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TESI DI LAUREA

"PONTE DO ARCO", ANALISI DI UN PONTE IN MURATURA

Analysis of *"Ponte do Arco"* masonry arch bridge

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ABSTRACT

Masonry arch bridges, as the other existing building, have specific peculiarity that cause a particular complexity to evaluate them behaviour under the actions (static and dynamic) and, consequently, them safety. A masonry bridge is compound by numerous parts that interact during the daily responses to the external actions: we are talking, as major, about the arch, vault, spandrel wall, abutment, backfill and parapet. Whole these parts determine an high level of uncertainty when we have to evaluate the relative effect. Due of these reasons is pretty easy to understand how the structural model adopted to study the behaviour of a construction, compares and contains all variables and hypothesis that we assume to obtain a result.

The present work has the main goal of providing a general approach that explores in the forecasted sectors to get a based evaluation of the bridge’s behaviour.

SOMMARIO

I ponti in muratura come gli edifici esistenti godono di peculiarità specifiche che determinano una certa complessità nell’affrontare la valutazione del loro comportamento sotto le diverse azioni, statiche o dinamiche, e quindi della loro sicurezza. Un ponte in muratura è costituito da numerose parti che interagiscono tra loro durante la risposta quotidiana alle azioni esterne stiamo parlando quindi dell’arco, della volta, del timpano, delle spalle, del riempimento, del parapetto per citare le principali. Tutti questi elementi determinano un grado elevato di incertezza e di difficoltà nel valutarne l’incidenza. Per questi motivi è facile comprendere come il modello strutturale utilizzato per studiare il comportamento di una costruzione mette assieme e contiene tutte le variabili e ipotesi che assumiamo al fine di ottenere un risultato.

Tale lavoro quindi si propone di fornire un approccio generale che si addentri negli ambiti previsti per ottenere una valutazione fondate del comportamento del ponte.

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1. INTRODUCTION

The structural analysis of masonry arch bridges is a topic that deserves consideration and further developed by the scientific community for several reasons. Primarily for the functional role that these bridges play as the backbone of many existing automobile and railroad networks still in use today. Therefore making the assessment and conservation of these infrastructures of vital importance to ensure functionality in the transportation networks.

Moreover, to date, the structural behaviour of this type of construction is still under study both as regards to the static and dynamic actions. In fact, the response to external loads is governed by different factors of difficult acquisition and evaluation: geometric data, material properties, but especially the interaction between the different constituent parts. To this is added the problem of defining a general model that can replicate the real behaviour as close as possible to the original and that can ensure a higher level of structural safety with fewer assumptions. In this regard we have developed different models in recent years some two-dimensional and some three-dimensional. Those two-dimensional seek to take into account more accurately the interaction between the arc and the backfill, and those three-dimensional attempt to assess the structure in its entirety. There are two possible approaches to analyzing these infrastructures: one uses limit analysis of equilibrium, which takes into consideration the possible mechanisms that could have developed, and the other is the finite element method.

Many times the masonry arch bridges prove to be of great historical importance, both for their age and for their importance in local development. Often they bring with them monument features expressing an aesthetic and a historical value based on two elements that unite these works: the arch and the masonry. The use of this form and this material has always been a part of the development of the history of architecture and construction. It has gone from “*trilitico”* ancient Greek system that uses stone beam and columns, to the Roman arch size and the brick wall. This combination can be found in churches, buildings, aqueducts and bridges in the form of vault or dome. The development of the masonry arc can be attributed to the innovative nature of humans and the determination to create a positive situation out of a negative or unfavorable one through the better use of available technology and materials. The arc brings together form and structure and at the same time expresses beauty and strength therefore combining both aesthetical and historical values. Through observation of the object you can then identify a historical period, a necessity, and an environmental and social context and beauty. These structures serve as testimonies of our past and therefore it is in our interest to protect and preserve them, and even more so to show them off and bring them to light. To make this possible the primary goal is to make these infrastructures stable and safe, both for everyday use and to increase its chances of lasting through time in order to perpetuate its presence in the future. This last aspect sees the masonry arch bridges as a cultural heritage and therefore of shared importance.

From these reasons it has dealt with the thesis work towards the "Ponte do Arco" starting from the study of the state of the art, which defines the theoretical basis on which to refer. After developing a certain sensitivity towards the issue, we now move to a study of actual facts, and begin the investigation phase of the bridge. We speak then of the geometry, of the state of conservation of the bridge and the definition of material properties. Once acquired, this information was passed to the evaluation phase, which evaluates the behaviour of the bridge through the two approaches outlined above. The first refers to the kinematic theorem of limit analysis. Following the hypothesis of Heyman we identified a maximum failure load, making the structure a mechanism and evaluating the balance in conditions of safety. The second phase is based on the finite element modeling. With this method, we analyzed both the global and especially the horizontal behaviour of the bridge which is fundamental for predicting its behaviour during seismic actions.

With this research we aim to point out the complexity of the issue and show a method that can attempt to answer the posing question that every visitor and resident is asking themselves. Is the *“Ponte do Arco”* safe? And if so up to what load?

INTRODUZIONE

L’analisi strutturale dei ponti ad arco in muratura è un argomento che merita di essere preso in considerazione e ulteriormente sviluppato dalla comunità scientifica per diversi motivi.
In primis per il ruolo funzionale che suddetti ponti svolgono: essi costituiscono ancora oggi l’ossatura di numerose reti infrastrutturali in tutto il mondo, sia automobilistiche che ferroviarie. L’uso rende quindi tali opere di importanza strategica, di conseguenza la valutazione dello stato di conservazione e del livello di sicurezza fondamentale.
Inoltre, ad oggi, il comportamento strutturale di questa tipologia costruttiva risulta ancora in fase di studio sia per quanto riguarda le azioni statiche che dinamiche. Infatti, la risposta alle azioni esterne è governata da diversi fattori di difficile acquisizione e valutazione: dati geometrici, proprietà dei materiali ma soprattutto l’interazione tra le diverse parti costitutive. A questo si aggiunge il problema di definire un modello generale, da utilizzare, che possa essere il più vicino possibile al comportamento reale o che possa garantire un maggiore livello di sicurezza con poche ipotesi. A questo proposito si sono sviluppati diversi modelli negli ultimi anni, quelli bidimensionali cercano di tener in conto in maniera più accurata dell’interazione tra l’arco e il riempimento, quelli tridimensionali tentano di valutare nella globalità il problema. Gli approcci possono essere legati o all’utilizzo dell’analisi limite dell’equilibrio, considerando i possibili meccanismi che si possono sviluppare, o al metodo agli elementi finiti.
Molte volte i ponti ad arco in muratura risultano essere di notevole importanza storica, sia per la loro età sia per la loro rilevanza nello sviluppo del luogo. Spesso portano con loro le caratteristiche di monumento, esprimendo di conseguenza un valore estetico e uno storico. Entrano quindi in gioco aspetti che fanno parte del mondo del restauro. Tutto questo nasce da due elementi che accomunano tali opere: l’arco e la muratura. L’utilizzo di questa forma e di questo materiale ha accompagnato lo sviluppo della storia dell’architettura e delle costruzioni. Si è passati dal sistema trilitico della Grecia antica, trave e colonna di pietra, a quello romano formato dall’arco e dal muro di mattoni. Questo connubio si può quindi trovare nelle chiese, negli edifici, negli acquedotti e nei ponti sotto forma di volta o cupola. Lo sviluppo dell’arco in muratura nasce dall’intuito di saper risolvere una condizione sfavorevole o da un’esigenza, attraverso l’abilità e l’utilizzo migliore del materiale e delle tecniche a disposizione. L’arco mette assieme forma e struttura esprimendo nello

stesso tempo bellezza e forza. In questo elemento coesistono quindi due valori: quello estetico e quello storico. A partire dall'oggetto si può quindi identificare un periodo storico, un esigenza, un contesto ambientale e sociale, un'estetica. La costruzione come testimonianza del nostro passato e quindi bene da tutelare e conservare, anzi ancor meglio da esaltare, mettere in luce. Perché tutto ciò sia possibile la necessità primaria è quella di rendere stabile e sicuro l'oggetto in questione, sia nell'uso che nel mantenimento. In modo tale da perpetuare la sua presenza nel futuro. Questo ultimo aspetto vede i ponti ad arco in muratura come patrimonio culturale e quindi di importanza condivisa.

A partire da queste motivazioni si è affrontato il lavoro di tesi nei confronti del *"Ponte do Arco"* iniziando dallo studio dello stato dell'arte, il quale definisce le basi teoriche sulle quali fare riferimento. Dopo aver conseguito una certa sensibilità nei confronti del tema, ci si è calati sull'oggetto concreto. Si è sviluppata quindi la fase conoscitiva del ponte. Sia nei confronti della storia della costruzione sia rispetto a tutti quei dati che risultano fondamentali per valutarne il comportamento strutturale. Parliamo quindi della geometria, dello stato di conservazione del ponte e della definizione delle proprietà dei materiali. Una volta acquisite queste informazioni si è passati alla fase di valutazione del comportamento del ponte attraverso i due approcci indicati poc'anzi. Il primo fa riferimento al teorema cinematico dell'analisi limite. Sfruttando le ipotesi di Heyman si è definito un carico ultimo di collasso, rendendo la struttura un cinematismo e valutandone l'equilibrio in condizioni di sicurezza. Il secondo basato sulla modellazione agli elementi finiti. Con questo metodo si è analizzato il comportamento globale del ponte considerando quindi anche quello trasversale, che risulta essere fondamentale per le azioni sismiche.

Con questo lavoro si vuol far emergere la complessità del tema, cercando di mostrare una metodologia con la quale approcciarsi e con la quale riuscire a dare risposta alla domanda che ogni abitante o visitatore si pone quando si trova davanti al *"Ponte do Arco"*: è sicuro? E per che carico?

2. LITERATURE REVIEW

The following chapter develops the evolution of the study of masonry arches and the bridges. It aims to show the progression from empirical rules and qualitative observations to models based on quantitative values, particularly the mechanics and behaviour of materials to understood as stress, strain, etc. Hereinafter the focus has shifted to legislation and about the structural analysis of a stone bridge through the instruments and procedures indicated, trying to identify the rules that provide a proper approach to present work. Thereby showing which models and theories refer to and highlighting any gaps or uncertainties of such instruments.

2.1 EMPIRICAL RULES

The design of masonry arches had always been based on rules and proportions handed down from the past experience and the traditional canons. In the ancient works, once fixed the module in example column's radius, the other parts would have been consequently determined. During the Roman age *ars* and *scientia* (practice and theory) had taught after the Pontifex Maximus's judgment (Heyman, 2002). These rules fixed the requirements that a building must had: *firmitas*, *utilitas* and *venustas* (solidness, utility, beauty). Vitruvius explains in his masterpiece titled De Architecura (15 B.C.) and he understands that arch pushes to the impost with slope force (Benvenuto, 1981). In the Architectura libri decem (23 B.C.) he suggests to use wider piers in spite of voussoir's size to set the whole structure. Joints thought voussoirs had to be oriented to the arch's center point (centring is the framework adopted during the arch's construction). Due of them geometry and once are loaded wedges press vertically to the core and thrust to imposts. This voussoir's feature had already used during the Etruscan age (Fig. 2. 1) (Huerta, 2001).

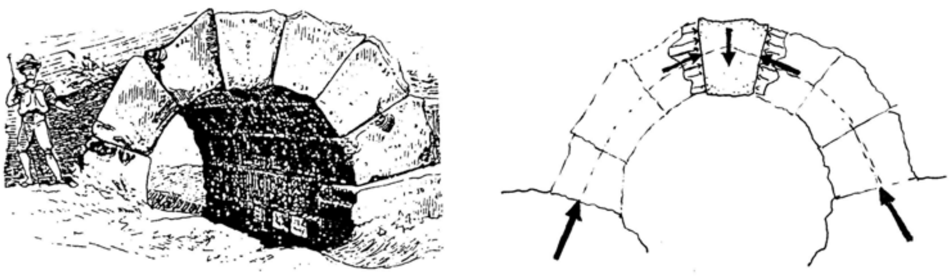


Fig. 2. 1 - Etruscan voussoir arch (Huerta, 2001)

Empirical rules change from the Romanic style with weighty columns and semicircular arches to the Gothic style with slim columns and pointed arches. Building rules were fundamental both for the proportions and make up the construction and were saved by roman *Frates Pontifices* and French/English *Frères Pontifes* (Heyman, 2002). According to Derand (1643), architects in the Middle Ages used to conform the piers' thickness with a geometric design (Fig. 2. 2): we can figure out how the column's thickness drop proportionally using the pointed arch due to the lower thrust (Heyman, 1982).

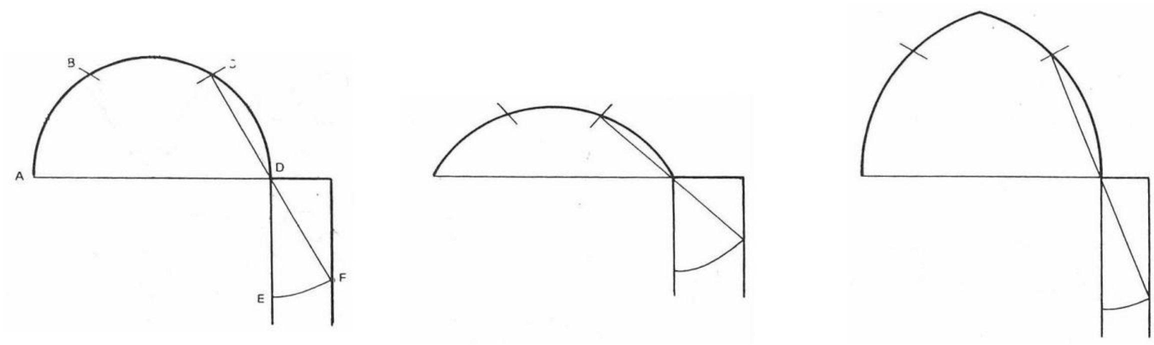


Fig. 2. 2 - Medieval empirical rules for the width of abutment pier by Derand (Heyman, 1982)

During the Italian Renaissance Leon Battista Alberti (1452) establishes a new design rule to build arches. He thought that the semicircular arch was the strongest with not need of a chain to balance the thrust. He fixed the rates that had to be used to build a bridge including the thickness of arches (Fig. 2. 3) (Benvenuto, 1981).

