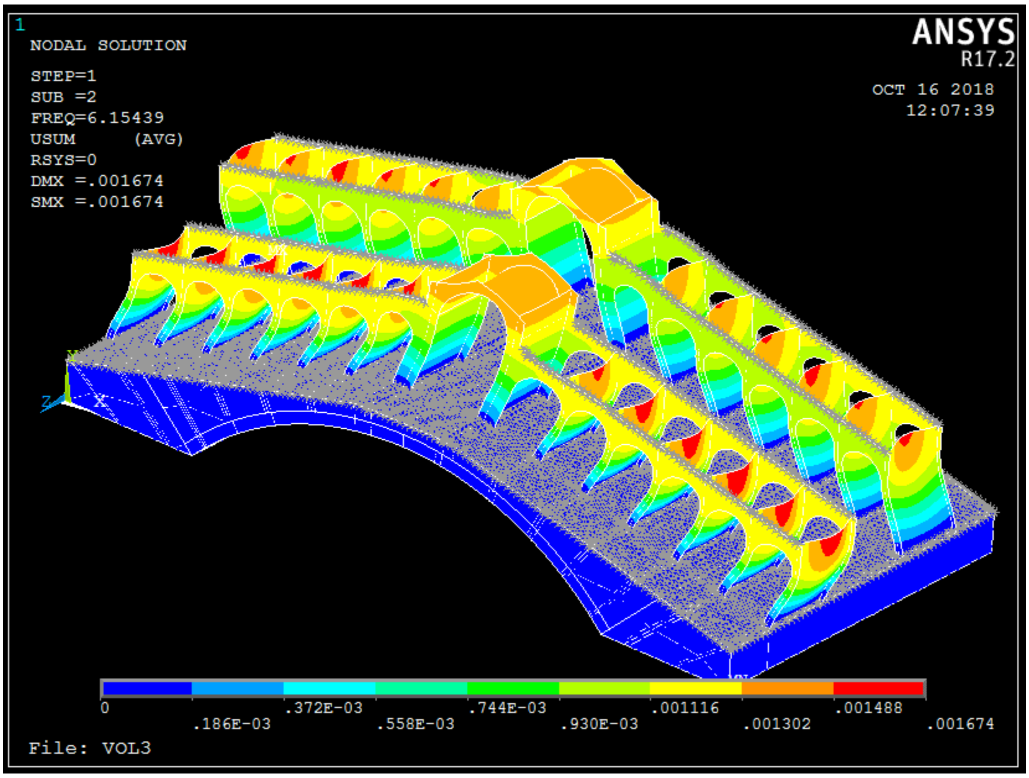


Figure 3-15 Second Mode

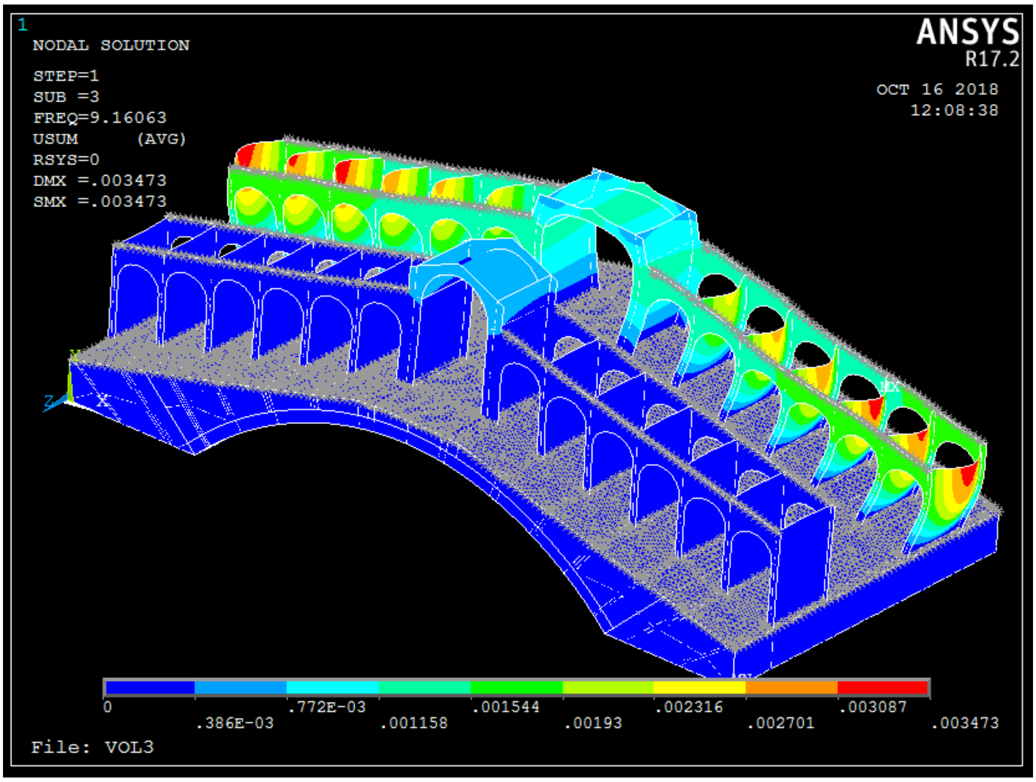


Figure 3-16 Third mode

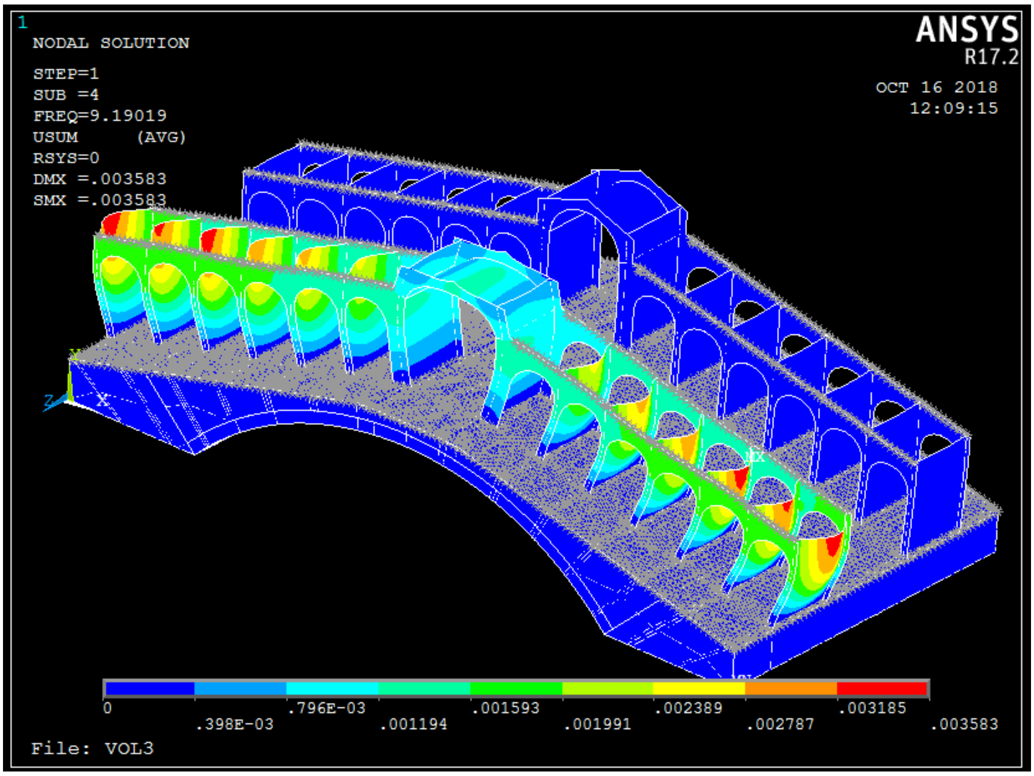


Figure 3-17 Fourth mode

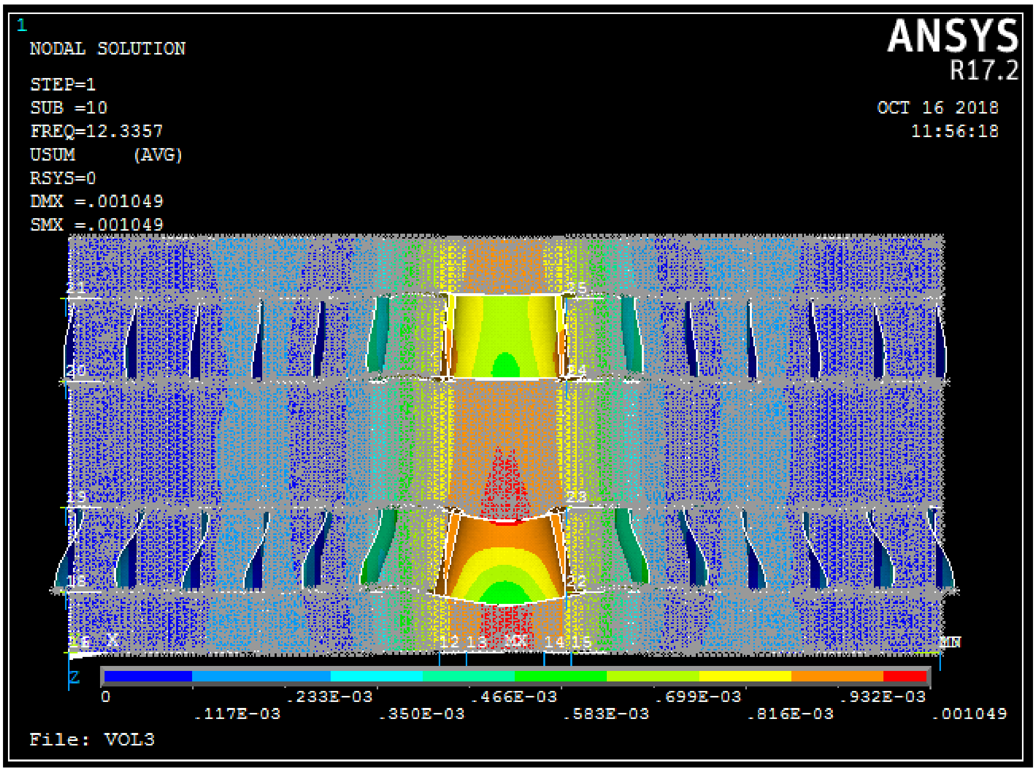


Figure 3-18 Arch mode

Chapter 4

4 Model Updating

The procedure of the Model Updating sets as objective the improvement of the performances of the structures of the civil engineering, but it finds application also in other fields of the mechanics.
With the progress of the modern instruments of study of the structures we have come across the real problem of a too high number of parameters to manage and therefore in an always greater complexity of the solutions to search: the mathematical models used equations of always higher order, which allowed to identify behaviors that the simplest systems were not able to appreciate. In this scenario, the Model Updating technique aims to improve the behavior of the model and eventually correct errors. The main application of the model updating is the identification of the model of a structure, correlating the results deriving from the elaboration with those obtained from experimental tests, essentially dynamic tests.
The first phase of the calibration process is the definition of adequate parameters of the model: in fact the purpose of the study is not simply to create the best concordance of the "numerical" system with the experimental data, but to go to deepen the physical meaning of the structure, and therefore to try to know with greater precision some characteristics of the structure, such as the mass or the rigidity of the constituent elements. With this approach models with a strong physical meaning are obtained and moreover it can be possible to detect eventual defects or damages in the real structure; if instead the parameters assume not plausible values this indicates an error in the acquisition of the data or in the definition of the model.
The Model Updating can also not bring to completely satisfactory results in the case in which there is incompleteness or imprecision in the survey of the data or to a wrong choice of the model of reference.