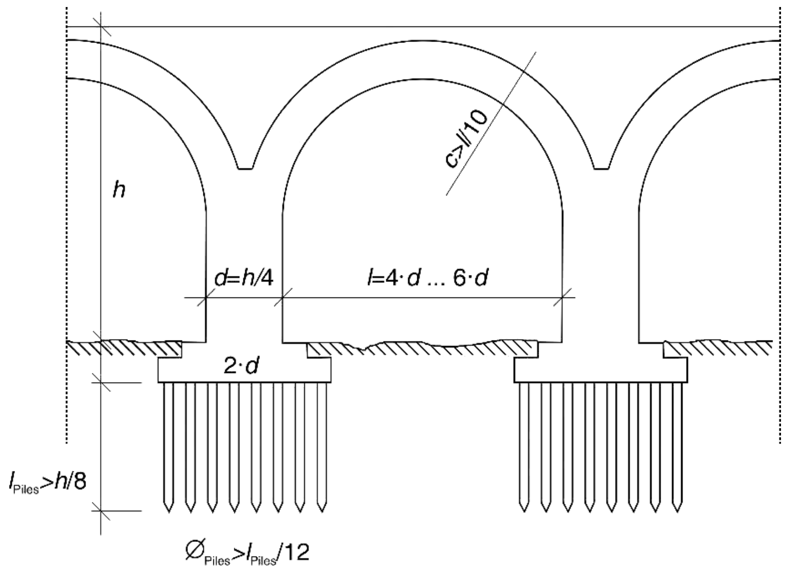
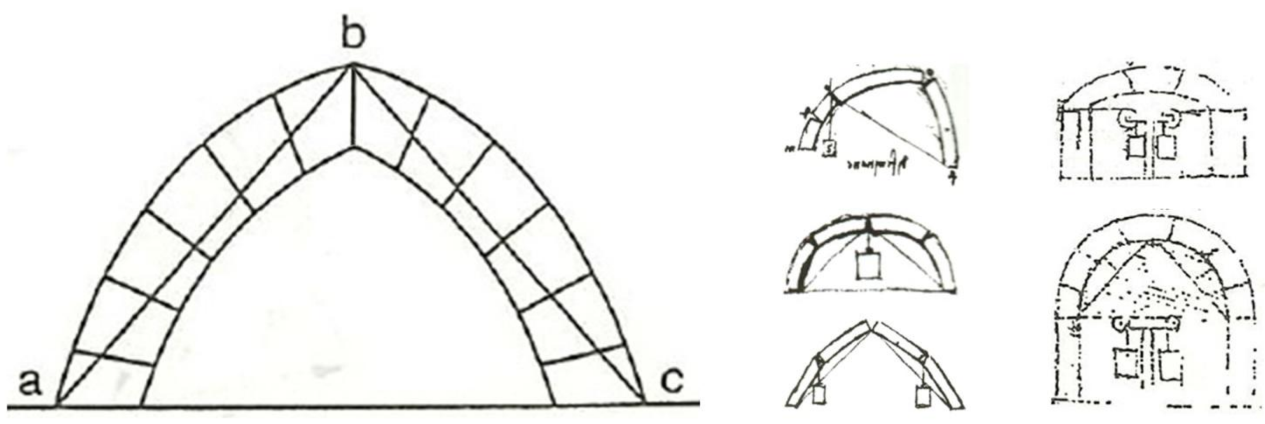


Fig. 2. 3 - Empirical rules for the design of arch bridges according to Leon Battista Alberti (Proske, van Gelder 2006)

Leonardo da Vinci (XV sec.) was the first who gave the efforts about the mechanic view of arch’s behaviour. He said that: *“Arco non è altro che una fortezza causata da due debolezze, imperoché l’arco negli edifizi è composto di due quarti di circulo, i quali quarti circuli ciascuno debolissimo per sé desidera cadere e oponendosi alla ruina l’uno dell’altro, le due debolezze si convertono in una unica fortezza”*. With a picture he describes how ab and bc string of extrados protect the arch with no touching the intrados (Fig. 2. 4 a). He deduces that in calm situation, thrust line is inside the arch. He was the fist who studied collapse’s causes depending on load’s position (Fig. 2. 4 b) and he tried to get arches’ pressure focusing on its features and weight (Fig. 2. 4 c) (Benvenuto, 1981).



a) b) c)

Fig. 2. 4 - Leonardo’s arch behaviour studies: a) stability condition (Benvenuto, 1981), b) mechanisms from *Codici di Madrid,* (Benvenuto, 1991), c) studies about thrust in *Codici Forster II* (Benvenuto, 1981)

Leonardo thought to the arch as two monolithic masses which become an unique body when they crash. The stereotomy as art used to cut stones whose goal is to build an architectural work is based on the monolithical idea got with the *règles de l’art* (Becci, Foce, 2002). For example to build an arch or a dome the crucial passage is the discretion of the whole work in a multitude of single parts. Starting from the need to make real a design, Stereotomy uses geometrical rules (like wedge’s mechanic and inclined plane) make it real; thanks to the Stereotomy, a work idea can be realized and this specific passage is crucial because the whole work might be detailed or changed. For the reasons above mentioned, the process of make real a designed work would change from the final work. So which is the relation that ties the global work with each passage and piece? Proportional’s rules can be taken as always correct for every measure’s building? Galileo (1638) showed that the load capacity of a structural element depends from its measures and he tried to make a failure’s analysis of a bracket, comparing the live load and the tensile strength (Fig. 2. 5).

Fig. 2. 5 - Galileo’s failure analysis (Benvenuto, 1981)

It comes that the proportional’s rules are partially wrong; however, these rules usually allow to masonry to work better because the stress during the service condition is lower and the collapse due material’s failure is less probably. So a correct shape guarantees a structural safe and the problem is solved by the equilibrium and geometry (Heyman, 2002).

2.2 ANALYSIS METHODS

The needs to combine the mechanic with the architecture become stronger and stronger during the XVII° century. To Galileo observations Hooke (1675) adds his own: *"ut pendet continuum flexible, sic stabit contiguum rigidum inversum"*. During this period were developed studies and observation on internal forces between voussoirs, optimal shape according to loads, minimal size to adopt in a arch to avoid mechanisms of rigid block and the specification of the deformable bodies theory. The last step of this evolution are the plastic theory and annexed methods of limit analysis (Becci, Foce, 2002).

2.2.1 MECHANICS APPLIED TO ARCHITECTURE

The turning personality of this change is Philippe de La Hire, who tried to consider the arch in its whole, combing stereotonimy's aspects with Hooke's observation and considering each voussoir with its weight and the general behaviour. From his work were born the two most important vault structure's studies approaches with the equilibrium use: the first is tied to the shape of chain the second to the collapse's analysis of mechanisms development (Becci, Foce, 2002). In the *Traité de mécanique* (1695) La Hire shows the relation between the shape of a rope and loads needs to hold it (Fig. 2. 6 a). The goal is to define the weight and the feature of voussoir that allow to exchange perpendicular mutual forces on joints passing thought its thickness. To solve this problem, La Hire suppose the absence of friction between voussoirs. Starting from the crown with fixed size, he establishes the other voussoirs' sizes, considering wedge's equilibrium (Fig. 2. 6 b). In *Sur le construction des voûtes* (1712) he exposes one more job about the topic of the comprehension of the vault's mechanism. He observes that, during the arch's failure its destroys in a lot of parts. The author thinks that the crown and the closer voussoirs press more than the other on the piers. He deduces that the joint failure is at 45° degrees from springing line of the arch (Fig. 2. 6 c). The voussoirs above mentioned can be considered as a unique stone like a wedge. Taking a look to this half structure, when this wedge moves down generates a thrust on the rigid body (arch portion and pier) falling around the external point of the basement. Considering the AOS lever we can find the Q value of the weight and the minimum size of the pier too. In fact the fallen mechanism is stopped if the width basement’s is bigger than the minimum size (Fig. 2. 6 d). The most important effort of La Hire is the intuition of two “simple machines”, inside the building the wedge and the lever based on the equilibrium. Minding these two “simple machines” is easier to get the arch’s behaviour (Benvenuto, 1981).

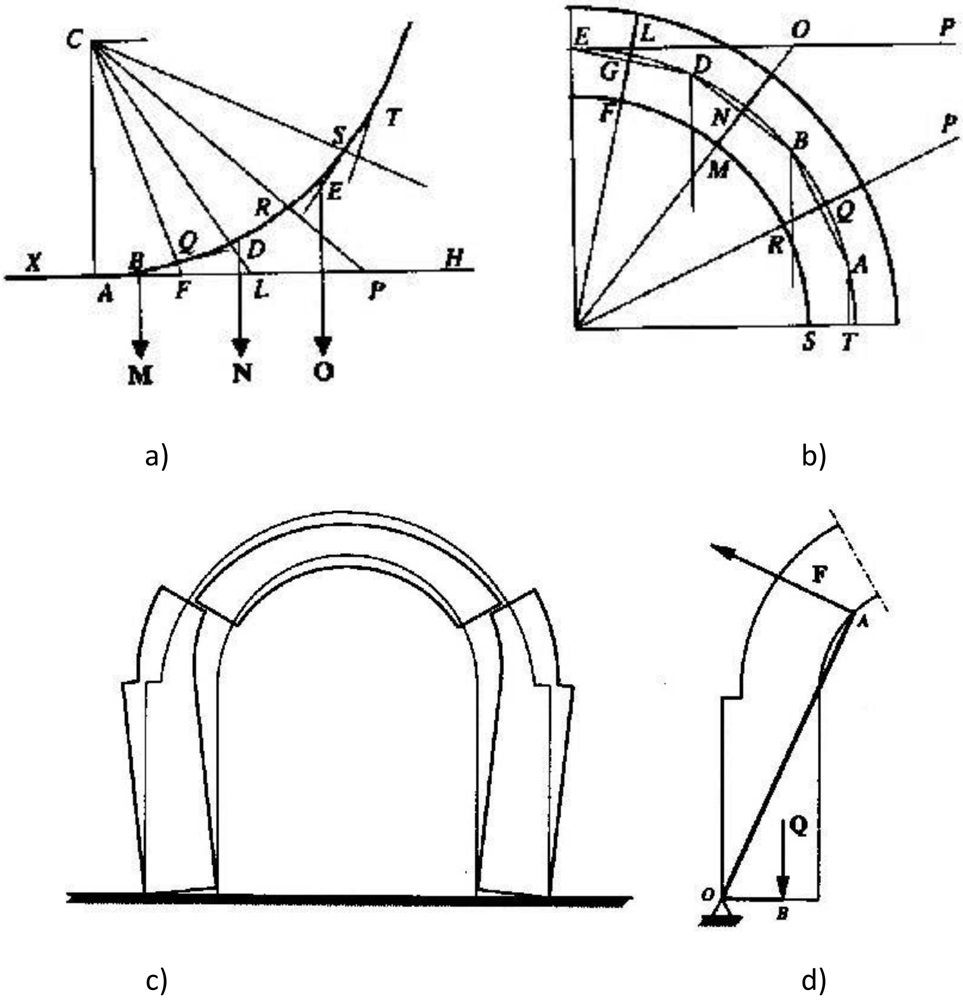


Fig. 2. 6 - La Hire’s arch behaviour studies: a) the relation between the shape of a rope and loads, b) equilibrium studies of voussoirs, c) wedge collapse mechanism, d) lever (Benvenuto, 1981)

The analysis of collapse mechanisms allows to find minimum thickness of arch and piers with considerations equilibrium only. At this point the evolution try to understand the problem of the modes of failure and to solve it.

According to Claude Couplet (1730) arch’s failure can occur only with rotation on hinge in four crack parts; those come from the crown and 45° degree joints and the whole structure generate a complex lever system (Fig. 2. 7).

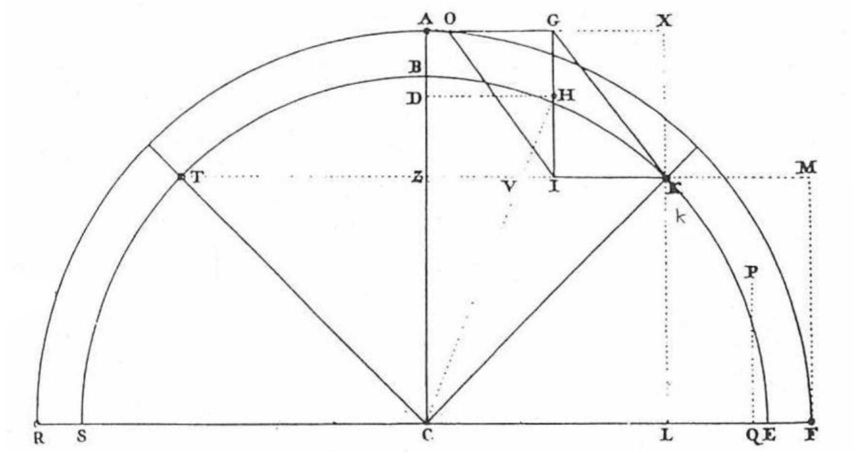


Fig. 2. 7 - Couplet's collapse mechanism (Heyman, 1982)

The two vault study's structure approach were adopted for the first time to the dome - tambour - buttress of St. Peter in Rome. The fist study is conducted using the principle of virtual work to the rotation mechanism of failure for the half spherical lune between present cracks (Fig. 2. 8). This approach is supported by the "*three mathematics"* Le Seur, Jacquier, Boscovich (1743).

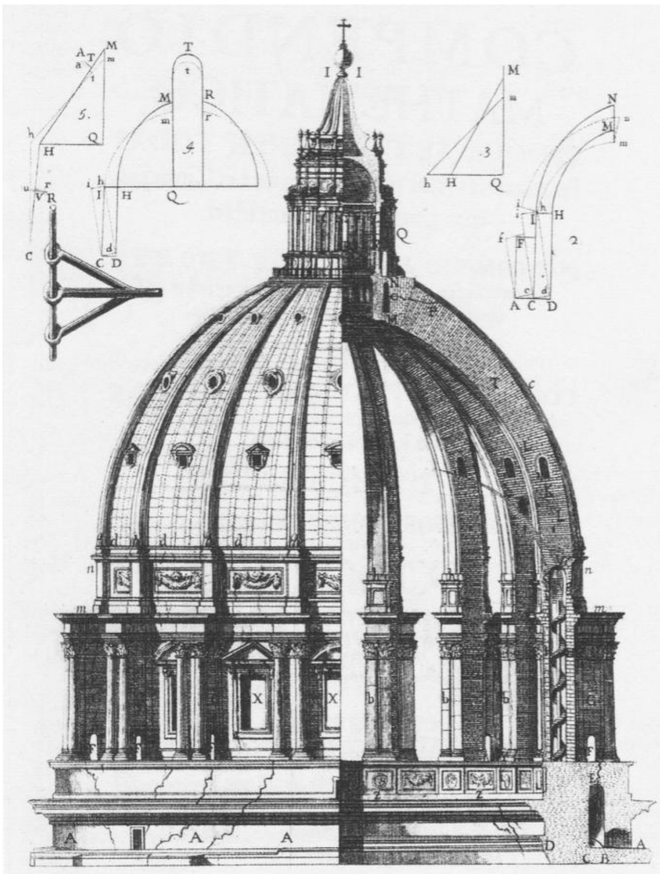
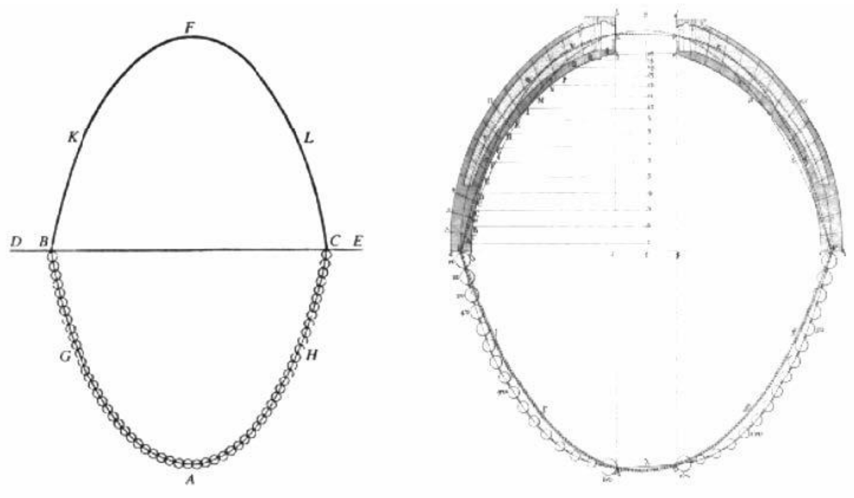


Fig. 2. 8 - Analysis of the Dome of St.-Peter's in Rome by three mathematics (Becci, Foce, 2002)

About this topic, was called physic professor Giovanni Poleni (1748) by the Pope. Referring to Hooke’s observation and Stirling (1717) writes chain’s shape, Poleni studies the dome’s portion through the model of hanging chain subdue to self weight (Fig. 2. 9 a). The nodal point of his work is to check if the chain was entirely within the thickness of the arch. That was the result from the experiment and the dome is stable (Fig. 2. 9 b) (Becci, Foce, 2002).



a) b)

Fig. 2. 9 - Poleni’s sudies: a) analogy between an arch and a hanging chain, b) analysis of the Dome of St.-Peter’s in Rome (Block et al., 2006)

The role of the friction occupies the following step in the analysis of arch behaviour. According to Charles Coulomb (1776) the friction developed between two rough surfaces is proportional to pressure that exits each other with no cohesion. Minding to this rule we can evaluate the equilibrium of a voussoir in the four option of failure: two by sliding two by rotation. The goal is to define the maximum and minimum values of P thrust when changes the voussoir length (Fig. 2. 10). The effective thrust will have to be found between these two limits to get the equilibrium and the difference shows how the arch is a statically indeterminate structure.

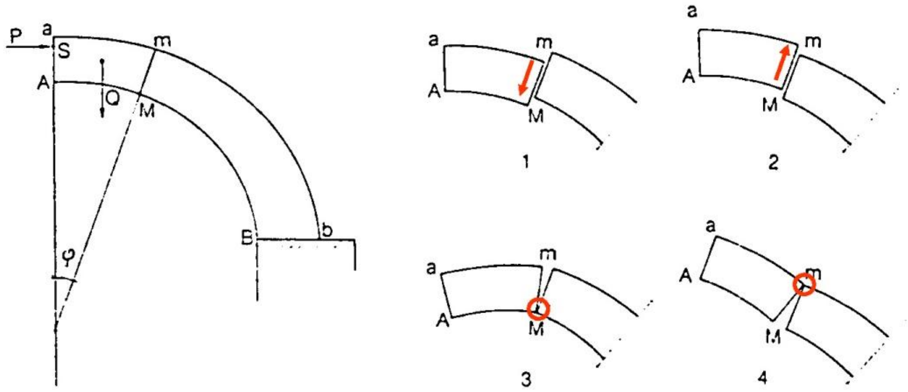
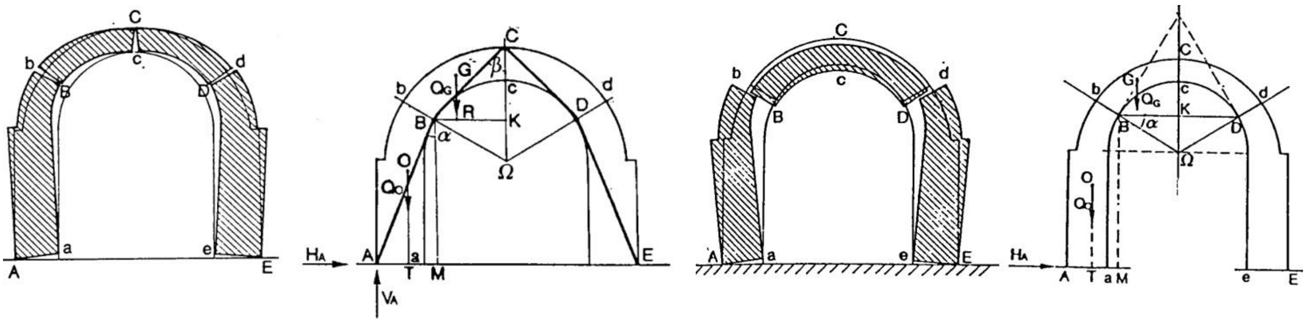


Fig. 2. 10 - Coulomb's study on vaults equilibrium (Benvenuto, 1981)

Instead if we would consider the equilibrium of a hypostatic system with more voussoirs, the instrument that we have to use is principle of virtual work. Lorenzo Mascheroni (1785) uses this principle to the different collapse's options, thinking that the joint's failure might be wherever (Fig. 2. 11). Rigid bodies are thought as rods. Moreover he considers friction and cohesion as zero to erase the problem calculating the influence only of external forces (Benvenuto, 1981).



a) b)

Fig. 2. 11 - Mascheroni's possible collapse mechanism: a) rotational, b) mixed (Benvenuto, 1981)

With Pierre Félix Michon the research of collapse's mechanisms accomplishes the objective to find a solution. In 1848 he publishes a series of tables to define the thickness of arches based on two elements: the stability's factor needed and the shape that is design. The stability factor was already adopted by Audoy (1820). About the shape, Michon says that arch might have a different thickness that grows from the crown to the impost. In 1857 he summarizes the research of older authors identifying the eight mechanisms of collapse with the specific vault with which it can happens (Fig. 2. 12) (Becci, Foce, 2002).

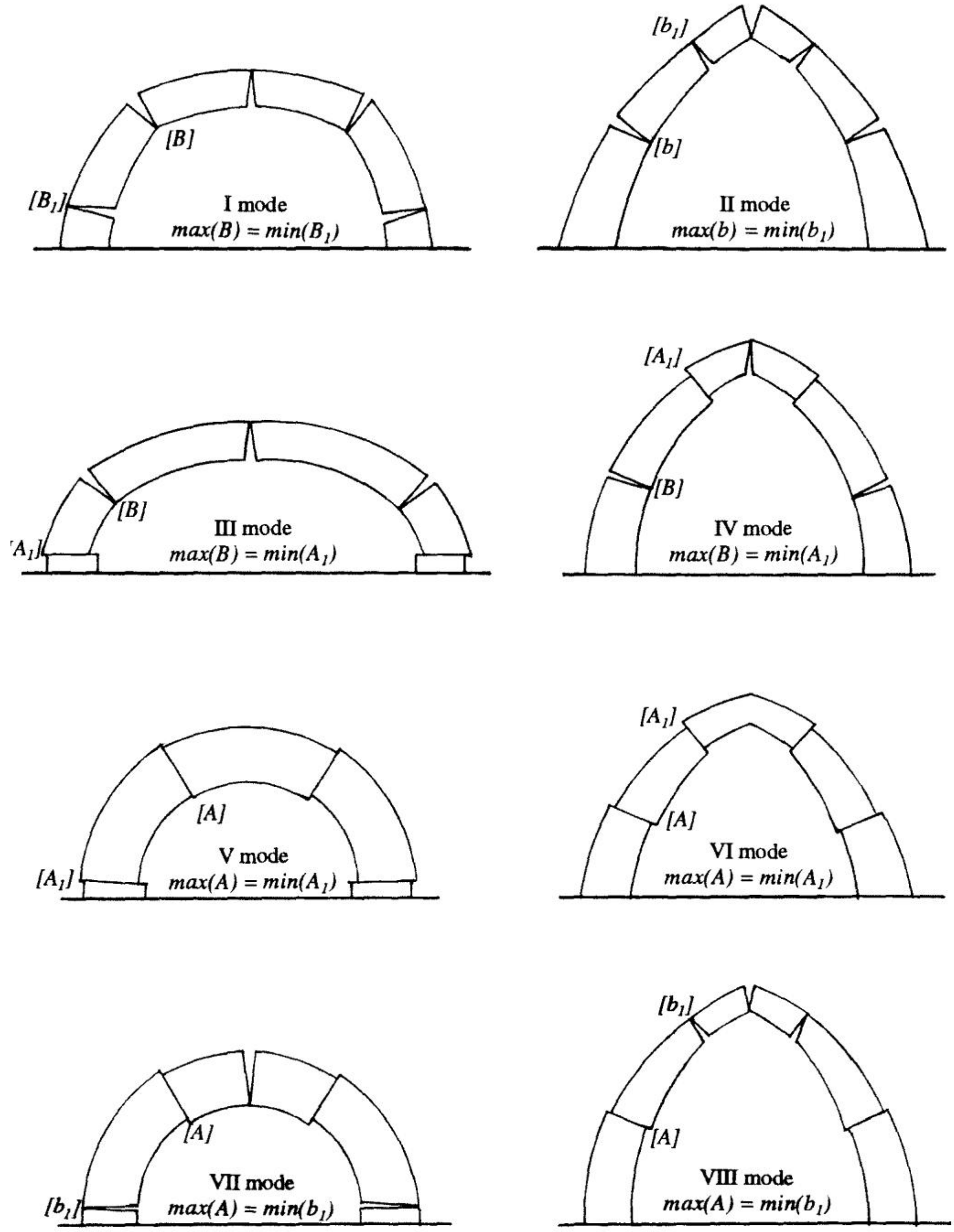


Fig. 2. 12 - The eight collapse modes of a symmetric arch according to Michon (Foce, Aita, 2003)

2.2.2 THE ELASTIC THEORY

The evolution from the rigid bodies mechanisms to deformable is well explained in Navier's job. Like the other authors, he studies the different types of collapses but he adds that voiussors are not *"parfaitement durs"* (Navier, 1826). The force cannot be only on the edge of rotation mechanism, but it needs a minimal area. He introduces the stress' concept as force per unit length. He considers no-tensile-resistant material and sees as limit condition the totally reactive section when joints are pre-cracking. In this condition the only distribution of stress compression that is possible is the triangular with force in the middle-third (Fig. 2. 13). The condition described by Navier is the local material's failure as elastic limit state and not the general collapse of the arch. To solve the problem is not sufficient to govern a statically indeterminate structure like arch the static equations. Navier (1826) sees the arch equilibrium as "*un cas particulier d'une question plus générale*". He understands that to find a solution the arch must be thought as a deformable body because, with this perspective more equations are available like the elastic that ties loads to displacements (Becci, Foce, 2002).