Model Updating techniques can be divided into two main categories:
- direct methods, which correct the initial model in a single step;
- indirect methods, which correct the initial model with an iterative calculation and the parameters are updated at each step.

The techniques sometimes present a solution that is not unique or even existing.
To this is added the possibility of errors in the choice of the parameters, in the definition of the boundary conditions or in the discretization of complex systems and the possibility that the equations associated to the model do not simulate sufficiently well the physical phenomenon.
To implement an algorithm of Model Updating is necessary:
- generate a numerical model of the system to analyze, that is a Finite Element Model, (FEM);
- choose the parameters of the Updating object;
- choice of a suitable "objective function", that allows to represent in synthetic way the correlation between measured parameters and parameters of the model;

The Model Updating makes a comparison between experimental data and data deriving from the FEM model, it is therefore of fundamental importance to use a synthetic method and computationally cheap to describe the correlation between the models being compared.
In order to measure the correspondence between the data, the most widely used index is the MAC (Modal Assurance Criterion), which can be expressed in its most general form with the following relation:

$$MAC_{jk} = \frac{(\{\Phi_m\}_j{}^T\{\Phi_a\}_k)^2}{(\{\Phi_a\}_k{}^T\{\Phi_a\}_k)(\{\Phi_m\}_j{}^T\{\Phi_m\}_j)}$$

Where:

- $\{\Phi_m\}_j$ is the eigenvector measured by experimental measurements, corresponding to the j-th mode;
- $\{\Phi_a\}_k$ is the theoretical eigenvector corresponding to the k-th mode.

The MAC can vary between 0 and 1, depending on the congruence between the theoretical and the measured vectors: it is cancelled if the two compared cars are completely unrelated and it is worth 1 in the case of total correlation. The comparison can be considered satisfied for a value higher than 0.8.
At each iteration the method updates only the chosen parameters, and the calibration is optimized looking for the minimum of a cost function; this cost function has as its object the modal forms and eigenvalues of the problem, i.e. the modal frequencies, and is the sum of the squares of the differences between estimated and measured data.

The advantage of the method consists in the possibility to choose a large number of parameters to be calibrated and to establish which weight to give to the single information. If you decide to opt for an indirect Model Updating, i.e. with an iterative calculation, you need to minimize a cost function, and the methods of finding the minimum of the function are the so-called optimization techniques;
In the case in which the model is characterized from many variables the process is complicated a lot because it is operated with a function of cost with the presence of relative minimums; the computational time can therefore become unacceptable and the solution not unique.

4.1 Model updating procedure

The calibration process took place by having the Matlab and Ansys software work simultaneously and in a coordinated manner.
The process can be outlined as follows:

- The starting values of the parameters to be calibrated are declared on the Matlab software;
- You choose the range of variation of the parameters, so that these parameters assume in any case physically possible values;
- Matlab launches the analysis in the Ansys environment and, with the parameters chosen in each iteration, orders the calculation of the solution;
- The Ansys software returns the frequencies of the model modes with the parameters assigned in the iteration and to the corresponding eigenvector, calculated in the position where the sensors are installed in the real structure;
- An optimization algorithm works to minimize the difference of modal response between the numerical model and the real one, making to vary in the successive iterations the variable parameters in the established range; such algorithm returns to every step the value of the function of error, that is graphicized from Matlab.
- The obtained results are evaluated, and eventually the starting variables or the variation ranges are changed, making the process restart.

For the iterative calculation of the best correlation the algorithm "particle swarm optimization" has been used, an optimization algorithm that belongs to a particular class

of algorithms used in different fields, including artificial intelligence. It is a heuristic method of research and optimization, inspired by the movement of swarms. At each iteration, the algorithm identifies a new "candidate for the best" in the research space, based on a specific quality measure (fitness). It does not make any assumptions about the problem and allows the exploration of very large spaces of solutions. However, as the algorithm is structured, there is no guarantee that the best solution will ever be found.
The algorithm does not make use of a gradient during optimization, so it is not required the differentiability of the problem to be analyzed, which instead happens in traditional optimization methods such as the descent of the gradient. For this reason, it can be successfully used in irregular, noisy, time-varying, etc. optimization problems. [25]
The analytical expression of the error function is:

$$e = \sum_j \alpha \left(\frac{f_{FEM,j} - f_{ID,j}}{f_{ID,j}} \right)^2 + \beta \left(1 - MAC(\phi_{FEM,j}, \phi_{ID,j}) \right)^2$$

Where:

$f_{FEM,j}$: frequency of the j-th numeric vibrate mode;

$f_{ID,j}$: Frequency of the j-th identified mode;

$\phi_{FEM,j}$: modal deformation of the j-th numeric vibrate mode;

$\phi_{ID,j}$: modal deformity of the j-th identified mode;

α : weight of the frequency deviation

β : weight of the scrap in shape

4.2 Monitoring system for the Rialto Bridge

Experimental tests for dynamic identification included checking the structure of the bridge and the shops above it to best calibrate the entire distribution of masses and rigidity of the structure. This control was carried out by means of accelerometric sensors placed on different points of the structure, in order to monitor the response of the different components.

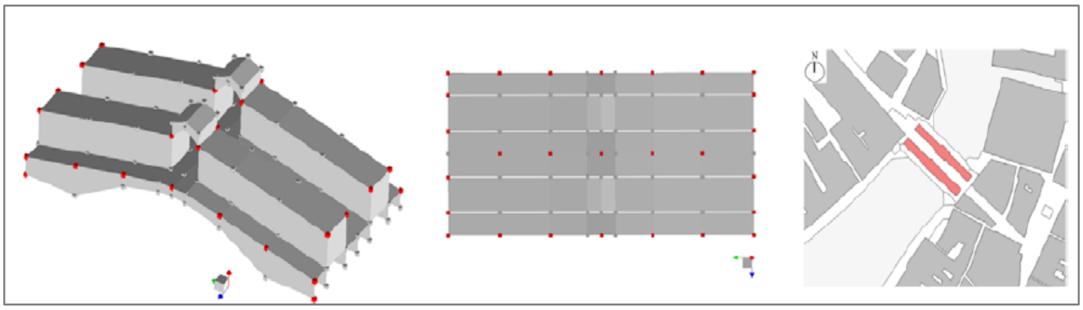


Figure 4-1 Positioning of the sensors

The vibrational measurements of the bridge structure were carried out using 21 accelerometric channels positioned at the extrados and at the bridge shutters as visible in the following figure:

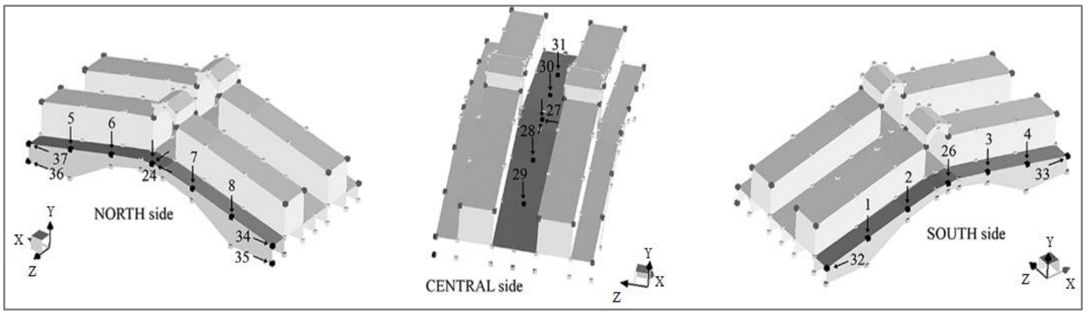


Figure 4-2 Arch sensors

The extrados sensors mainly control the y-direction and two of them positioned in the middle (sensors 24 and 27) also control the x- and z-directions. Sensors located at the bridge arch shutters only control the x-direction.
The shop structures were controlled by 32 accelerometers positioned at the top of the corners of the main four volumes (all corners were controlled in both the x and y directions), as visible in Figure 4.3.

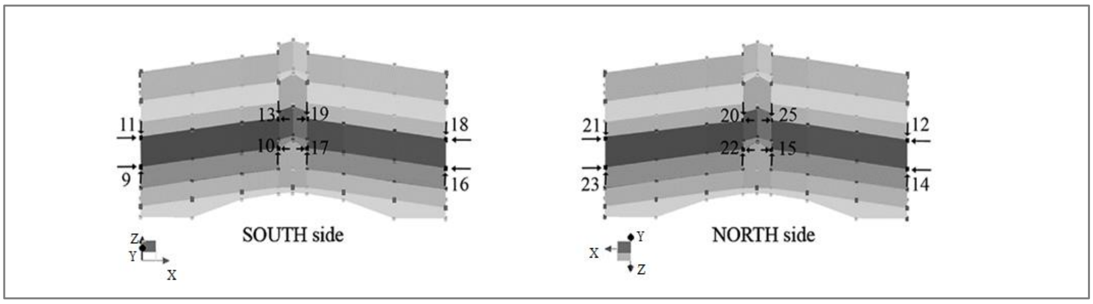


Figure 4-3 Shops sensors

4.3 Operational Modal Analysis

The dynamic monitoring has used as natural sources the pedestrian traffic, the wind and the shakes induced by the waves caused by the boats in the Grand Canal, with recordings of the duration of about 30 'each. The type of analysis adopted that takes into account only the environmental vibration, and is therefore defined as OMA Operational Modal Analysis.

Thanks to these tests it is possible to check the vibration response of the global structure or of the sub-parts.

The vibrational survey was carried out by digitizing the signals coming from the accelerometers at a frequency of 600 Hz/channel. In the case of excitation from ambient noise, signals were acquired for a total duration of about 8488 seconds.

The signal-to-noise ratio (S/N) expressed in decibels in time domain recordings is acceptable if S/N>10 dB, while it needs to be filtered when 6<S/N<10 dB. The reliability of the signals is demonstrated by the less acceptable acquisition, which in the case under analysis has a ratio S/N = 21.3 Db.

The signal was then treated to obtain more reliable results: the normalization of the amplitudes of the recorded accelerations was performed, taking into account the sensitivities of the sensors used, in order to express their amplitudes in g; each signal was subdivided into 2048 points sub signals, with mutual overlap of 66%. Subsequently, a frequency analysis was carried out.

4.3.1 Identification results

In order to correctly evaluate the modal forms of the structure under examination, it would be necessary to extract a number of modal forms equal to the degrees of freedom;

this condition would imply analysing a too high number of modal forms, a procedure not possible in practice.