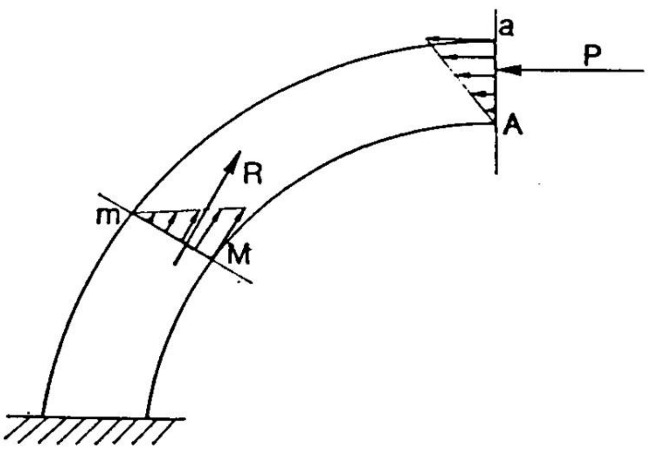


Fig. 2. 13 - Arch elastic limit condition according to Navier (Benvenuto, 1981)

The nineteen century is characterized by the research of physic - mechanic material's properties and by the development of elastic theory; this focus came also from the affirmation of new materials like steel and concrete. About arches, another crucial goal is to find the line of thrust that is developed within the thickness of the arch. The idea of thrust-line is introduced by Gerstner

(1831) and represents the polygon that ties pressure’s centers to each joint’s plane. This graphic method allows to get that the arch’s shape presents endless lines of thrusts that are possible and consequently endless equilibrium conditions too. If the thrust-line come out from ones arch’s extrados or intrados the crack is immediate; if it is tangent, it develops a rotation between voussoirs and this specific situation is possible only in endless resistance of material case. So, the arch is statically indeterminate structure (Fig. 2. 14) (Benvenuto, 1981).

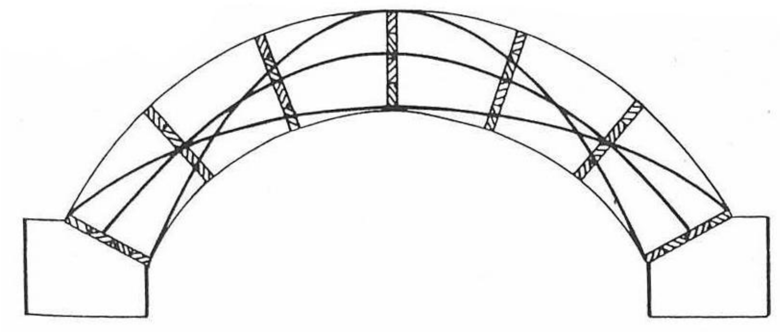


Fig. 2. 14 - Possible thrust lines according to Barlow (Heyman, 1982)

The analysis of arches with the thrust-line is used by a lot of authors, like Méry (1840) (Fig. 2. 15), Moseley (1843), Barlow (1846), Winkler (1867) and each one try to find out a method to define the shape.

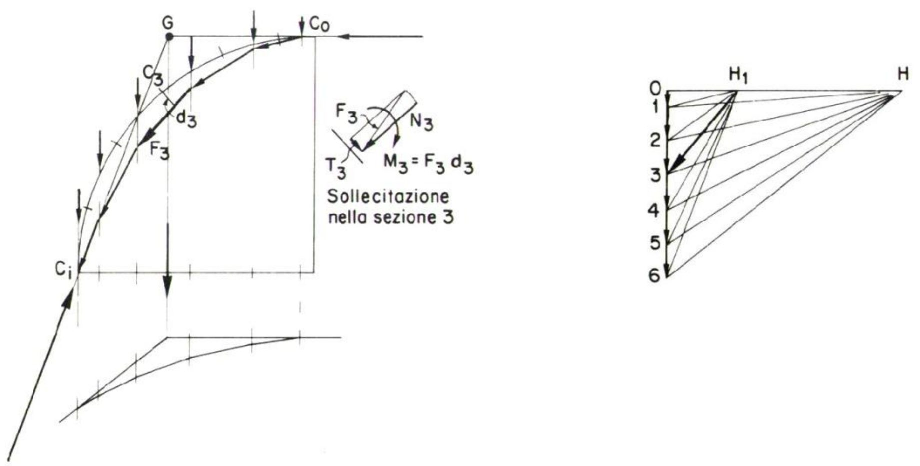
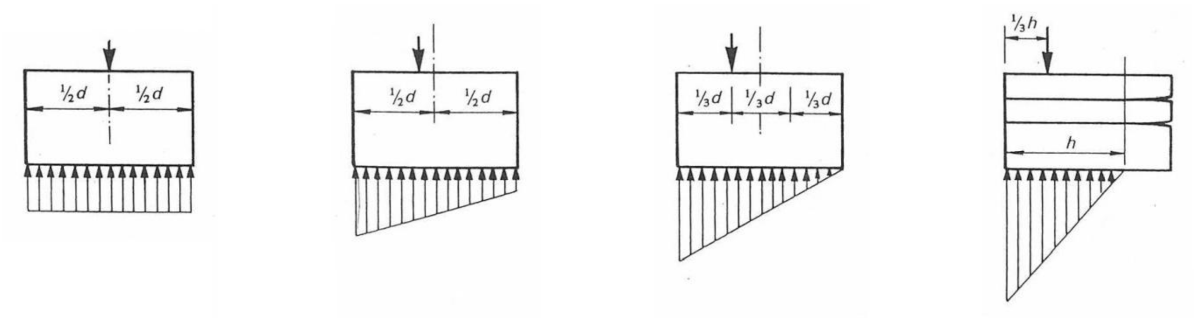


Fig. 2. 15 - Mery’s method (Giuffrè, 1986)

Castigliano (1879) develops an iterative method to determine the line of thrust. He considers the masonry as no-tensile-resistant (NTR) material and tries to define the reagent area of sections which have to be compressed and with a lower that yield stress (Fig. 2. 16) (Brencich, Morbiducci, 2007).



a) b) c) d)

Fig. 2. 16 - Tension distribution within cross section (Heyman, 1982)

The application of the elastic theory to masonry vaults makes a great deal of dubts to insiders due both to tricks adopted to avoid the problem of static indetermination and material properties (e.g. three-pin arch) and to the differences between theorical hypothesis and real conditions. Séjourné (1931) e Swain (1927) suggest many differences: first, vaults do not have the same resistance because they are composed by voussoirs and mortar; secondary the Young modulus changes with stress level; framework's movements alter randomly the line of thrust and boundary conditions can stress arch's conditions. Moreover, Swain leave as second aspect the focus of local material problem underlining the importance of global stability of structure (Becci, Foce, 2002).

2.2.3 LIMIT ANALYSIS

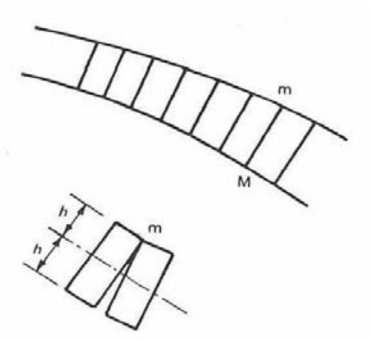
Beside the elastic analysis during the 1930s a new trend starts to develop in the structural analysis, starting from the observation of steel frame's behaviour. A lot of experiments show differences between the real reaction and the expected and from this point the plastic analysis was born. In fact the steel presents a ductile behaviour: it guarantee some strains that overgo the elastic field. This new field is where the strain is developed with energy consumption. About the structural safety the elastic analysis is based on a local evaluation of the problem, looking the stress that comes through in the material in a given set of forces; however, the plastic analysis considers the general equilibrium of the structure, trying to find out which are the forces what causes the collapse of the building when stock strength are ended. Kooharian (1953) uses for the first time this method on masonry arch, but Jacques Heyman (1966) uses this approach as general perspective (Becci, Foce, 2002). With a short list of hypothesis he extends the theory of plastic analysis to masonry structures and arches too. Heyman (1982) considers masonry with the following behaviour:

- masonry has an infinitive compressive strength;
- masonry has no tensile strength;
- sliding failure cannot occur.

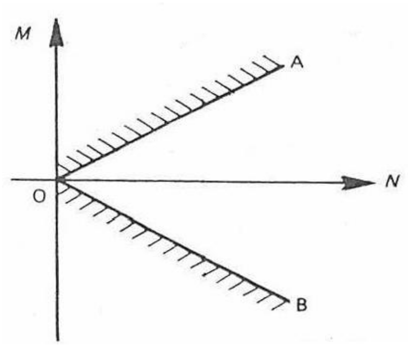
The masonry section can express:

1) a strength to the bending moment that result between the times from normal force and eccentricity of this force with the center line;

2) a ductile moment capacity that means a yield (plastic) hinges (Fig. 2. 17) (Gilbert, 2007).



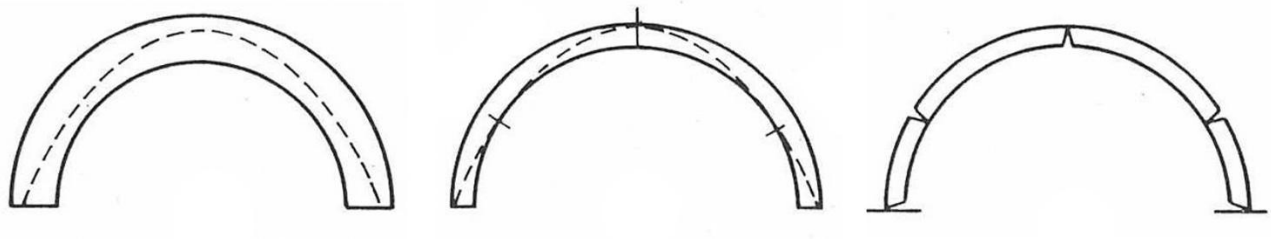
a)



b)

Fig. 2. 17 - Relation between normal force and moment in cross section (Heyman, 1982)

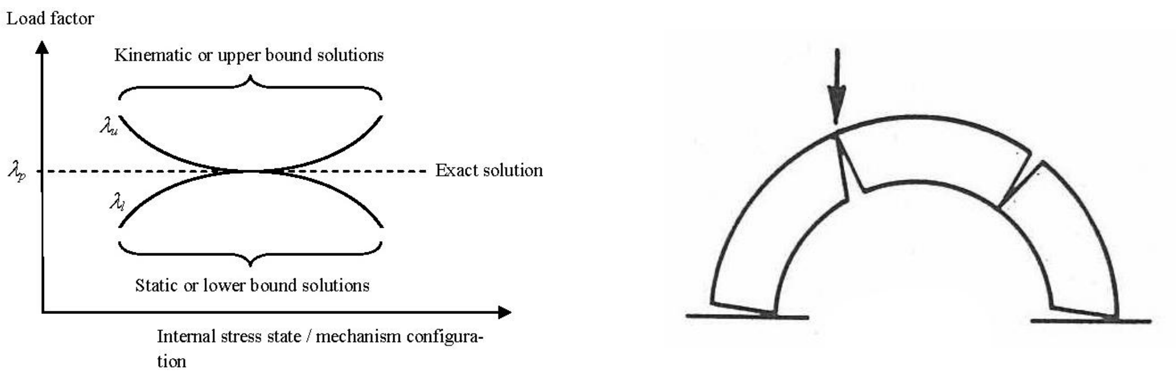
Moreover, the *safe theorem* (Heymann, 1982) says: “*If a thrust line can be found, for the complete arch, which is in equilibrium with the external loading (including self-weight), and which lies everywhere within the masonry of the arch ring, then the arch is safe”* (Fig. 2. 18 a). The safety assessment is not referred the effective condition of the arch; in fact, if we find a set of internal forces that satisfies the theorem the global structure of the bridge is stable. This theorem represents the lower bound theorem of plasticity. However if the line of thrusts is tangent to the extrados or to the intrados a hinge takes place (Fig. 2. 18 b). The arch is a structure that has three redundancies; so to convert it in a mechanism 4 hinges are need or 5 in case of symmetric load (Fig. 2. 18 c).



a) b) c)

Fig. 2. 18 - Thrust line and mechanism of arch (Heyman, 1982)

Consequently, given a load's forces we can figure out which is the most probable mechanism setting hinges on the arch's shape as the kinematic approach of limit analysis says. So, with the application of the principle of virtual work, we can find the value of the multiplier of loads and the constraining reactions. Drawing the thrust's line which passes through given hinges it has to be verified if it occurs within the arch thickness as said in the safe theorem. The solution of limit analysis when the collapse occurs is only one and it does not reflect small settlements of arch (Fig. 2. 19 a). The calculus is done with small displacements and fixed impost hypothesis.



a) b)

Fig. 2. 19 - a) Solutions of plastic analysis (Gilbert, 2007), b) possible mechanism with single force (Heyman, 1982)

The approach of Heyman allows to divert the problem of static indetermination of the structure: in fact deforming characteristic of the material and tension's state of the construction are not needed. When is built, the arch, might be considered as compound by rigid, hinged macro elements. Masonry constructions has strength reserve tied by the geometry and construction elements mass instead of material. The resulting shape takes a crucial role on equilibrium state maintenance. Heyman combines XVII° century studies about collapse with the line of thrust application, developing a strong theory with an easy application.

2.2.1 ADVANCED ANALYSIS METHODS

The research about masonry bridges is became so deep during the last decades. From one side Heyman’s theory is accepted; from the other the massive use of software to finite elements (in structure engineering) generates new horizons about the assessment of behaviour masonry bridges. These methods are summed up by Hughes (1996), Boothby (2001) and Lourenço (2002). A well developed job was performed by Sheffield University during the 1990s, particularly by Dr Matthew Gilbert. Based on lab-test on masonry bridges with full-scaled model, Gilbert develops in 1992 a software named RING to understand and analyze the results. The project had been continued and in 2011 the third edition was published. The software find out the collapse load of a masonry bridge and the referred mechanism, through the limit analysis. The problem is 2D view and the arch is seen as a rigid blocks system. Each block is tied with the other by restrains that allows sorted movements. Joints are thought as natural weakness’ planes of the structure (Fig. 2. 20). The model of discrete limit analysis was already adopted by Livesley (1978) (Gilbert, 2007).

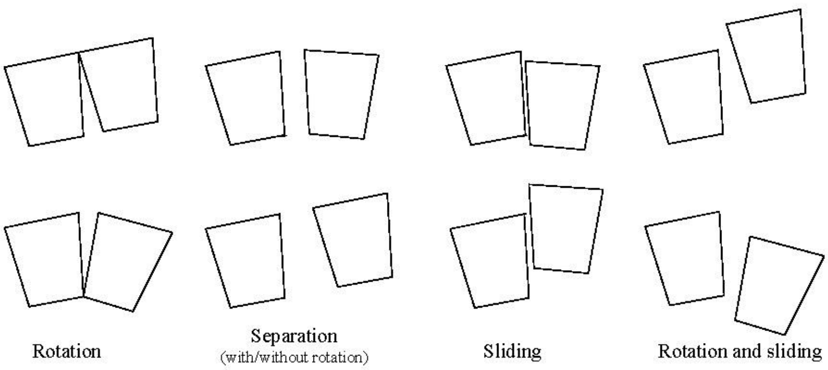


Fig. 2. 20 - Potential relative movements between blocks (Gilbert, 2007)

The software exploits the extensions of Heyman’s theory developed by Harvey (1988). The plastic analysis might be used also when the compressive strength is not infinite. The concept of thrust line diverts to thrust zone and this implicates a minimum width of cross section to transfer the load. Consequently the hinge takes place within the section instead of the edge supposing the material crushing. This assumption implicates that normal force and moment must be inside a dominium (Fig. 2. 21).

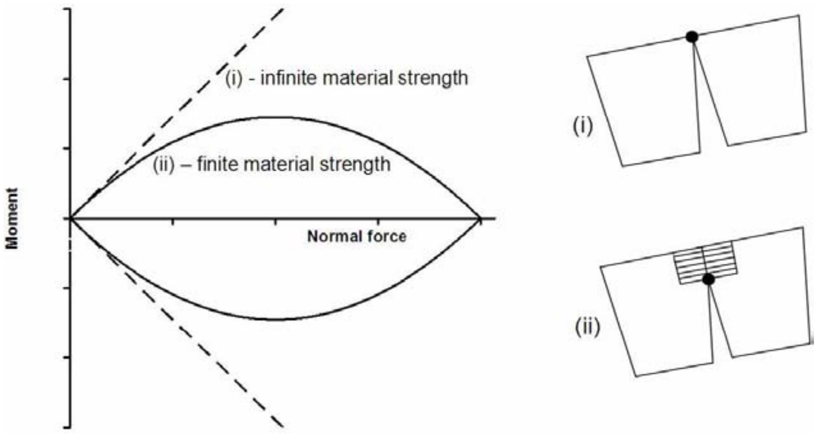


Fig. 2. 21 - Contact surface moment vs. normal force failure envelopes for: (i) infinite; (ii) finite masonry crushing strengths (Gilbert, 2007)

Harvey also develops the hypothesis of sliding failure. This mechanism can appears if the angle slope of the thrust line (or thrust zone) has a major slope's angle than *φ* (friction angle). RING allows to use the friction coefficient *μ* that is alike to *tan(φ)* and considers a sawtooth or associative fiction model that satisfies plastic limit analysis theorems (Fig. 2. 22).

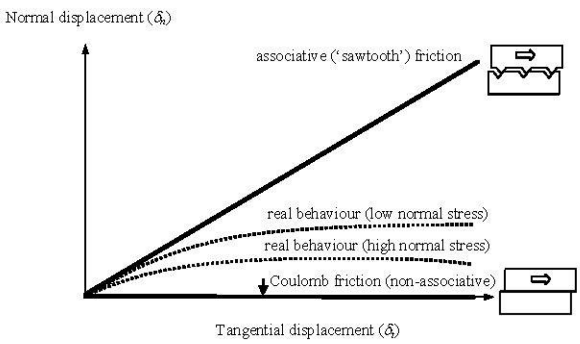


Fig. 2. 22 - Idealized sliding models and real behaviour of masonry joints (Gilbert, 2007)

Mechanism might be rotational, sliding or mixture (Fig. 2. 23).

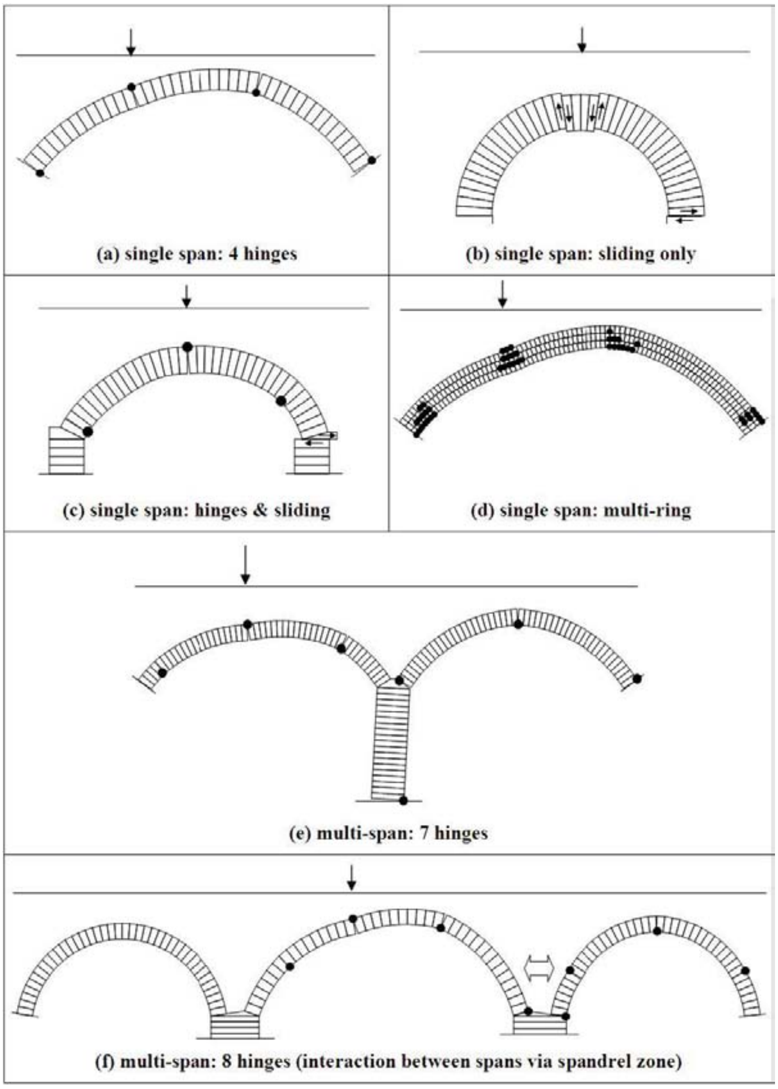


Fig. 2. 23 - Selection of potential failure modes (Gilbert, 2007)

The software allow to consider the interaction between the structure and backfill. This material disperse live loads and tend to restrain movement of the arch. The restrain is developed by one dimensional bar elements which work only with compression (Fig. 2. 24) (LimitState, 2014).

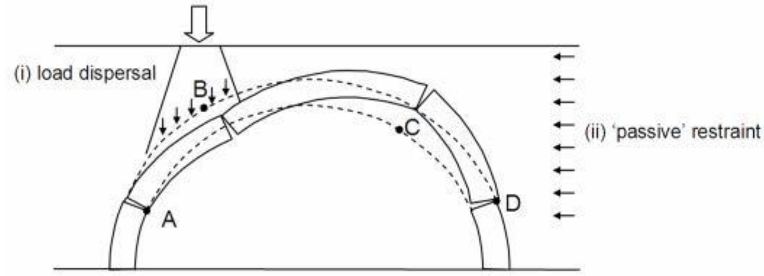


Fig. 2. 24 - Masonry bridge soil-structure interaction (LimitState:RING Manual, 2014)

The horizontal passive restraint is based on the classical lateral earth pressure theory and depends from the characteristics of the backfill, friction angle and cohesion.
The limit analysis application approach to the mechanism is used from a lot of authors (see Gilbert, Melbourne, 1994; Hughes, Blackler, 1995; Oliveira et al. (2010)). For example Crisfield and Packam (1985), Huges (2002) consider the interaction between arch and backfill; the soil pressure is thought as horizontal force added to the arch (Cavicchi, Gambarotta, 2004). Clemente (1998) develops the first study on dynamic bridge's behaviour. He adopts the analysis of mechanisms to the longitudinal orientation of the structure, trying to get the role of the backfill during the earthquake. About this topic Zampieri et al. (2013) use the kinematic analysis to the transversal direction, considering the 3D problem.

One more tool to make the structural analysis of masonry's bridge is offered by the Finite Element method. In finite element computations, the structure is divided, or discretized, into smaller elements, each with their own material properties. Relations between the nodal forces and displacements are known and the result is the assembly with the boundary conditions. The ultimate and definitive result is a system of equations whose solution can be used to compute nodal displacements as well as strains and stresses at integration points (Fig. 2. 25) (Bathe, 1982).

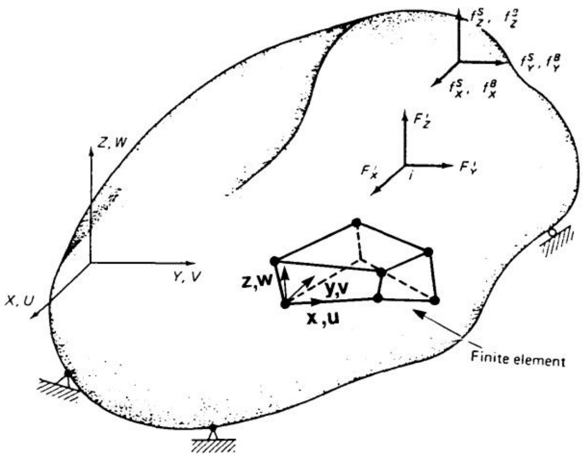


Fig. 2. 25 - General three-dimensional body (Bathe, 1982)

The structure can be shaped thanks to 1D element, e.g. beam; 2D shell; 3D brick (Fig. 2. 26).

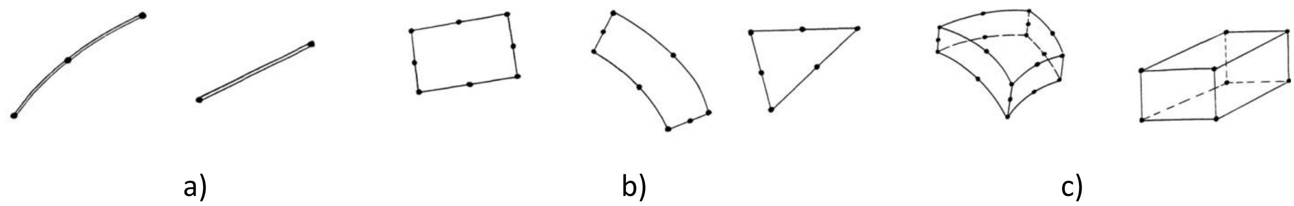


Fig. 2. 26 - Some typical continuum elements: a) 1D, b) 2D, c) 3D (Bathe, 1982)

The use of this tool comes from the need to get the evolution of the structural behaviour, from elastic to plastic and from the possibility to adopt different type of models (1D, 2D, 3D). The first examples of this application to the bridge analysis with masonry are given by Towler (1985) and Crisfield (1985) (Proske, van Gelder 2006). With this method start to be present physical - mechanical properties of the materials. Masonry is an heterogeneous material, compound by units (brick and stone) and joints (mortar), anisotropic with high compression strength, characterized by a non linear behaviour and softening phenomenon (Fig. 2. 27). This is typically of quasi-brittle material and consists in a gradual decrease of mechanical resistance under a continuous increase of deformation. The crack is due to a process of progressive internal crack growth.

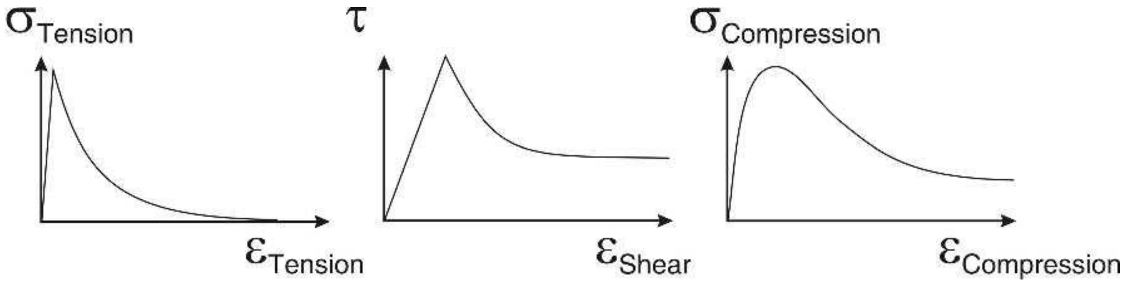


Fig. 2. 27 - Stress-strain behaviour of masonry according to Schlegel (2004) (Proske, van Gelder 2006)

These peculiarity make the finite elements methods so hard to carry out. Because of this problem many strategy of shaping are developed (Lourenço, 2002):

- detailed micro-modelling,

units and mortar in the joints are represented by continuum elements whereas the unit–mortar interface is represented by discontinuum elements (Fig. 2. 25 a);

- simplified micro-modelling or meso-modelling, expanded units are represented by continuum elements whereas the behaviour of the mortar joints and unit–mortar interface is lumped in discontinuum elements (Fig. 2. 25 b);

- macro-modelling,

units, mortar and unit–mortar interface are smeared out in a homogeneous continuum (Fig. 2. 25 c).