For this reason, the extraction of modal shapes is done by means of dynamic identification, a procedure that utilizes the acquisitions of accelerometers placed on the bridge.

Dynamic identification uses the stabilization diagram as an operational tool: in the following images are visible the diagrams relating to the arch of the bridge and the superstructure containing the shops.

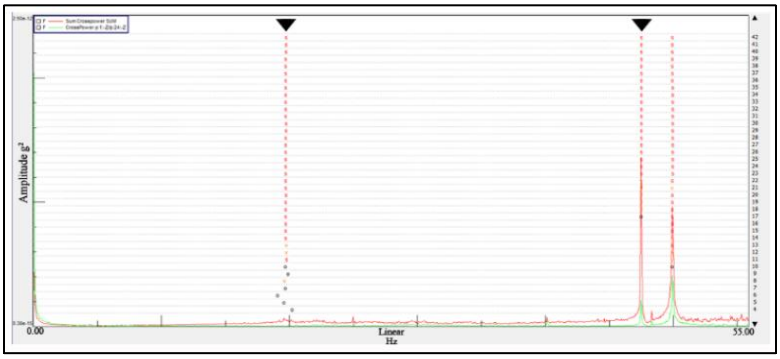


Figure 4-4 Bridge Arch Stabilization Diagram

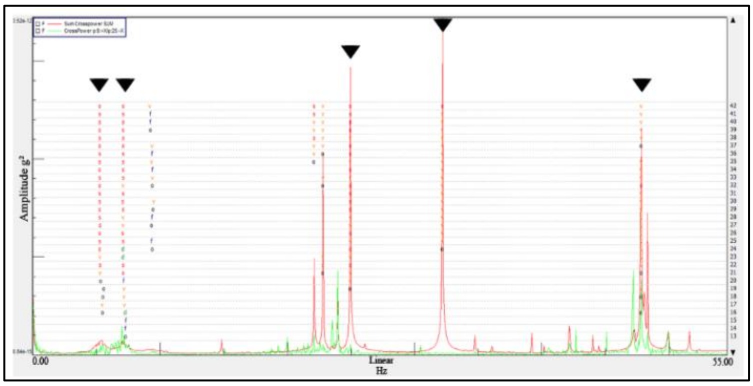


Figure 4-5 Bridge Shops Structure Stabilization Diagram

The structural sub-parts were then the subject of a separate study in the identification phase: during the data acquisition campaign, instead of integrating the information simultaneously from all the accelerometers, the operators decided to study the structural response to natural excitations in a separate way; during a part of the recordings, the accelerometers positioned on the corners of the superstructure hosting the shops were active, while during others only the sensors positioned on the extrados of the arch and at the bridge shutters were active.

In the above diagrams, the peaks corresponding to the modes of the bridge are visible, in particular five for the superstructure and three for the arch.
Among the modes identified, with regard to the shops, only the first four were taken into consideration, which in the stabilisation diagram coincide with the first two initial peaks. In fact, the PolyMAX software, used by the operators who extracted the modal forms, does not make distinctions between modes with very similar frequency, and given the geometric and structural symmetry of the superstructure, the first modes of vibrating involving two rows of shops in a distinct way have very similar frequency.
With regard to the arc, only one modal form has been chosen to calibrate the FEM model, with data available for this frequency only.
The results concerning the shops structure are shown in Appendix A, istead that concerning the arch of the bridge are shown in Appendix B.
The main features are summarised in the following table:

Frequency [Hz]	Damping [%]	Localization
5.27	2.23	Shops- North side
5.45	2.7	Shops- South side
7.08	2.64	Shops- North side
7.76	0.91	Shops- South side
19.75	2.23	Arch
48.2	0.02	Bridge Shutters

Table 6

The modal deforms have been graphized on a three-dimensional model schematizing the structure under examination; in order to present intelligible modal deformed, the displacements of some nodes have been calculated by means of a linear combination of the displacements of the instrumented nodes with the accelerometric directions.

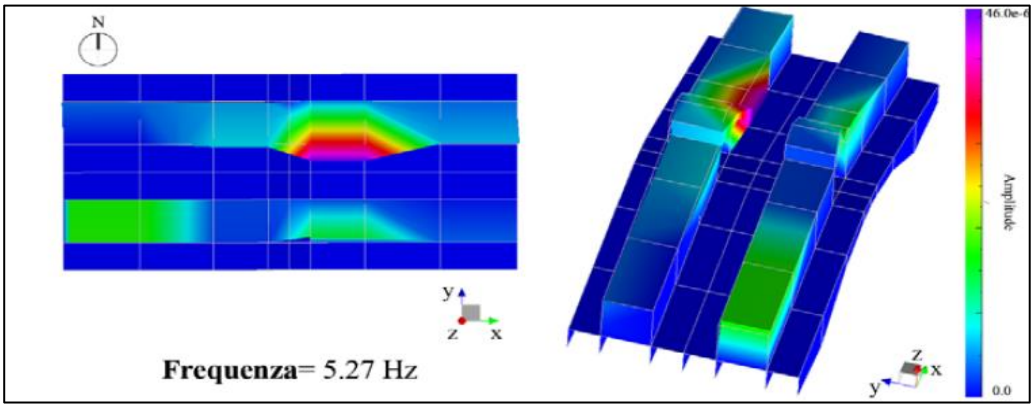


Figure 4-6 First sperimental mode

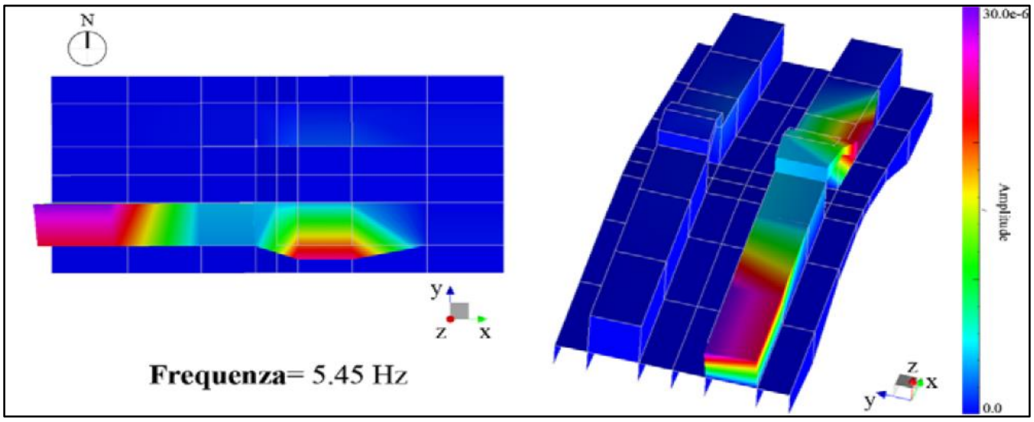


Figure 4-7 Second sperimental mode

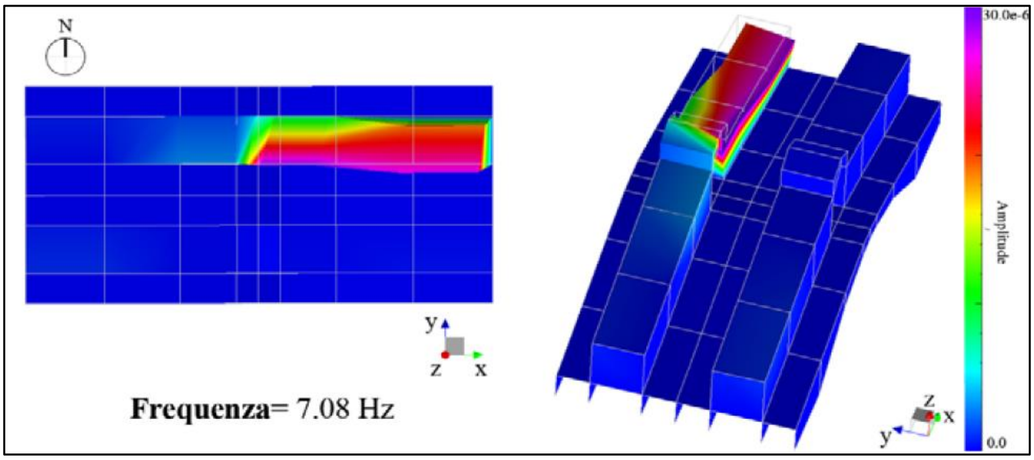


Figure 4-8 Third sperimental mode

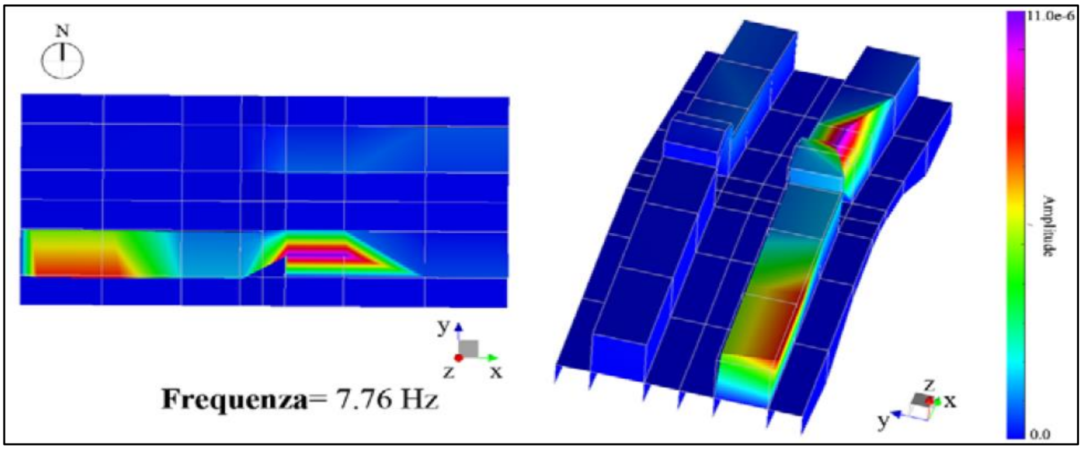


Figure 4-9 Fourth sperimental mode

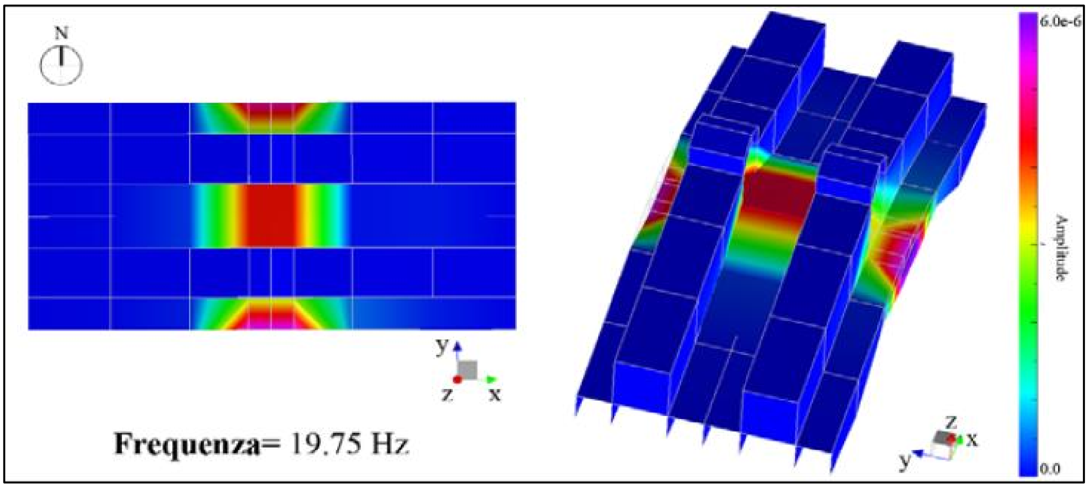


Figure 4-10 Arch sperimental mode

For the sake of completeness, we also report the modal forms not considered, with higher modal frequencies.