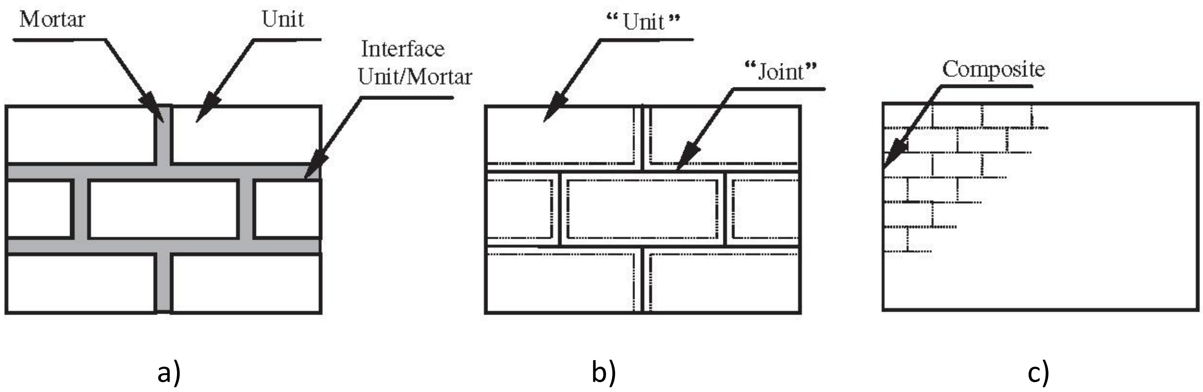
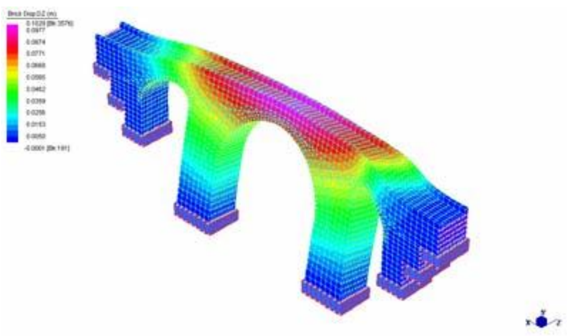


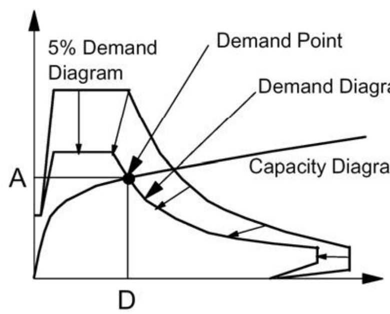
Fig. 2. 28 - Modelling strategies for masonry structures: (a) detailed micro-modelling; (b) simplified micro-modelling; and (c) macro-modelling (Lourenço, 2002)

The first and second approach allow to consider the natural weakness of joints of plane, approaching to the natural material's behaviour, but they need a great deal of input parameter and computational cost. There are many examples of contact element techniques with interfaces application for the masonry bridge's analysis like Fanning and Boothby (2001), Gago et al. (2002), Ford et al. (2003) and Drosopoulos et al. (2006) (Sarhosis et al., 2014). Usually the micro modeling is used with the objective to understand the local behaviour of masonry structure (Lourenço, 2002). Instead, if we consider the global reaction of the structure, the third approach is the most practical especially in the 3D modeling. Masonry is considered as homogeneous continuum material. The masonry is modelled as a homogeneous material so that average response properties. There are different options to make this process of homogenization like suggested by Pande et al. (1995) and Lourenço (1996). Different constitutive law can be applied to the material according to the structural analysis that we want to use. For example we can adopt the elastic-plastic method, considering the post-peak behaviour with one of the failure criterion material originally adopted for soils. The most common are Tresca, Von Mises, Mohr-Coulomb e Drucker-Prager methods (Nunziante et al., 2010), other are explained by Lourenço et al. (1998). Mohr-

Coulomb and Drucker-Prager are used (Pelà et al., 2009; Riveiro et al. 2010; Stavroulaki et al., 2016) for modeling masonry and backfill of bridges and they need just two parameters: friction angle φ e cohesion c. Continuum methods are usually used to determine the dynamic characteristics of a structure. Pelà et al. (2009) have used this approach to make the seismic assessment of masonry bridges combining the nonlinear static (pushover) analysis (Fig. 2. 29 a) with the response spectrum method in according to *performance-based* design philosophy (Priestley et al., 2007). The procedure used by Pelà is also known as N2 method; it is developed by Fajfar (2000) and represents an evolution of “Capacity Spectrum method” (Freeman et al.,1975; Freeman, 1998) in which the capacity of the structure is directly compared with the demand of the earthquake ground motion on the structure, it is reported by several modern codes, like Eurocode 8 (2004), OPCM 3274 (2003) (New Seismic Italian Code) and FEMA 440 (2005) (Fig. 2. 29 b). Resemini (2003) shows that the pushover analysis allows to a slightly overestimates in a conservative way the displacement obtained by nonlinear dynamic direct integrations (Pelà et al. 2009).



a)



b)

Fig. 2. 29 - Pushover analysis: a) FEM model by Pelà, b) N2 graphic (Bonaldo, 2014)

The topic of the interaction between the ground ant the arch has been observed with the FEM. For example Crisfield (1985) and Choo et al. (1991) model the lateral response of the fill by one-dimensional horizontal elements having an elastic–ideal plastic constitutive equation with different responses at active and passive fill states in the nonlinear incremental FEM models. Cavicchi and Gambarotta (2005) try to understand with lab test on the Prestwood Bridge (Page, 1987) and FEM model if the application of kinematic theorem of limit analysis is acceptable. In fact this theorem forecasts as hypothesis the endless ductility of cross sections. The study’s results show that the limit analysis can be applied with a good approximation in case of masonry’s bridge.

An alternative and appealing approach is represented by the Discrete Element Method (DEM), where the discrete nature of the masonry arch is truly incorporated. The advantage of the DEM is that it considers the arch as a collection of separate voussoirs able to move and rotate to each other. In fact, the finite element method can shows convergence problems especially under great cracks and assuming homogenous material properties over certain space regions cannot hold anymore. The DEM was initially developed by Cundall (1971) and was lately used to model masonry structures including arches (Lemos, 1995; Mirabella and Calvetti, 1998; Toth et al., 2009; Sarhosis and Sheng, 2014), where failure can realistically occurs along mortar joints (Sarhosis et al., 2014; Proske, van Gelder, 2006).

From masonry bridge analysis can be taken out some data typically owned by medial material properties. The following Tab. 2. 1 shows some values found by the literature which can be consider as reference ones deeply read because of the specifically masonry response given in each single case.

Tab. 2. 1 - Literature's average properties of materials

	Property	**Symbol**	**Unit**	**Oliveira et al. (2010)**	**Sarhosis et al. (2014)**	**Lourenço (2002)**	**Pelà (2009)**	**Page (1987)**	**Cavicchi e Gambarotta (2005)**
Masonry	Unit weight	γ	kg/m^3	2500	2700	2000	2200	2000	2000
	Friction angle	φ	deg	30			55		
	Cohesion	c	MPa			0.3	0.05		
	Young's modulus	E	MPa		5000	10000	6000 5000	15000	15000
	Poisson ratio	v	-		0.2	0.2	0.2		
	Compressive strength	f_c	MPa	5	unlimited	unlimited	4.5	4.5	12
	Tensile strength	f_t	MPa			0.2	0.3		
Backfill	Unit weight	γ	kg/m^3	2000		1500	1800	2000	1800
	Friction angle	φ	deg	30		37	20	37	30
	Cohesion	c	MPa				0.05	0.01	0.02
	Young's modulus	E	MPa				500	300	300
	Poisson ratio	v	-				0.2		

2.3 SAFETY ACCORDING TO CODES

The items of the structural safety referred to the existing constructions is the core of an intense discussion. The are many problems along this issue both because of the limited applicability of available codes and the characteristics of the existing building. Generally an existing construction is also a cultural heritage with an innate artistic and historical value which need a specific conservation to be preserved. The idea of preservation has been evolved through centuries combined with the use of the building. Preserve and use the architectural heritage of a territory means, in fact, strengthen cultural identity and, at the same time, preserve and possibly improve the quality of life and economic well-being of the community that lives in that territory. The use of buildings is strictly connected with a performance and requirements of structural safety as explained and implemented within codes; these elements usually fight with the idea of work preservation. A structure is considered safe because it has been "verified" that is "adequate" compared to pre-established levels of "resistance" or, more generally, of mechanical performance by a series of "precise" engineering operations (Modena, 2008). The verification of structural safety is more efficient in the design of new buildings where the work is not yet existing and thus is not already known: the process is thus applied on an ideal model. The calculations will be much closer to reality if the construction of the building will be carefully performed, the use will be appropriate, a proper maintenance will be performed, and the less the expected load conditions will deviate from what was forecasted in the design phase. The verification of an existing building in which theoretically is possible to measure all elements is difficult. The uncertainties related to the characteristics of the building are even less defined in statistical terms: on one hand the variability of the types of materials and construction techniques used in historic buildings; on the other hand there is a practical impossibility to perform appropriate tests (in terms of type and number of samples) and inadequacy of the available computational models. It is increasingly becoming clear that the logic of design choices must be different: in the case of new constructions the extra cost of a "conservative" design is actually marginal, while it may even be unacceptable in the case of an existing construction, especially if the execution of an intervention, that in some cases can also compromise details of artistic or historic value, could be avoided if more accurate assessments were possible. For the reasons above mentioned there is an important development of analysis tools (test methods, calculation models) specific for existing buildings which have increased the attention paid to the need to take full advantage of the fact that the existing building itself is a potential source of information on its structural behaviour and its actual safety conditions, and this information is more specific and meaningful comparing to those that can be provided by sophisticated calculation methods.

The verification of structural safety and the definition of interventions to do cannot be based solely on the results of calculations especially when these results are away from the real conditions of the work (ISO 13822). So the doctrine is passed to consider the state "verification" of safety for existing building to the "evaluation" in which the engineer is called to make a judgement on the real safety level of the work underlining how to improve the value of the safety or restrict the use of the work. The main goal is to guarantee an "acceptable" level of safety in which the work is, preserving the artistic and historical values; it would be better to talk about "risk" and the social and economic acceptance. The study method developed to solve the complexity of an existing building with a multidisciplinary tasks, can be scheduled in these steps: anamnesis, diagnosis, therapy and controls, corresponding respectively to the condition survey, identification of the causes of damage and decay, choice of the remedial measures and control of the efficiency of the interventions (Lourenço et al., 2015).

In whole process' phases take part qualitative values (such as historical investigation and inspection) and quantitative data (such as monitoring and structural analysis). The subjectivity is certain strong and defined and reflects to the final judgement; due of the reasons above mentioned the knowledge is extremely important like the flexibility on the model to use to make the structural analysis with different approaches options.

To underline the crucially of this topic, a great deal of documents at different levels (in terms of field of application and of degree of cogency) are been produced by scientific bodies in particular RILEM (Recommendation 1996) and ICOMOS-ISCARSAH (Recommendations, 2003), by standardization bodies such as ISO (ISO 2394, 1998; ISO 13822, 2010) , by the competent institutions in Italy (OPCM 3274, 2003; Guidelines, 2007; NTC 2008; Circolare, 2009) and at European level (Eurocode 8: Part 3, 2005; CEN TC 346).

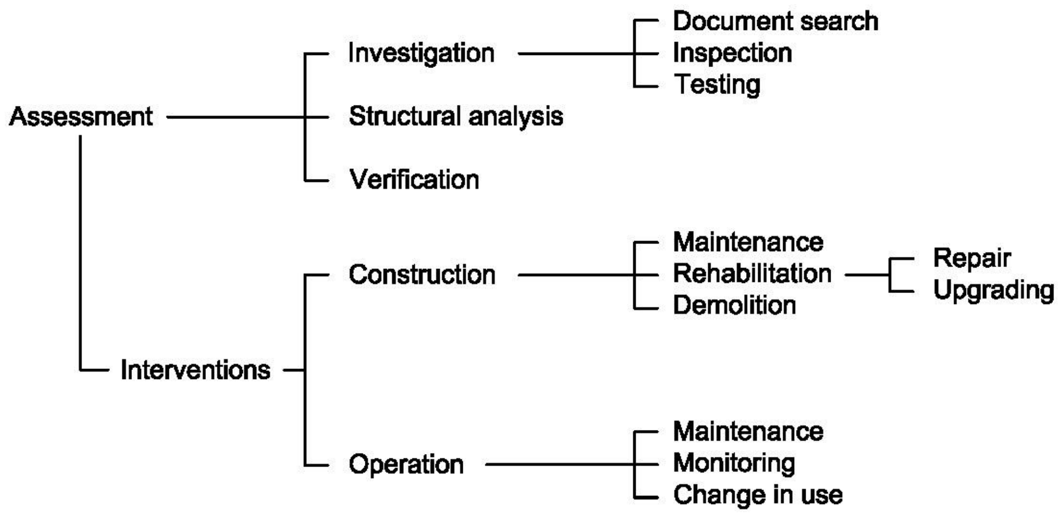


Fig. 2. 30 - Scheme of assesement pahses (ISO 13822, 2010)

Italy occupies a crucial role in this evolution both about the study of the structural behaviour of masonry (existing building) and for the development of rules and codes. This specific evolution had took place because of catastrophic consequences on the building which were cause by seismic phenomena. Those event had shown how the cultural heritage is in a critic conditions. In 1986 were published the firsts "*Norme tecniche relative alle costruzioni antisismiche*" which introduced the "improvement" ("*miglioramento*") through possible interventions on an existing building. The goal of this intervention in to improve the building's safety level without a substantial modification of the general behaviour. This idea is also considered in the following rules and codes in which the goal is giving a general approach and method to identify the problem of the safety in the existing building and moreover try to specifically define the contents of improvement as above mentioned. The OPCM 3274 (2003) and the "*Linee guida per la valutazione e riduzione del rischio sismico del patrimonio culturale"* (2007) ("*Guidelines for evaluation and mitigation of seismic risk to cultural heritage*") are documents which handle to reassume the effort given. Concepts were lately scheduled to "*Norme tecniche per le Costruzioni"* (NTC 2008) that represents the Italian governing law with a general approach. This law will be the reference for the case study of the present work and, for this reason, is necessary to analyze it. The main principles that base the NTC are stated at chapter 2, paragraph 1:

- the safety and performances of a work have to be evaluated referred to the limit state that might happens during the design working life; these can be ULS (Ultimate Limit

States) or SLS (Serviceability Limit States). The State Limit is the condition that, ones exceed, the structure no longer fulfils the relevant design criteria;

- for the existing building is possible to refer to different safety level instead of the new buildings and works;
- for the existing buildings only the ultimate state limit can be considered.