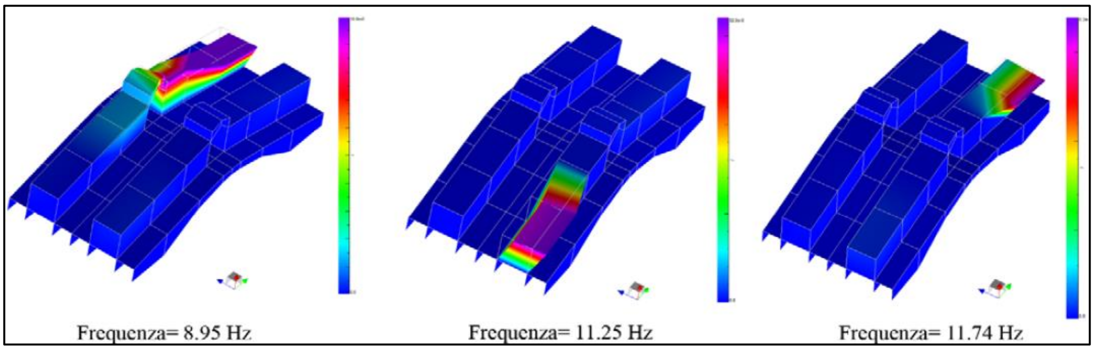


Figure 4-11 Local sperimental mode

With respect to the experimental results we can draw the following conclusions:

- as shown in table 7.1, the first frequencies of shop buildings can be inserted in the range 5-8Hz.
- The fundamental frequency of the bridge, 19.75Hz, highlights the structural non-interaction between the bridge structure and the shop buildings;
- The bridge's vibration modes are predominantly flexural;
- The first and second way of vibrating the structure of the shops shows the lower rigidity of the north side;
- the dissipative capacity expressed by the damping coefficient is within the range of 2.20%-2.70% for both the bridge and the shop buildings;

4.4 Results

4.4.1 Arch Results

During the updating phase of the model, several attempts were made to change the parameters, concluding that the significant parameters for the calibration of the arc are the modules of elasticity of the materials making up the arc itself (in fact, the results are influenced in a negligible way by variations in stiffness on the boundary conditions and on the elements of the superstructure), also in an attempt to remain faithful to the available information, it was decided to impose a very limited range of variation to the parameters that have been tested in the laboratory.

Therefore the most important uncertainty in the model remains on the knowledge of the filling masonry, and being this a significant portion of the structure in terms of volume, the response and therefore the goodness of the results is significantly linked to this component of the arc.

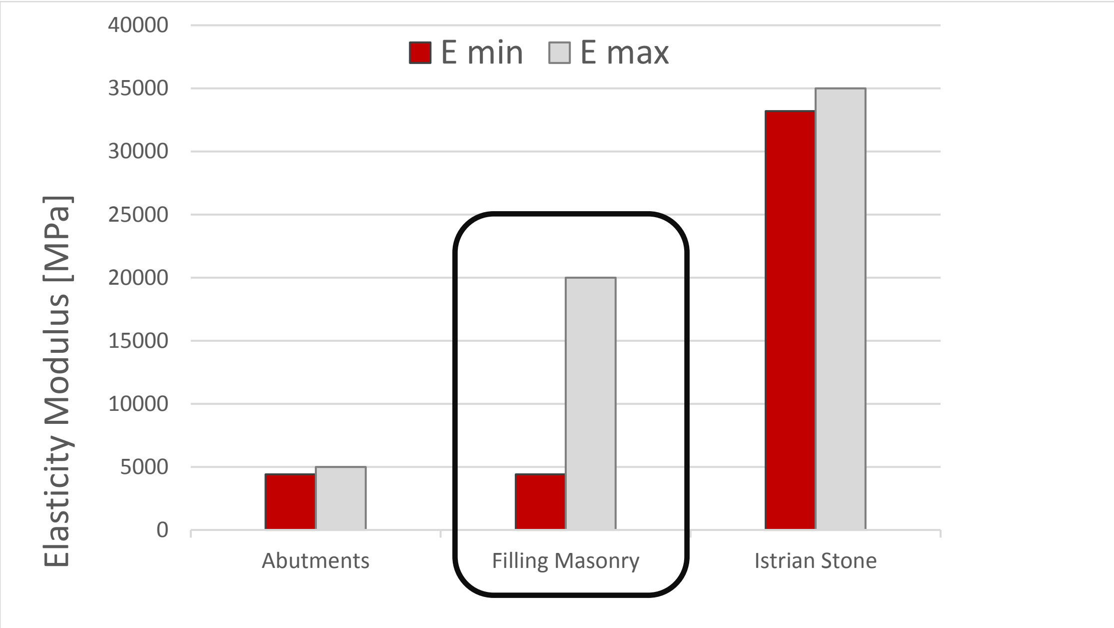
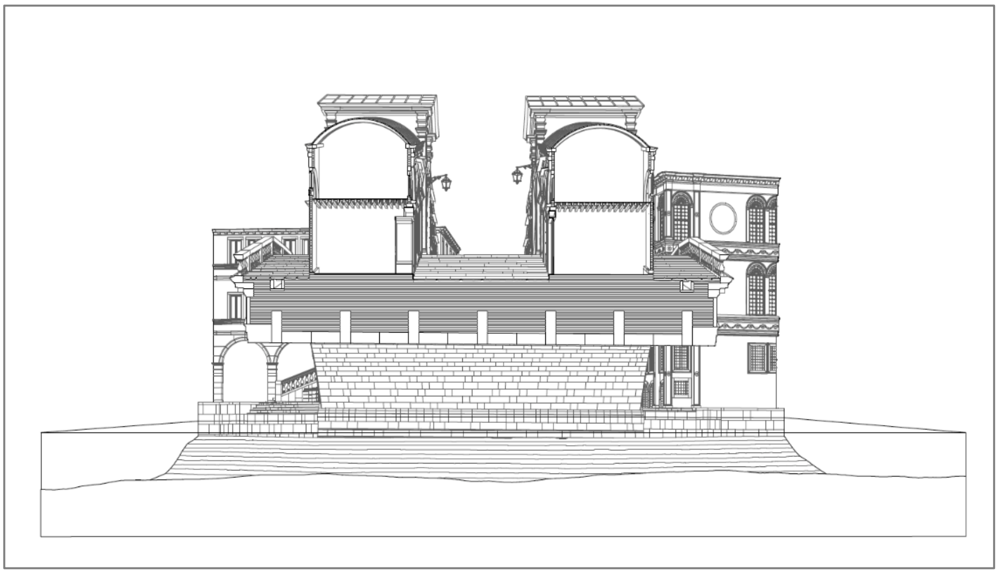
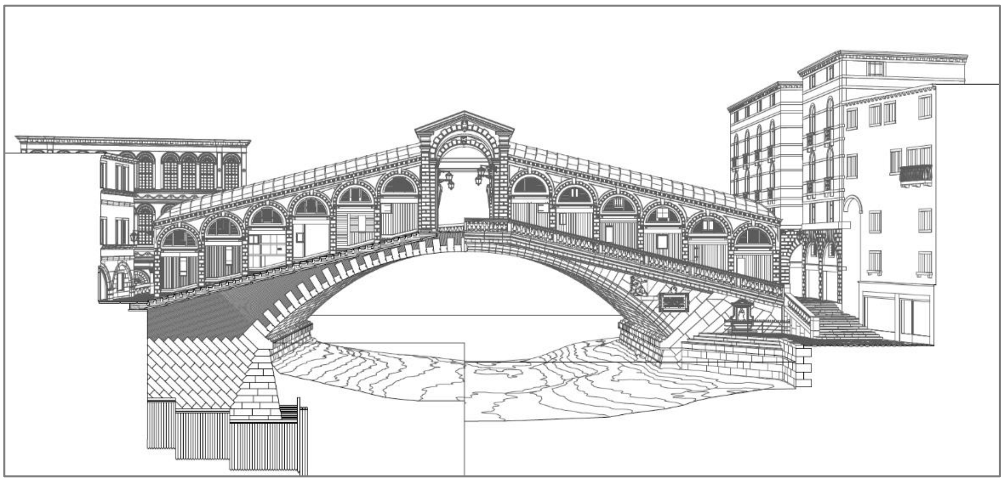


Figure 4-12 Range for arc parameters

The modal shape corresponding to the frequency identified for the arc shows a flexural behaviour of the deck, as shown previously.

The model shows this flexural mode at a significantly lower frequency before the Updating (12.33 Hz), underlining that the numerical model was less rigid than the real structure, and this rigidity was justified by the presence of the Istria stone teeth visible in the figure, which improve the grip between the stone intrados and the filling masonry.





This construction peculiarity was not geometrically modelled due to lack of time, but an equivalent modelling was used, considerably widening the range of variation of the elastic modulus of the filling masonry, as shown in the histogram, and also the relative density of the material, but even after this attempt the increase in modal frequency is not sufficient to justify the difference with that identified.