The safety evaluation have to be done with the semi-probabilistic method through the use of the partial factors (γ_M, γ_F) to apply to characteristic values action effects and resistances to find out the design values. Ones done is possible to compare the terms like in the following equation:

$$R_d \geq E_d$$

R_d ,design value of the resistance.

E_d ,design value of effect of actions.

As said we can get the importance assumed from the existing building for the Italian law; in fact is dedicated the whole 8 chapter in which the general criteria to define the analysis of this work are state. The scheduled procedure for the evaluation of the safety is established as follows:

- HISTORICAL AND CRITICAL ANALYSIS
  Investigation of various sources to find the building's process of making, modifications and events suffered by, with the final goal to identify the structural system and the stress state;
- SURVEY
  Complete description e redraw of geometry and structural parts of building with individuation and representation of damage's phenomena;
- MECHANICAL PROPERTIES OF MATERIALS
  Investigations with inspections and tests to define the material's properties considering the preservation status and damage of , that have to be used on the model for the analysis structure;
- LEVELS OF KNOWLEDGE AND CONFIDENCE FACTORS
  Based on the studies adopted the phases above mentioned, "knowledge level" will be found of referred to the difference parameters involved in the model (geometry,

constructive details and materials) and defined the confidence factors related which will have to be adopted as partial factors which counts model’s knowledge deficiency;

- ACTIONS

  Actions’ values and combination have to be considered in the calculation both for the safety evaluation and intervention’s project, are established for the new buildings.

Now the operator must defines and justifies the model and the method’s analysis for the evaluation of structural analysis. This evaluation has to point at the actual safety level allowing to state if:

- the use of the building can proceed without other interventions;
- the use has to be modified (downgrade, destination change and or new limits of use);
- is need to strengthening the structure system.

The law states three kind of intervention and establishes when it has to be applied (chap.8, par. 4). It says:

- <u>retrofitting:</u> when the achieved safety level is equal to the preview;
- <u>improvement:</u> when there is an upgrade of the safety;
- <u>repair:</u> when the intervention is focused and located.

The masonry existing building show some critical moments with the seismic actions with the apparitions of local and global mechanisms under those actions. The chap. 8, par. 7.1 of NTC point at the needs to evaluate of both mechanisms and find the limit analysis of equilibrium with kinematic approach as method to conduct this operation (linear kinematic analysis, for the evaluation of the horizontal action that activates the kinematism, and non-linear kinematic analysis, for the determination of capacity curve analyzing the evolution of the mechanism until the annulment of horizontal force). Chap. 7 introduces other methods:

- linear static analysis consists in the application of a gravity loads and a set of static forces equivalent to mass given from the seismic action to the building, which are distributed along the building height assuming a linear distribution of displacements;
- non linear static analysis (or pushover), is carried out under conditions of constant gravity load and monotonically increasing horizontal loads. At least two vertical distributions of

lateral loads should be applied: a "uniform" pattern, based on lateral forces that are proportional to mass regardless of elevation (uniform response acceleration); a "modal" pattern, proportional to lateral forces consistent with the lateral force distribution determined in elastic analysis. The result is a base-shear force and the control displacement graphic ("capacity curve");

- linear dynamic analysis: is referred to the determination of the different modes of vibration of the building; to the calculation of the seismic action's effects for each vibration mode, showed by each design model spectrum; to the combination of modal responses.
- non linear dynamic analysis, is defined as the calculation of the response of the structure may be obtained through direct numerical integration of its differential equations of motion, using the accelerograms to represent the ground motions.

The Circolare n. 617/2009 and the *"Linee Guida per la valutazione del rischio sismico del patrimonio culturale"* (DPCM 09/02/2011 renewed from 2007 version) are added from the precede rules and codes; they tend to explain clearly the issues just eposes. Both underline the importance of the knowledge level and the referred confidence factor. The Circolare defines as "structural evaluation" the quantitative process oriented to:

- establish if a structure con fight to the design's combinations of actions described in NTC or
- find the action value that the structure can support with the safety limit imposed by the NTC, using the partial factors of safety.

However it should not be consider the best practical solution to impose the intervention or the mutation of intend use or, extremely, the out servicing, if the construction is not adequate. The decision will have to be chosen according to the single and specific case (referred to the inadequacy level, or, generally, the economic and other implication to the public interest). The liability of the decision in the single decision will be on the owner; generally, for the cultural heritage the improvement intervention are able to combine conservation needs with safety.
About the evaluation on structural safety of existing bridges nowadays there is not enough specific regulation. To the Circolare, is annexed a chapter titled *"Indicazioni aggiuntive relative ai ponti*

esistenti” which consider concepts already explained about existing building (chap. C8, par. A8). In the Guidelines the topic is regulated in the paragraph “*Ponti in muratura, archi trionfali ed altri strutture ad arco*” (chap. 5, par. 4.5). In this document is described a different approach inherent to the existing building because of the indication about models of evaluation based to the construction type (churches, palaces, arches); these kind are characterized by common level of criticality and structural behaviour. The arch and vault under vertical loads collapse because of equilibrium loss with formation of hinges. About horizontal forces, the determination of the forecasted behaviour is more difficult because a systemic observation of post-quake are not diffused within the professional field. In this case is possible to consider the collapse with mechanism with hinges formation too. In the collapse study it has to be considered the backfill’s role if adequately constructed to avoid an excessive precautionary estimation. The arch’s structures are prove to be very sensitive in case of motion from imposed especially in greatly lighted arches. In masonry arch bridges local collapse mechanism are possible particularly in spandrel wall. To make an evaluation of seismic capacity is possible to use the finite element method with a detailed modeling of masonry arch considering non-linear constitutive law of material or kinematic approach of limit equilibrium analysis.

A specific document has been published by the Italian Consiglio Nazionale delle Ricerche (CNR) titled “*Istruzioni per la Valutazione della Sicurezza Strutturale di Ponti Stradali in Muratura*” (CNR-DT 213/2015). Also in this job appears the problem to understand the masonry bridge behaviour towards seismic action. The topic is highlighted in the introduction where is said that is not yet mature to formulate some Instruction. An acquired data is the consideration that masonry bridges do not show a particular sensibility to seismic action and big damages are not known. The document is scheduled following phases already mentioned to analyze existing construction pointing out the particular aspects of masonry bridges. For example the document says how to consider the fill and backfill in the structural analysis and how they can be seen (inactive material that spread accidental load or as elements that add resistance to the vault - chap. 3, par.8). for the seismic actions verification (chap. 3, par. 12) it must be consider that the structure’s failure is caused by its geometry variation that is aligned with the breaking down of masonry structure; so it has to be considered the “Displacement-Based-Design”, minding that the bridge feature during the waggle stays inside a given parameter. The preliminary evaluation of the bridge suitability has to be calculated trough the kinematic theorem of limit analysis (chap. 7, par. 3). About the structural modeling (chap. 8, par. 3) the finite element method allows to represent completely the

3D geometry of the bridge; the same for a plane modeling of the construction minding to a synthetic cross behaviour. Generally the structural analysis have to be filled by an analysis of the sensitiveness structural response main parameters assumed.

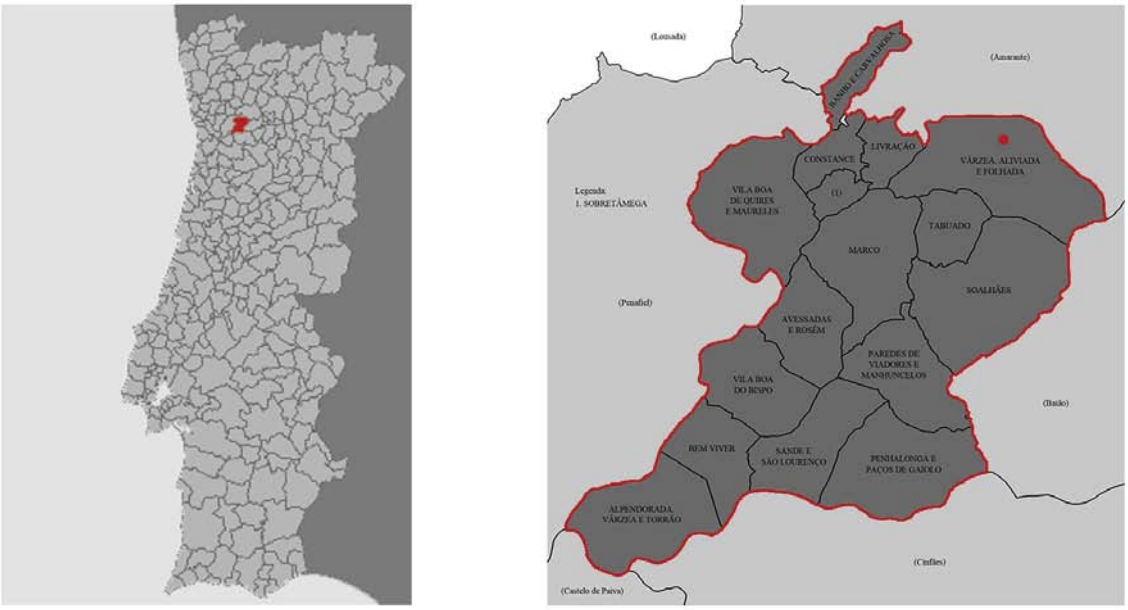
3. THE “PONTE DO ARCO”

In the masterpiece "Building Dwelling Thinking" M. Heidegger philosopher states: "Generally people think to a bridge as only a bridge". In this fragment, the philosopher takes the "symbolic" aspect of a construction which has always escorted the human genre. The bridge attaches a great deal of contrasting aspects: it "unifies" but at the same time it "divides"; is apparently stable but weak and dangerous too. It is suspended between two worlds and might be "alone" or "dwelled", it can "collapses" and "moves". It is an instruments to the world achievement by the human; by the way, the most sacrilegious work because it attacks the water, all cultures’ holiest element beyond the land (Cassani, 2014).

The operator who is going to make an evaluation of an historical bridge, must be aware by the analysis: the judgement that follows, will make a mark for the future and whatever is referred to, like daily myths and legends. Due of this, the approach adopted for the present work, has tried to discover as more aspect as possible to get an unlimited knowledge of the construction, according to time available; its behaviour has been scanned to complete the said general knowledge considering static loads and dynamic actions too. This chapter will explain the history starting from the documents to achieve the direct study of the building with the survey step with the main goal of getting data inherent the geometry, structure, conservation state and material used.

3.1 HISTORY

The *“Ponte do Arco”* masonry arch bridge is located in the north of Portugal, NEL district of *Porto* (Fig. 3. 1 a). It is built over the *Rio* (i.e.river) *Ovelha* and connects the banks of two parishes, *Folhada* and *Várzea da Ovelha e Aliviada*, in the current municipality of *Marco de Canaveses* (Fig. 3. 1 b). Until the 19th century, it stood at the heart of the municipality of *Gouveia*. 60 km distant da *Porto*, second bigger city of the Portugal .



a) b)

Fig. 3. 1 - *Carta Administrativa Oficial de Portugal (CAOP):* a) Municipalities of Portugal, b) Parishes of Municipality of Marco de Canaveses (www.dgterritorio.pt)

The bridge is located in a natural environment far away from the villages between the hills of *Rio Tâmega* and *Rio Douro* (one of the most important river of the Portugal) (Fig. 3. 2).

Together with the bridge of Aliviada, located downstream, the Bridge of Arco was part of a municipal or inter-parish network of roads that connected relatively close villages. The regional roads were located to the north (Amarante-Lamego) or to the south (Penafiel-Douro) and crossed, respectively, the bridges of Amarante- Padronelo and Canaveses (which no longer exists).