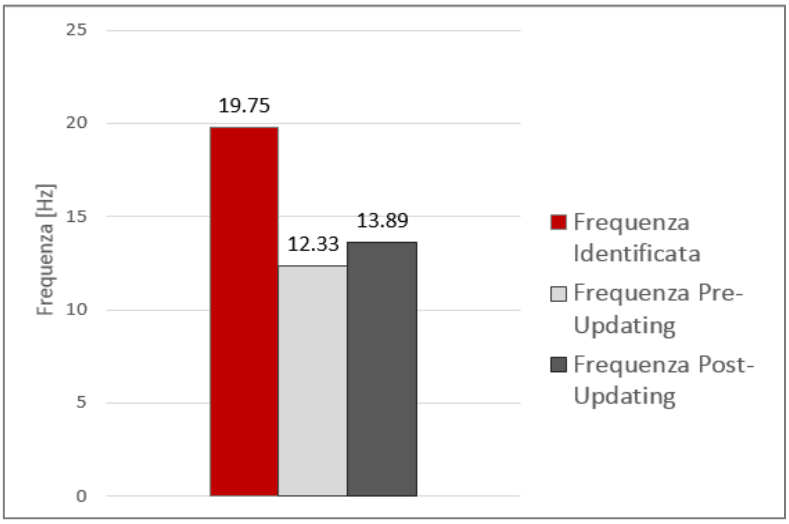
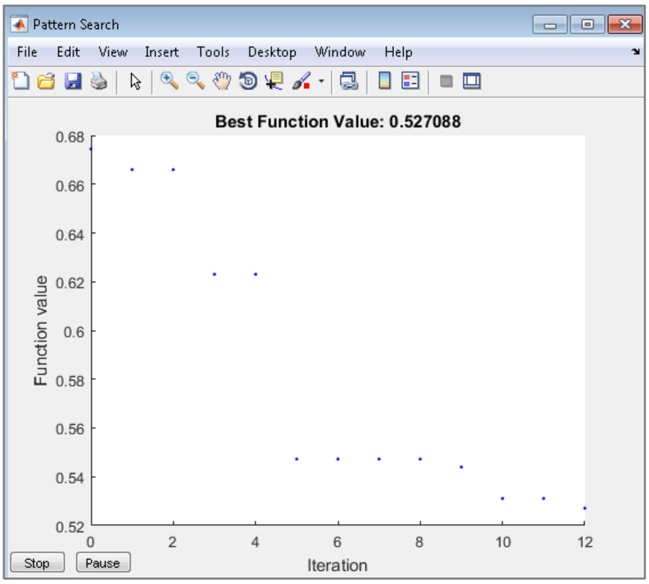


Figure 4-13 Modal Frequency Pre and Post Updating

The graph below shows the trend of the "Fitness" function during the first generations:



It can be seen that the Best Function value is considerable in the first iteration and that it tends to gradually decrease with the progression of the generation while the average value tends to settle already after the tenth iteration.

Therefore, in the model there are still some factors of uncertainty that may be investigated in future thesis work.

	Pre Updating	Post Updating
MAC	0.46	0.60

4.4.2 Shops Results

When sensors are arranged and used during the registration campaigns for the dynamic identification of a structure, if different sensors are used during the acquisitions, it is necessary to have in each set up common sensors that allow to join the modal forms, in order to study the structures globally, while in the case under analysis there were two disconnected and local set ups available.

The modal forms related to the shops were recorded by a series of accelerometric sensors, visible in the figure, and the basic problem was the fact of not having available reference channels in common between the acquisition set ups, so the vector of modal shifts available for all five modes is always partial, as visible in Annexes A and B.

Having available the shapes in the complex, therefore on all the channels it would have been possible to calibrate the model in a global way, and the phase of Updating would have been much simpler, in fact the modal shapes of the shops and those relative to the arc not resulting scaled between them.

In addition, the calibration of the upper structure used to house the shops, and 'was addressed in the knowledge that it can not achieve an adequate correlation between the numerical model and the actual structure, also for the following two reasons:

- The lack of information on the cracking state of the masonry;
- The impossibility of establishing the type of connection between the walls.

The calibration of the superstructure is particularly complex, due to the ascending arrangement of the partitions and the lack of a rigid plane that ensures a diaphragmatic behavior.

The wooden mezzanines and the roofs of the buildings that house the shops do not guarantee box-like behaviour, and this is demonstrated by the local character of the modal shifts identified.

Despite all the difficulties just described to ensure a general calibration, it was decided to carry out a manual calibration to approach the values of modal frequencies recorded and calculated.

It is necessary to underline that the attribution of an elastic modulus to the wall partitions presupposes the fact of considering the masonry as a continuous, homogeneous and isotropic material, therefore, in order to take into account the cracked conditions, it was decided to reduce the wall stiffness.

The results of the calibration are summarized in the following diagrams:

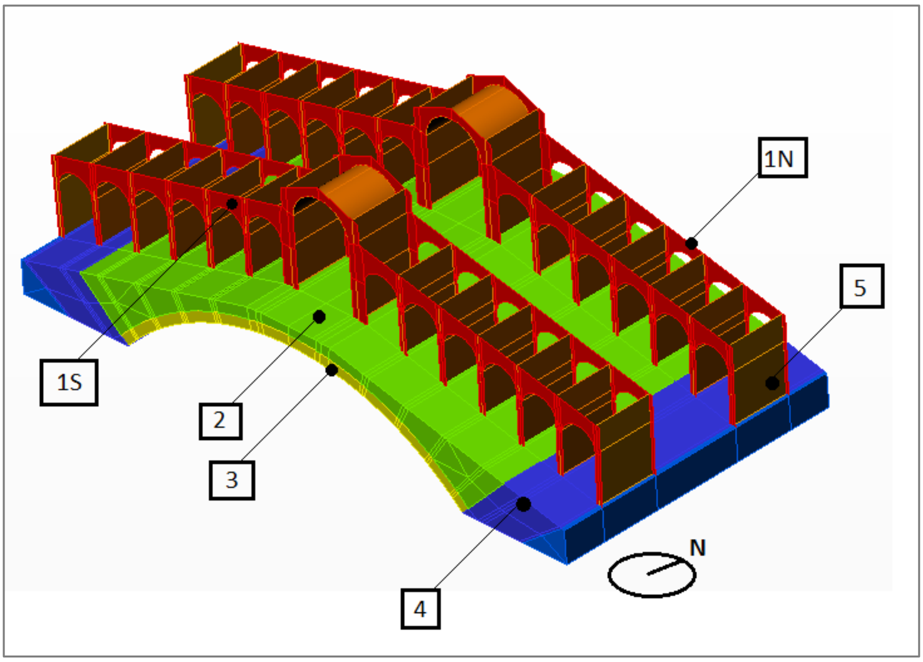


Figure 4-14 Elements numeration

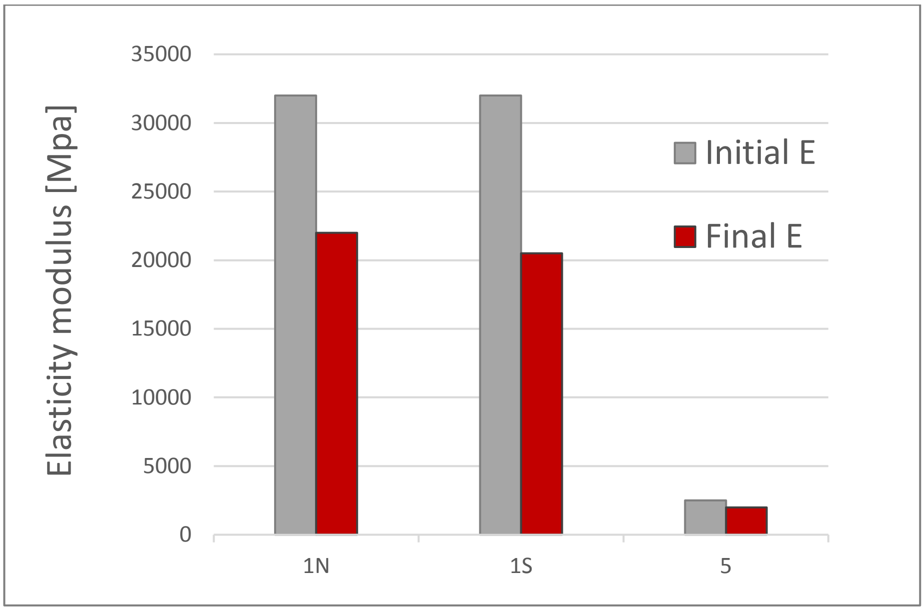


Figure 4-15 Updating for Shops parameters

The elasticity modules of the wall partitions made of blocks of Istrian stone, i.e. the partitions that develop along the longitudinal axis of the bridge, have been differentiated for the buildings located along the north and south sides, and both have been demolished by about a third of the hypothesized value, as shown in the diagram above.

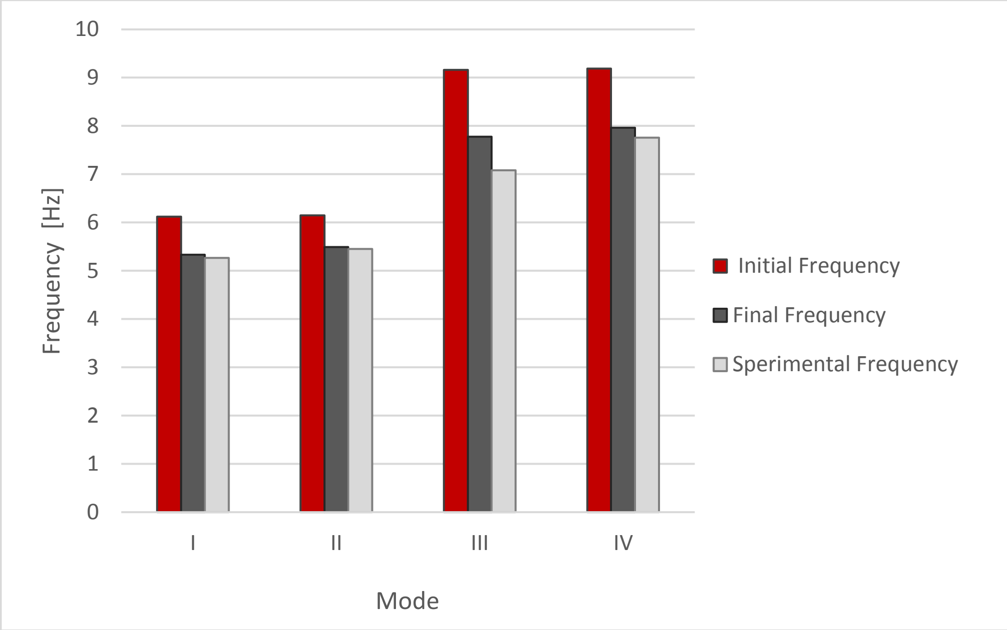


Figure 4-16 Shops Frequency modes

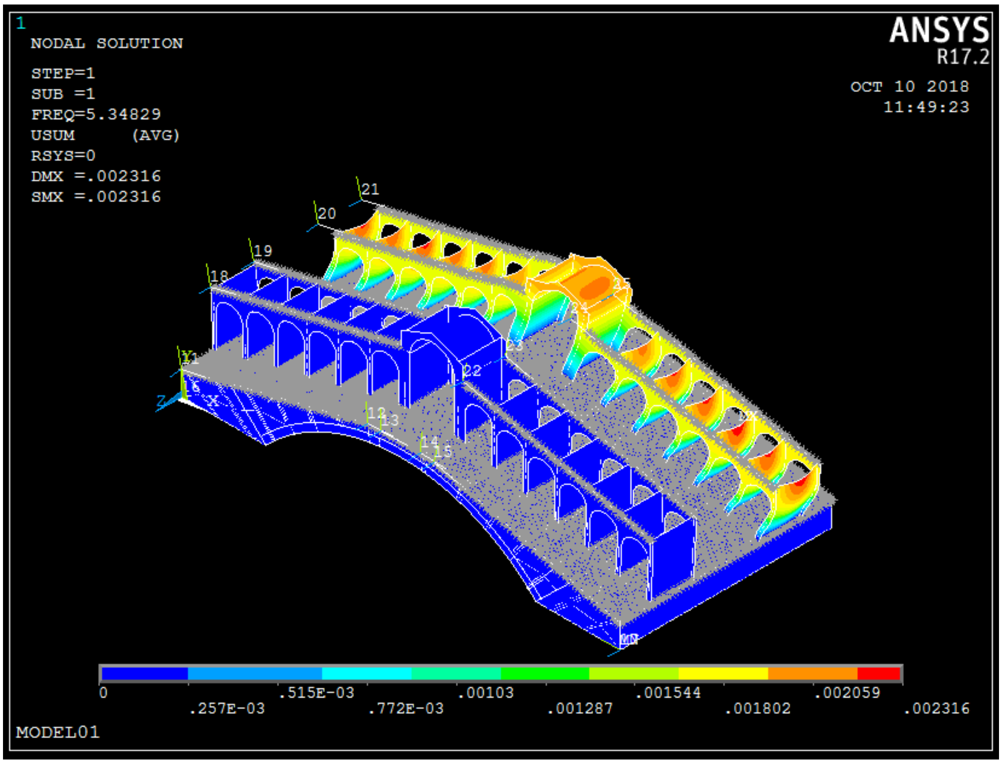


Figure 4-17 First Post updating Mode

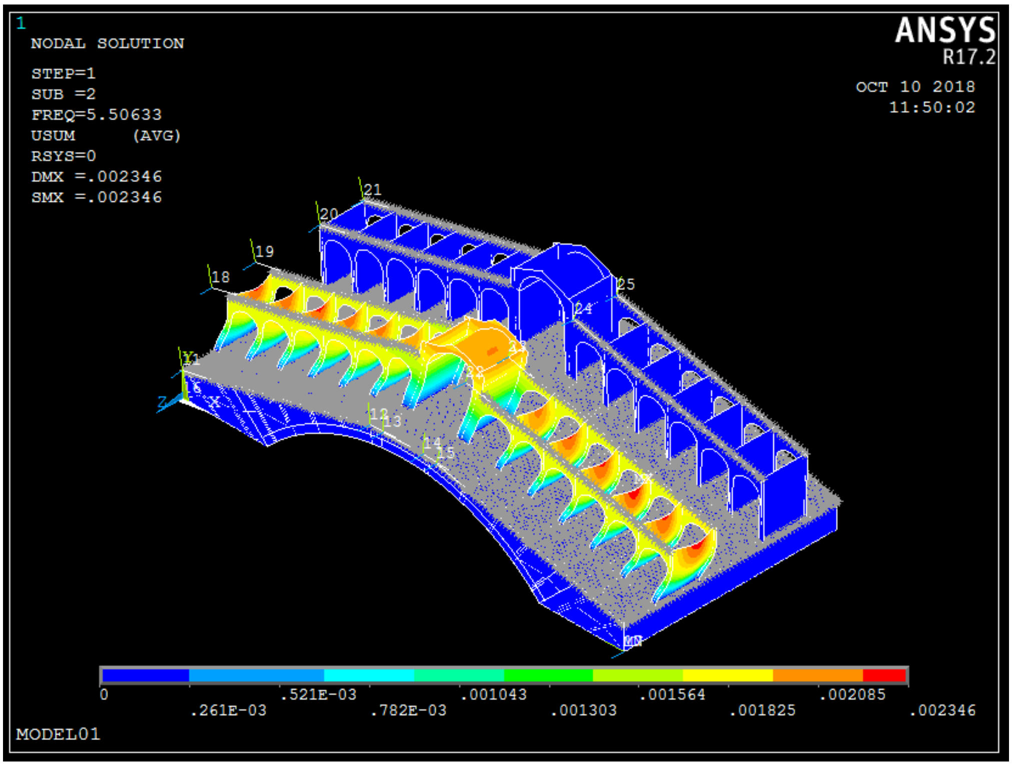


Figure 4-18 Second Post Updating Mode

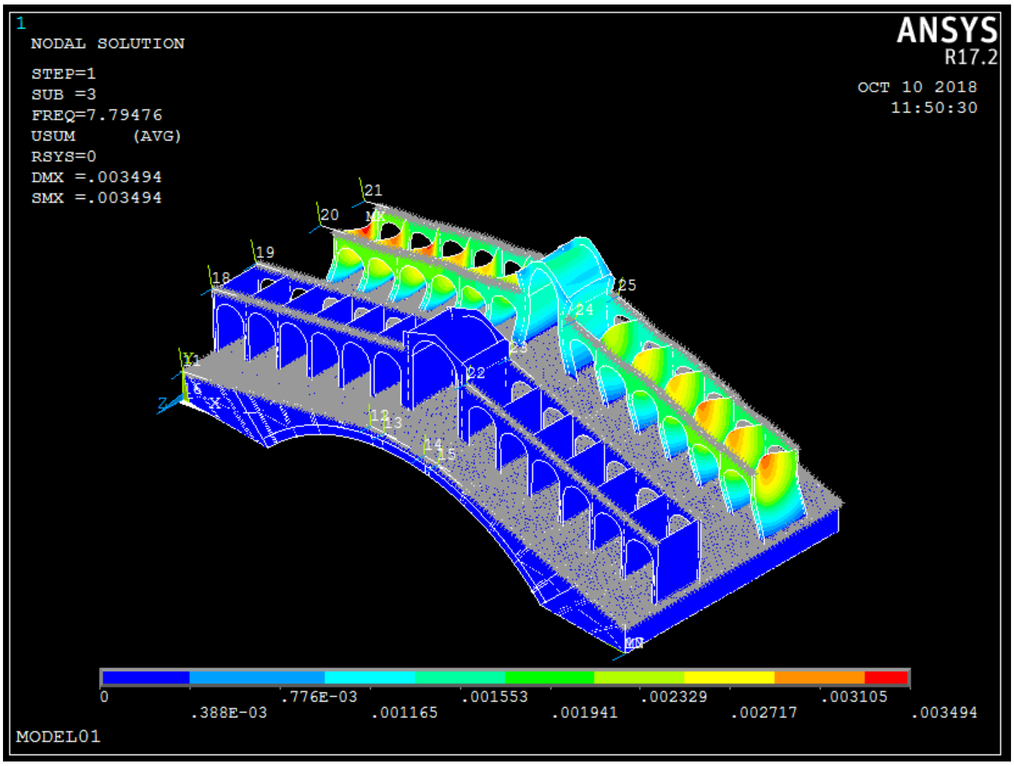


Figure 4-19 Third Post Updating Mode

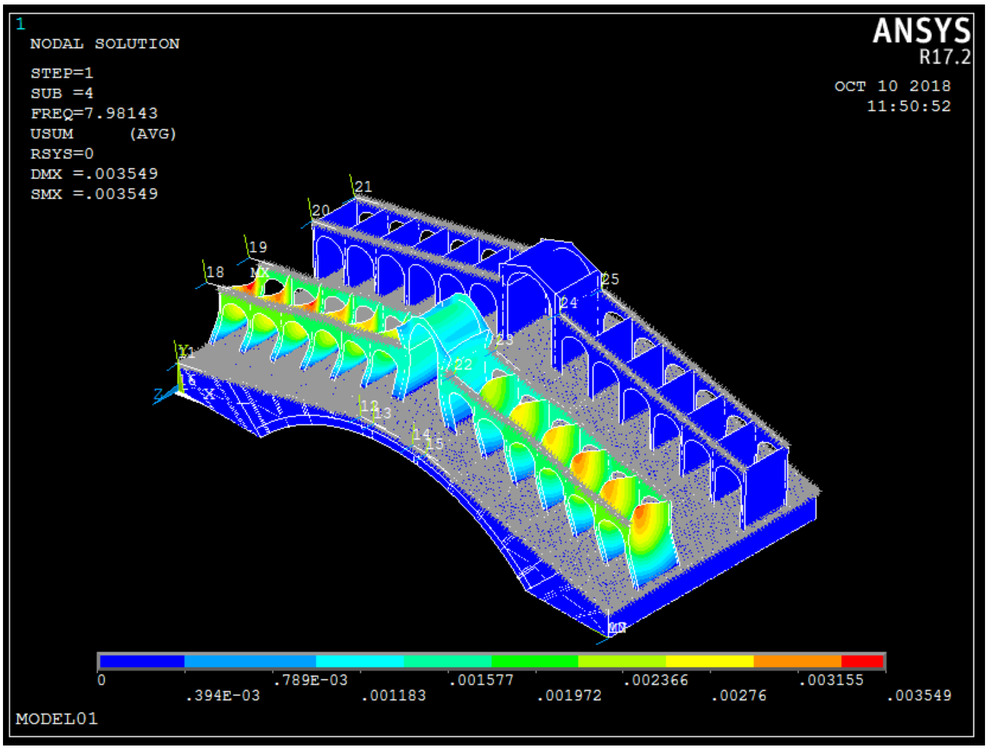


Figure 4-20 Fourth Post Updating Mode

The modal forms of the buildings above the arch show an independent behaviour of the structures to the north and south.

Having noticed the difference between the modal frequencies identified, concerning separately the buildings arranged in the north and south, it was decided to attribute different values of elastic modulus to the wall partitions arranged along the longitudinal axis of the bridge, and this has led in the first two ways to different modal forms from those obtained in the preliminary definition phase of the modele. The first two modes in fact no longer concern both buildings, but each one is characterized by a different modal frequency.

The third and fourth modes instead assume the same configuration from the point of view of modal shifts, while the corresponding modal frequency is significantly reduced.

Having more detailed information on the cracking pattern and having complete modal forms available on the entire structure, it would be possible to carry out a more precise calibration study, which also takes into account the correlation between the modal displacements recorded on the shops and those calculated by the model.

Conclusions

Going back over the work done, we can see how, starting from the experimental data recorded during a campaign of acquisition of vibrations of the bridge under the effect of environmental agents, it was possible to reconstruct the dynamic behavior of the bridge in stone and masonry, of neat geometric complexity.

Moreover, starting from the knowledge of the geometry, a finite element model has been realized that, once calibrated, partially simulates the real behavior of the structure.

This model can be used as a starting point for further calibration, starting from a more detailed knowledge of the state of the structure and following a more in-depth investigation of the cracking state of the walls. The difficulty of extracting good results from the model was mainly due to the fact of having vectors of local and disconnected modal shifts, one related to the arc and the others related to the shops, therefore, since there is not a good spatial resolution of the acquisition channels, the search for modes was very conditioned by the uncertainty of being able to evaluate a global situation. This factor has caused many problems, also due to the fact that, given the considerable geometric complexity, the onset of many local modal forms has made it difficult to identify the frequency range in which to search for the modal form related to the arc. Having global modal shifts available, the correlation between numerical and experimental data would have been much more appreciable and easy to identify.

However, this model, even if it has some limits, can be useful for the possible development of a more precise model, having more detailed information on the overall behaviour of the bridge and on the interaction between the elements available.

In the context of a more general discourse, research efforts are concentrated on the implementation of structural monitoring systems in order to assess the state of health of buildings; in fact, this diagnostic tool allows the construction examined to provide real-time and remote information on their state of health.

All this is related to the subject of seismic prevention, in the knowledge that the construction techniques used in the past are now inadequate considering the many seismic events that have occurred in the last forty years in Italy, which have dramatically highlighted the extreme seismic vulnerability of our national architectural heritage

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Appendix A

MODE	1	2	3	4
Frequency [Hz]	5.27	5.45	7.08	7.76
Damping [%]	2.23	2.7	2.64	0.91
p:9:+X	0.29311	0.82884	-0.031139	0.47204
p:9:+Z	0.067977	0.042797	-0.012846	-0.29344
p:10:+X	0.039633	0.11289	0.00072082	0.145
p:10:+Z	0.003174	-0.0075387	0.0021088	-0.22091
p:11:+X	0.35651	1	-0.020138	-0.91355
p:11:+Z	0.036634	0.068384	-0.0036385	-0.30642
p:12:+X	-0.0042234	-0.00082865	-0.0038887	0.003915
p:12:+Z	0.0023737	0.0012311	0.0085523	-0.0021578
p:13:+X	0.051082	0.14144	-0.0016156	-0.15716
p:13:+Z	0.0072473	0.015593	-0.002067	-0.23263
p:14:+X	0.12855	-0.00010809	-0.021553	-0.001869
p:14:+Z	0.12866	0.00060298	-0.04932	0.0010487
p:15:+X	0.0058124	0.00090975	0.053108	-0.0030868
p:15:+Z	-0.14072	-0.0066946	-0.023164	-0.0028029
p:16:+X	-0.065467	0.029227	0.018178	-0.031495
p:16:+Z	0.020194	0.0067699	0.019068	0.032365
p:17:+X	-0.010085	-0.02599	0.0015751	0.33994
p:17:+Z	0.28099	0.8033	0.011178	1
p:18:+X	-0.014281	0.0045372	-0.0017819	-0.0066027
p:18:+Z	0.0042925	0.00086466	0.0022252	0.0029446
p:19:+X	0.046606	0.12229	-0.0020673	-0.14001
p:19:+Z	-0.01256	-0.019547	0.0015367	0.19978
p:20:+X	-0.31984	-0.012303	0.97653	0.017332
p:20:+Z	-1	-0.049256	0.2472	0.01293
p:21:+X	-0.32304	0.010342	1	-0.02281
p:21:+Z	0.083178	0.0076357	0.14796	0.11883
p:22:+X	-0.14444	-0.0041043	0.50668	0.036272
p:22:+Z	0.063249	-0.005513	-0.0011916	0.008043
p:23:+X	0.038873	0.0079194	0.56603	-0.0033959
p:23:+Z	0.11468	-0.010987	0.18396	-0.11901
p:25:+X	0.054305	-0.0022562	0.10013	-0.00091943
p:25:+Z	-0.13257	-0.0068489	-0.040057	0.0028614

Appendix B

Frequency [Hz]	19.75
Damping [%]	2.23
p:1:+Y	-0.0079694
p:2:+Y	0.30647
p:3:+Y	0.33395
p:4:+Y	0.0128
p:24:+X	0.18544
p:24:+Z	-0.61848
p:24:+Y	1
p:26:+Y	0.82342
p:27:+X	0.0028496
p:27:+Z	-0.0039603
p:27:+Y	0.034386
p:28:+Y	-0.043863
p:29:+Y	-0.0013083
p:30:+Y	0.10225
p:31:+Y	-0.0087855
p:32:+X	-0.063513
p:33:+X	-0.0062641
p:34:+X	0.0089928
p:35:+X	0.0030157
p:36:+X	-0.0027986
p:37:+X	-0.022477

Appendix C

Ansys Text

```
/CLEAR,START
/COM,ANSYS RELEASE Release 17.2
BUILD 17.2      UP20160718
/input,start,ans,'C:\ProgramFiles\ANSYSInc\v
172\AN-YS\apdl\'
/REPLOT,RESIZE
/CWD,'C:\Users\computer\Desktop\RIALTO
BRIDGE'
/TITLE,RIALTO
/FILNAME,RIALTO,0

/AUX15
IOPTN,IGES,SMOOTH
IOPTN,MERGE,YES
IOPTN,SOLID,YES
IOPTN,SMALL,NO
IOPTN,GTOLER,FILE

------------------------------------------------------
!IMPORT OF GEOMETRY

IGESIN,'P.1','igs','FILE.IGES'
APLOT
CM,P.1,AREA
CMSEL,NONE

IGESIN,'P.3.1','igs','FILE.IGES'
APLOT
CM,P.3.1,AREA
CMSEL,NONE

IGESIN,'P.3.2','igs','FILE.IGES'
APLOT
CM,P.3.2,AREA
CMSEL,NONE

IGESIN,'P.3.3','igs','FILE.IGES'
APLOT
CM,P.3.3,AREA
CMSEL,NONE

IGESIN,'P.3.4','igs','FILE.IGES'
APLOT
CM,P.3.4,AREA
CMSEL,NONE
IGESIN,'P.3.5','igs','FILE.IGES'
APLOT
CM,P.3.5,AREA
CMSEL,NONE

IGESIN,'P.3.6','igs','FILE.IGES'
APLOT
CM,P.3.6,AREA
CMSEL,NONE

IGESIN,'P.3.7','igs','FILE.IGES'
APLOT
CM,P.3.7,AREA
CMSEL,NONE

IGESIN,'P.3.7.1','igs','FILE.IGES'
APLOT
CM,P.3.7.1,AREA
CMSEL,NONE

IGESIN,'P.4','igs','FILE.IGES'        !VAULT
APLOT
CM,P.4,AREA
CMSEL,NONE

IGESIN,'C','igs','FILE.IGES'          !COVER
APLOT
CM,C,AREA
CMSEL,NONE

IGESIN,'P.5','igs','FILE.IGES'        !LOFT
APLOT
CM,P.5,AREA
CMSEL,NONE

IGESIN,'P.6','igs','FILE.IGES'           !RIGID
LINK
APLOT
CM,P.6,AREA
CMSEL,NONE

~SATIN,'VOL1.0','sat','FILE.IGES'
VPLOT
CM,VOL1.0,VOLU
/PREP7
VA,ALL
/AUX15
ASEL,NONE
```

```
CMSEL,NONE

~SATIN,'VOL1.1','sat','FILE.IGES'
VPLOT
CM,VOL1.1,VOLU
/PREP7
VA,ALL
/AUX15
ASEL,NONE
CMSEL,NONE

~SATIN,'VOL1.2','sat','FILE.IGES'
VPLOT
CM,VOL1.2,VOLU
/PREP7
VA,ALL
/AUX15
ASEL,NONE
CMSEL,NONE

~SATIN,'VOL2','sat','FILE.IGES'
VPLOT
CM,VOL2,VOLU
/PREP7
VA,ALL
/AUX15
ASEL,NONE
CMSEL,NONE

~SATIN,'VOL3','sat','FILE.IGES'
VPLOT
CM,VOL3,VOLU
/PREP7
VA,ALL
CMSEL,NONE

ALLSEL
APLOT

-------------------------------------------------------
!THICKNESS DEFINITION

/PREP7
ET,1,SHELL181
ET,2,SOLID185


SECTYPE,1,SHELL !P.1
SECDATA,0.35
SECOFFSET,BOT

SECTYPE,2,SHELL !P.3.1
SECDATA,0.265
SECOFFSET,BOT

SECTYPE,3,SHELL !P.3.2
SECDATA,0.257

SECTYPE,4,SHELL !P.3.3
SECDATA,0.238

SECTYPE,5,SHELL !P.3.4
SECDATA,0.252

SECTYPE,6,SHELL !P.3.5
SECDATA,0.252

SECTYPE,7,SHELL !P.3.6
SECDATA,0.263

SECTYPE,8,SHELL !P.3.7
SECDATA,0.897

SECTYPE,9,SHELL !P.3.7.1
SECDATA,0.642

SECTYPE,10,SHELL !P.4 volta
SECDATA,0.285

SECTYPE,11,SHELL !C
SECDATA,0.185

SECTYPE,12,SHELL !P.5
SECDATA,0.25

SECTYPE,13,SHELL !P.6
SECDATA,0.10

-------------------------------------------------------
!CREATE MATERIAL

MP,EX,1,3365e6   !X DIRECTED WALLS
MP,DENS,1,2400
MP,NUXY,1,0.3

MP,EX,2,449e6    !ARC MASONRY
MP,DENS,2,1706
MP,NUXY,2,0.3

MP,EX,3,3387e6   !ISTRIAN STONE
MP,DENS,3,2657
MP,NUXY,3,0.3

MP,EX,4,449e6    !ABUTMENT MASONRY
MP,DENS,4,1706
MP,NUXY,4,0.3
```

```
MP,EX,5,255e6    !Z DIRECTED WALLS
MP,DENS,5,1800
MP,NUXY,5,0.3

MP,EX,6,82e6     !COVERING
MP,DENS,6,232
MP,NUXY,6,0.3

MP,EX,7,82e6     !LOFT
MP,DENS,7,472
MP,NUXY,7,0.3

MP,EX,8,1e20     !RIGID LINK
MP,DENS,8,1
MP,NUXY,8,0.3

--------------------------------------------------------
!PROPERTY ATTRIBUTION (SHELL)

CMSEL,S,P.1
AATT,1,,1,,1
CMSEL,NONE

CMSEL,S,P.3.1
AATT,5,,1,,2
CMSEL,NONE

CMSEL,S,P.3.2
AATT,5,,1,,3
CMSEL,NONE

CMSEL,S,P.3.3
AATT,5,,1,,4
CMSEL,NONE

CMSEL,S,P.3.4
AATT,5,,1,,5
CMSEL,NONE

CMSEL,S,P.3.5
AATT,5,,1,,6
CMSEL,NONE

CMSEL,S,P.3.6
AATT,5,,1,,7
CMSEL,NONECMSEL,S,P.3.7
AATT,5,,1,,8
CMSEL,NONE

CMSEL,S,P.3.7.1
AATT,5,,1,,9
CMSEL,NONE

CMSEL,S,P.4
AATT,5,,1,,10
CMSEL,NONE

CMSEL,S,C
AATT,6,,1,,11
CMSEL,NONE

CMSEL,S,P.5
AATT,7,,1,,12
CMSEL,NONE

CMSEL,S,P.6
AATT,8,,1,,13
CMSEL,NONE

ASEL,ALL
APLOT

--------------------------------------------------------
!PROPERTY ATTRIBUTION (VOLUMS)

CMSEL,S,VOL1.0
VATT,4,,2
CMSEL,NONE

CMSEL,S,VOL1.1
VATT,4,,2
CMSEL,NONE

CMSEL,S,VOL1.2
VATT,4,,2
CMSEL,NONE

CMSEL,S,VOL2
VATT,2,,2,,
CMSEL,NONE

CMSEL,S,VOL3
VATT,3,,2,,
CMSEL,NONE

VSEL,ALL
VPLOT
ASEL,ALL
APLOT


--------------------------------------------------------
!THE AREAS MESH

ESIZE,0.4
```

```
AMESH,P.1
AMESH,P.3.1
AMESH,P.3.2
AMESH,P.3.3
AMESH,P.3.4
AMESH,P.3.5
AMESH,P.3.6
AMESH,P.3.7
AMESH,P.3.7.1
AMESH,P.4
AMESH,C

------------------------------------------------------
!THE VOLUMS MESH

MSHKEY,0
MSHAPE,1,3d

VSEL,NONE
VSEL,S,,,VOL1.0
VSEL,A,,,VOL1.1
VSEL,A,,,VOL1.2
VSEL,A,,,VOL2
VSEL,S,,,VOL3
VMESH,ALL
VMESH,ALL

!NODE SELECTION FOR MASS CREATION (PAVIMENTATION)

CMSEL,S,SUP,NODE

NSEL,NONE
NPLOT
LOCAL,11,0,0.204E-03,2.52,0,14
NSEL,S,LOC,X,0,20.7
NSEL,R,LOC,Z,0,-23
NSEL,R,LOC,Y,0,0.02
CM,SUP,NODE

LOCAL,12,0,20.075,7.537,0,3
NSEL,S,LOC,X,-0.01,1.5
NSEL,R,LOC,Z,0,-23
NSEL,R,LOC,Y,0,0.02
NPLOT

CMSEL,A,SUP
CM,SUP,NODE

LOCAL,13,0,21.568,7.626,-0.591E-10
NSEL,S,LOC,X,0,4.252
NSEL,R,LOC,Z,0,-23
NSEL,R,LOC,Y,0,0.02
CMSEL,A,SUP
CM,SUP,NODE

LOCAL,14,0,25.819,7.626,0.76-10,-3
NSEL,S,LOC,X,-0.01,1.5
NSEL,R,LOC,Z,0,-23
NSEL,R,LOC,Y,0,-0.02

CMSEL,A,SUP
CM,SUP,NODE

LOCAL,15,0,27.312,7.537,0.11E-12,-14
NSEL,S,LOC,X,0,20.7
NSEL,R,LOC,Z,0,-23
NSEL,R,LOC,Y,0,-0.02

CMSEL,A,SUP
CM,SUP,NODE
NPLOT

------------------------------------------------------
!MASS ATTRIBUTION

/PREP7
*GET,NUMNODES, NODE, 0,COUNT

M_TOT=996233
ET,3,MASS21
R,1,M_TOT/NUMNODES


*VGET,NODE_LIST,NODE,,NLIST
KP_INIT=150000

*DO,ii,1,NUMNODES
KNODE,KP_INIT+ii,NODE_LIST(ii)
*ENDDO
CMSEL,S,SUP,NODE
KSEL,S,KP,,KP_INIT,KP_INIT+NUMNODES

!MESH KP WITH MASS21

TYPE,3
REAL,1
KMESH,ALL

!NODE SELECTION FOR MASS CREATION (BALAUSTRAD-NORTH SIDE)

NSEL,NONE
NPLOT
LOCAL,11,0,0.204E-03,2.520,0,14
```

```
NSEL,S,LOC,X,0,20.7
NSEL,R,LOC,Z,0
NSEL,R,LOC,Y,0,0.02
NPLOT
CM,LINE1,NODE

LOCAL,12,0,20.075,7.537,0,3
NSEL,S,LOC,X,-0.01,1.5
NSEL,R,LOC,Z,0,
NSEL,R,LOC,Y,0,0.02
NPLOT

CMSEL,A,LINE1
CM,LINE1,NODE

LOCAL,13,0,21.568,7.626,-0.5916E-10
NSEL,S,LOC,X,0,4.252
NSEL,R,LOC,Z,0
NSEL,R,LOC,Y,0,0.02
NPLOT

CMSEL,A,LINE1
CM,LINE1,NODE

LOCAL,14,0,25.819,7.62,0.769E-10,-3
NSEL,S,LOC,X,-0.01,1.5
NSEL,R,LOC,Z,0
NSEL,R,LOC,Y,0,-0.02

CMSEL,A,LINE1
CM,LINE1,NODE

LOCAL,15,0,27.312,7.537,0.110E-12,-14
NSEL,S,LOC,X,0,20.7
NSEL,R,LOC,Z,0
NSEL,R,LOC,Y,0,-0.02

CMSEL,A,LINE1
CM,LINE1,NODE
NPLOT


-------------------------------------------------------
!MASS ATTRIBUTION

/PREP7
*GET,NUMNODES, NODE, 0,COUNT

M_TOT=10121.3
ET,3,MASS21
R,1,M_TOT/NUMNODES

*VGET,NODE_LIST,NODE,,NLIST
KP_INIT=100000

*DO,ii,1,NUMNODES
KNODE,KP_INIT+ii,NODE_LIST(ii)
*ENDDO
CMSEL,S,LINE1,NODE
KSEL,S,KP,,KP_INIT,KP_INIT+NUMNOD
ES

!MESH KP WITH MASS21

TYPE,3
REAL,1
KMESH,ALL

-------------------------------------------------------
!NODE SELECTION FOR MASS
CREATION (BALAUSTRAD-SUD SIDE)

CMSEL,S,LINE2,NODE

NSEL,NONE
LOCAL,11,0,0.204E-03,2.520,0,14
NSEL,S,LOC,X,0,20.7
NSEL,R,LOC,Z,-23,-22.9
NSEL,R,LOC,Y,0,0.02
NPLOT
CM,LINE2,NODE

LOCAL,12,0,20.075,7.537,0,3
NSEL,S,LOC,X,-0.01,1.5
NSEL,R,LOC,Z,-23,-22.9
NSEL,R,LOC,Y,0,0.02

CMSEL,A,LINE2
CM,LINE2,NODE

LOCAL,13,0,21.568,7.626,-0.59E-10
NSEL,S,LOC,X,0,4.252
NSEL,R,LOC,Z,-23,-22.9
NSEL,R,LOC,Y,0,0.02
NPLOT

CMSEL,A,LINE2
CM,LINE2,NODE

LOCAL,14,0,25.819,7.626,0.769E-10,-3
NSEL,S,LOC,X,-0.01,1.5
NSEL,R,LOC,Z,-23,-22.9
NSEL,R,LOC,Y,0,-0.02

CMSEL,A,LINE2
CM,LINE2,NODE
```

```
LOCAL,15,0,27.312,7.53,0.11E-12,-14
NSEL,S,LOC,X,0,20.7
NSEL,R,LOC,Z,-23,-22.9
NSEL,R,LOC,Y,0,-0.02

CMSEL,A,LINE2
CM,LINE2,NODE

-------------------------------------------------------
!MASS ATTRIBUTION

*GET,NUMNODES, NODE, 0,COUNT

M_TOT=10121.3
ET,3,MASS21
R,1,M_TOT/NUMNODES


*VGET,NODE_LIST,NODE,,NLIST
KP_INIT=200000

*DO,ii,1,NUMNODES
KNODE,KP_INIT+ii,NODE_LIST(ii)
*ENDDO
CMSEL,S,LINE2,NODE
KSEL,S,KP,,KP_INIT,KP_INIT+NUMNOD
ES

!MESH KP WITH MASS21

TYPE,3
REAL,1
KMESH,ALL
-------------------------------------------------------
!CREATE SPRINGS FOR SOIL IN
DIREZIONE VERTICALE

*GET,NUMNODES, NODE, 0,COUNT
LOCAL,16,0,0.204E-03,-0.4E-04,0

NSEL,NONE
NSEL,S,LOC,Y,0.01,-0.01
NPLOT
COORD=2            !Y DIRECTION
k_TOT=1.25e12
K_springs1=k_TOT/NUMNODES

*get,maxtype,ETYP,,NUM,MAX
*get,maxreal,RCON,,NUM,MAX
ET,4,COMBIN14
R,maxreal+1,K_springs1

TYPE,4
REAL,maxreal+1
DELTA=0.01

*get,N_lower,NODE,,NUM,MAXD
*get,E_lower,ELEM,,NUM,MAXD

N_lower=N_lower+1
E_lower=E_lower+1
create_springs,N_NUM,DELTA,COORD,N_
Lower,E_Lower

-------------------------------------------------------
!CREATE SPRINGS FOR SOIL IN
DIREZIONE VERTICALE

*GET,NUMNODES, NODE, 0,COUNT
NSEL,NONE
NSEL,S,LOC,Y,0.01,-0.01
NPLOT
COORD=3
k_TOT=1.25e12
K_springs2=k_TOT/NUMNODES

*get,maxtype,ETYP,,NUM,MAX
*get,maxreal,RCON,,NUM,MAX
ET,4,COMBIN14
R,maxreal+1,K_springs2

TYPE,4
REAL,maxreal+1

DELTA=0.01
*get,N_lower,NODE,,NUM,MAXD
*get,E_lower,ELEM,,NUM,MAXD

N_lower=N_lower+1
E_lower=E_lower+1
create_springs,N_NUM,DELTA,COORD,N_
Lower,E_Lower


-------------------------------------------------------
!CREATE SPRINGS FOR SOIL IN
DIREZIONE VERTICALE

*GET,NUMNODES, NODE, 0,COUNT
NSEL,NONE
NSEL,S,LOC,Y,0.01,-0.01
NPLOT
COORD=1
k_TOT=1.25e12
K_springs3=k_TOT/NUMNODES

*get,maxtype,ETYP,,NUM,MAX
```

```
*get,maxreal,RCON,,NUM,MAX
ET,4,COMBIN14
R,maxreal+1,K_springs3

TYPE,4
REAL,maxreal+1
DELTA=0.01  !DIST

*get,N_lower,NODE,,NUM,MAXD
*get,E_lower,ELEM,,NUM,MAXD

N_lower=N_lower+1
E_lower=E_lower+1

create_springs,N_NUM,DELTA,COORD,N_
Lower,E_Lower

-----------------------------------------------------
!CREATE SPRINGS FOR SOIL IN
DIREZIONE VERTICALE

K_springs4=8e12 !RIG MOLLE

*get,maxtype,ETYP,,NUM,MAX
*get,maxreal,RCON,,NUM,MAX
ET,4,COMBIN14
R,maxreal+1,K_stair

TYPE,4
REAL,maxreal+1
DELTA=0.01
*get,N_lower,NODE,,NUM,MAXD
*get,E_lower,ELEM,,NUM,MAXD
N_lower=N_lower+1
E_lower=E_lower+1


NSEL,NONE
NSEL,S,LOC,X,0.01,-0.01

LOCAL,17,0,47.38742,-0.4005920E-04,0,90
NSEL,A,LOC,Y,0.01,-0.01
NPLOT
COORD=1

create_springs,N_NUM,DELTA,COORD,N_
Lower,E_Lower


-----------------------------------------------------
MODAL ANALYS

ALLSEL

NUMMRG,NODE

PERFORM MODAL ANALYSIS
/SOLU
ANTYPE,MODAL
MODOPT,LANB,10,0,100, ,OFF
/STATUS,SOLU
SOLVE

!SAVE,MODEL01,DB
```