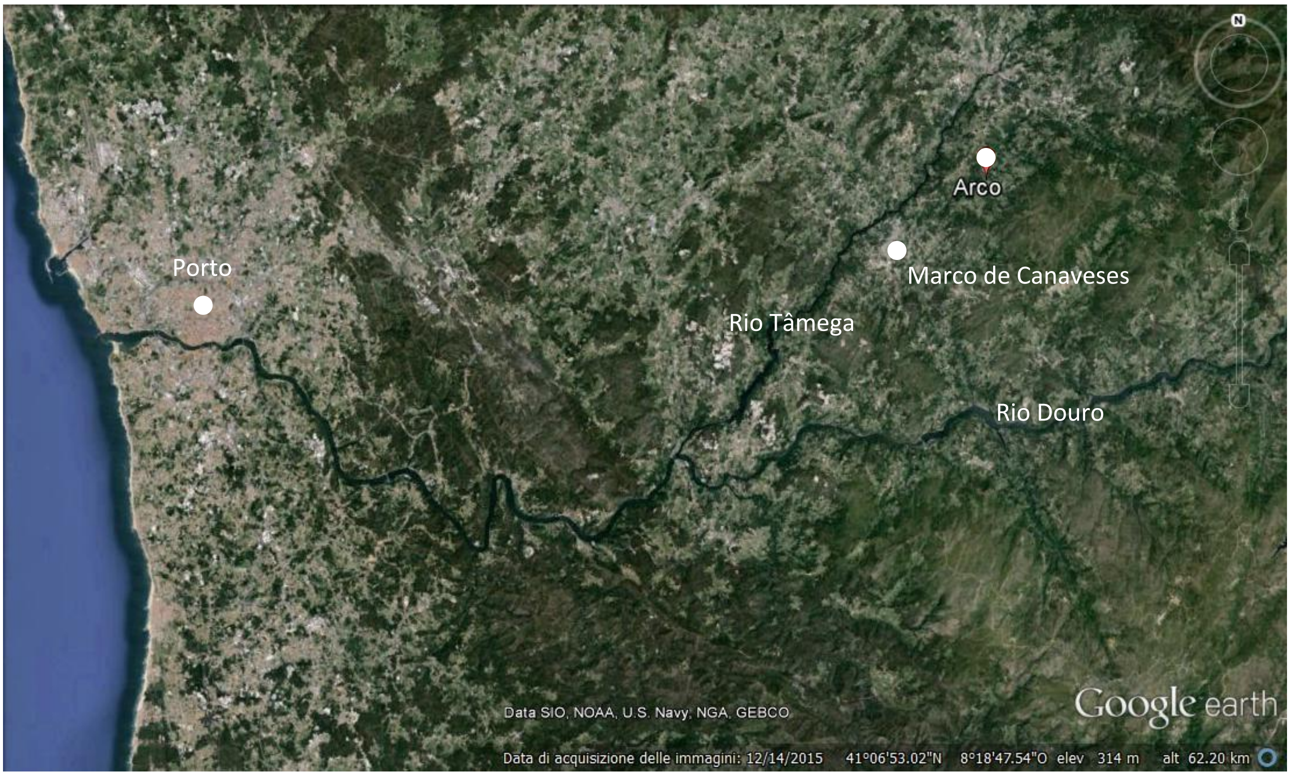


Fig. 3. 2 - Satellite Picture (Google Earth, 2015)

The topic of the communications transport network has been considered from many authors (Resende, 2014) to define the closer and trustable age of the bridge because there have not been found any documents or other papers that confirm it and overall because of the absence of initials in the wall faces of it.

The first mentioner was José Franco Bravo of Ponte do Arco abbot, the parish priest of Folhada who made a reference to it in 1758 using the following words:”*and features another great bridge at the end of this parish, called Ponte do Arco, by presenting a very large and hideous arch and very small guards. And because the bridge is not flat, since it is of stone, safe and old”* (Bravo, 1758). The same source says that the bridge “*serves this country and both its sides*”; due of those fragments is possible to state the hypothesis that this construction like other of the same period could be intended as complementary of the mentioned transportation network. Furthermore, despite the difficulties on fitting the exact dating of the construction, is not so distant from the reality to consider it as a late construction of the Middle Ages or as Modern when occasional and medium-distance journeys came possible (for instance processions, consecration of churches with tabernacles which required better roads and, consequently, suitable crossing). Finally we can highlight that the Ponte do Arco is located in a junction of multiple road branches derived from a major Medieval road of Tabuado, Soalhaes, and the hamlet of Giesta do Padroes in the 18th and 19th century.

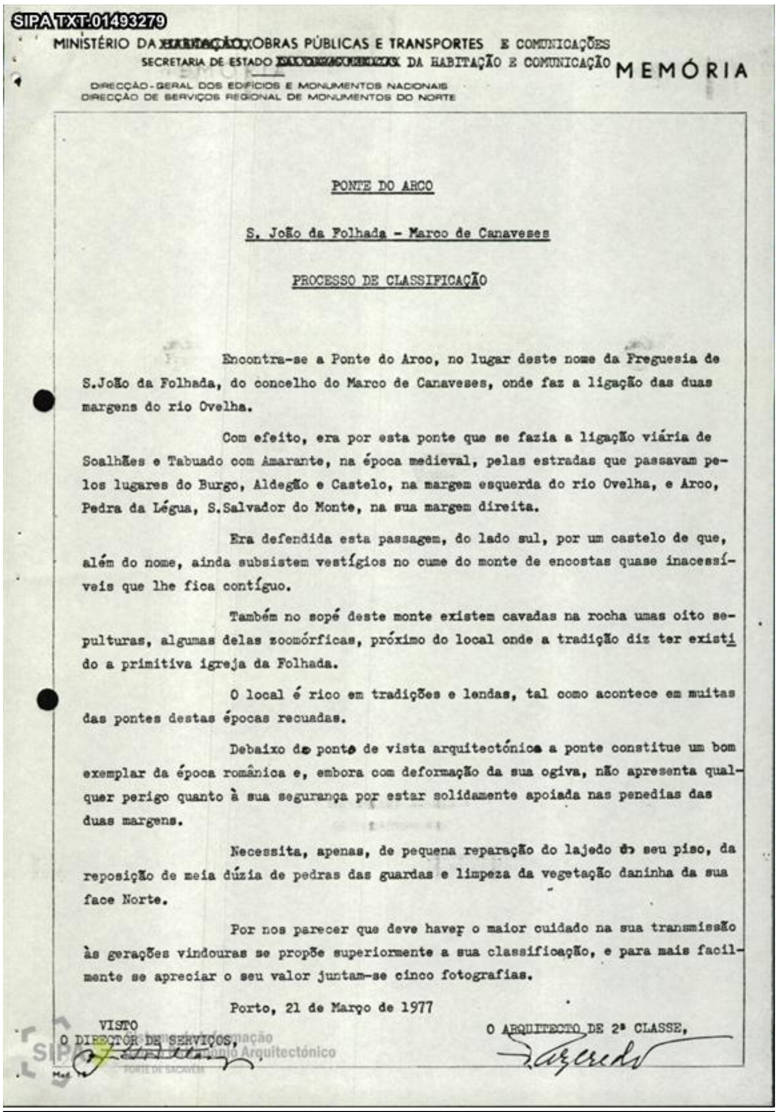
The feature and building method can be minded to establish the age of the building . The “*Ponte do Arco*” is a single and slightly pointed arch with a trestle-shaped elevation (Fig. 3. 3). Unfortunately is so difficult to associate totally the bridge with the traditional characteristics of the Gotic broken arch; this way has to be discovered to get some conclusion.

Fig. 3. 3 - Downstream side (Rocco, 2015)

Is possible to rewrite the inferred history thanks to the photographic sources as main. We can deduce some consideration, handling and appreciation and use that the locals had reserved to the this construction with the information gotten by the historical pictures. Now will follow some events linked with the “*Ponte do Arco”*.

1977 - Public Interest Building demand

During this year was demanded and required the classification as “Public Interest Building” fot the construction as highlighted from the annexed document; in the petition are scheduled the reasons and justification escorted by photos and documents as follow (Fig. 3. 4)



SIPA TXT.01493279

MINISTÉRIO DA ~~HABITAÇÃO,~~ OBRAS PÚBLICAS E TRANSPORTES E COMUNICAÇÕES

SECRETARIA DE ESTADO ~~[illegible]~~ DA HABITAÇÃO E COMUNICAÇÃO

MEMÓRIA

DIRECÇÃO-GERAL DOS EDIFÍCIOS E MONUMENTOS NACIONAIS
DIRECÇÃO DE SERVIÇOS REGIONAL DE MONUMENTOS DO NORTE

PONTE DO ARCO

S. João da Folhada - Marco de Canaveses

PROCESSO DE CLASSIFICAÇÃO

Encontra-se a Ponte do Arco, no lugar deste nome da Freguesia de S.João da Folhada, do concelho do Marco de Canaveses, onde faz a ligação das duas margens do rio Ovelha.

Com efeito, era por esta ponte que se fazia a ligação viária de Soalhães e Tabuado com Amarante, na época medieval, pelas estradas que passavam pelos lugares do Burgo, Aldegão e Castelo, na margem esquerda do rio Ovelha, e Arco, Pedra da Légua, S.Salvador do Monte, na sua margem direita.

Era defendida esta passagem, do lado sul, por um castelo de que, além do nome, ainda subsistem vestígios no cume do monte de encostas quase inacessíveis que lhe fica contíguo.

Também no sopé deste monte existem cavadas na rocha umas oito sepulturas, algumas delas zoomórficas, próximo do local onde a tradição diz ter existido a primitiva igreja da Folhada.

O local é rico em tradições e lendas, tal como acontece em muitas das pontes destas épocas recuadas.

Debaixo do ponto de vista arquitectónico a ponte constitue um bom exemplar da época românica e, embora com deformação da sua ogiva, não apresenta qualquer perigo quanto à sua segurança por estar solidamente apoiada nas penedias das duas margens.

Necessita, apenas, de pequena reparação do lajedo do seu piso, da reposição de meia dúzia de pedras das guardas e limpeza da vegetação daninha da sua face Norte.

Por nos parecer que deve haver o maior cuidado na sua transmissão às gerações vindouras se propõe superiormente a sua classificação, e para mais facilmente se apreciar o seu valor juntam-se cinco fotografias.

Porto, 21 de Março de 1977

VISTO
O DIRECTOR DE SERVIÇOS,

O ARQUITECTO DE 2ª CLASSE,

Fig. 3. 4 - Public Interest Building demand (SIPA)

1982 - Ponte do Arco is recognized as a Building of Public Interest

The process of legal protection advanced with an important step in Feb. 26th 1982 with then official recognition as “Building of Public Interest” by Decree n.28 published Governmental Official Gazette n.47 (Fig. 3. 5).

MINISTÉRIO DA CULTURA E COORDENAÇÃO CIENTÍFICA

Instituto Português do Património Cultural

Decreto n.º 28/82

de 26 de Fevereiro

De acordo, nomeadamente, com os artigos 2.º, 24.º e 30.º do Decreto n.º 20 985, de 7 de Março de 1932, do n.º 1 do § 1.º do artigo 19.º do Decreto n.º 46 349, de 22 de Maio de 1965, do n.º 1 do artigo 1.º e n.º 1 do artigo 2.º do Decreto-Lei n.º 1/78, de 7 de Janeiro, da alínea *a*) do artigo 2.º e alínea *a*) do artigo 9.º do Decreto-Lei n.º 59/80, de 3 de Abril, e do artigo 3.º do Decreto Regulamentar n.º 34/80, de 2 de Agosto, o Governo decreta, nos termos da alínea *g*) do artigo 202.º da Constituição, o seguinte:

Artigo 1.º São classificados como monumentos nacionais os seguintes imóveis:

Distrito de Bragança:

Concelho de Mirandela:

Ponte de Pedra, sobre o rio Tuela, 3 km a oeste da Torre de Dona Chama.

Distrito do Porto:

Concelho de Amarante:

Casa do Carvalho, situada em São Salvador, na freguesia de Real.

Igreja de S. Pedro, incluindo a sacristia, bem como os retábulos de talha e os azulejos que revestem o interior, em Amarante.

Solar de Vila Garcia ou de Igreja, situado na freguesia de Vila Garcia.

Concelho de Felgueiras:

Cruzeiro do Bom Jesus de Barrosas, no lugar de Barrosas, freguesia de Idães, junto à estrada de Lousada a Felgueiras.

Concelho de Marco de Canaveses:

Ponte do Arco, no lugar do Arco, freguesia de São João da Folhada.

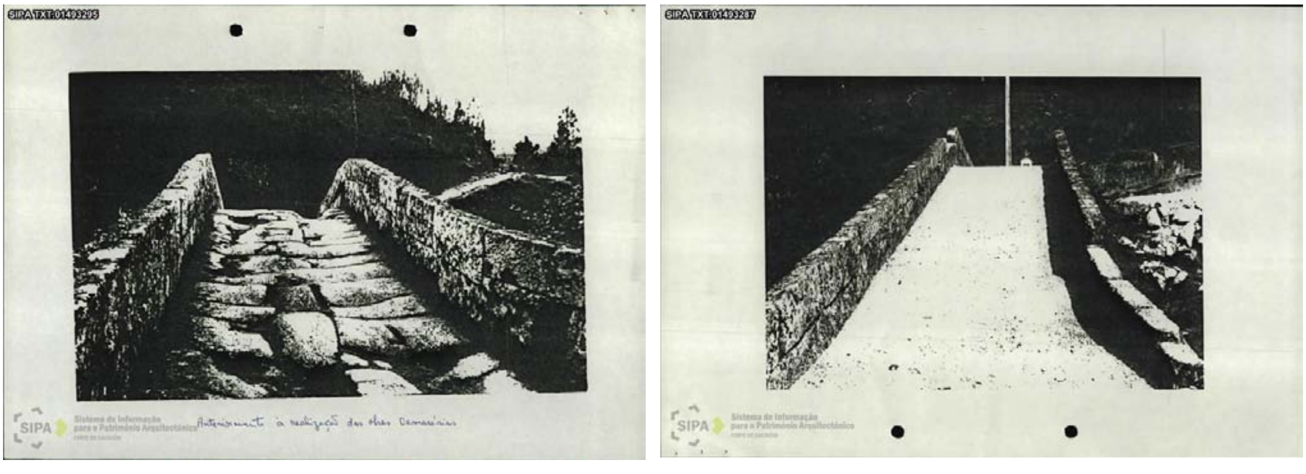
Concelho de Matosinhos:

Igreja paroquial de Matosinhos, incluindo o seu recheio.

Fig. 3. 5 - Ponte do Arco is recognized as a Building of Public Interest (SIPA)

1985 - Pavement intervention

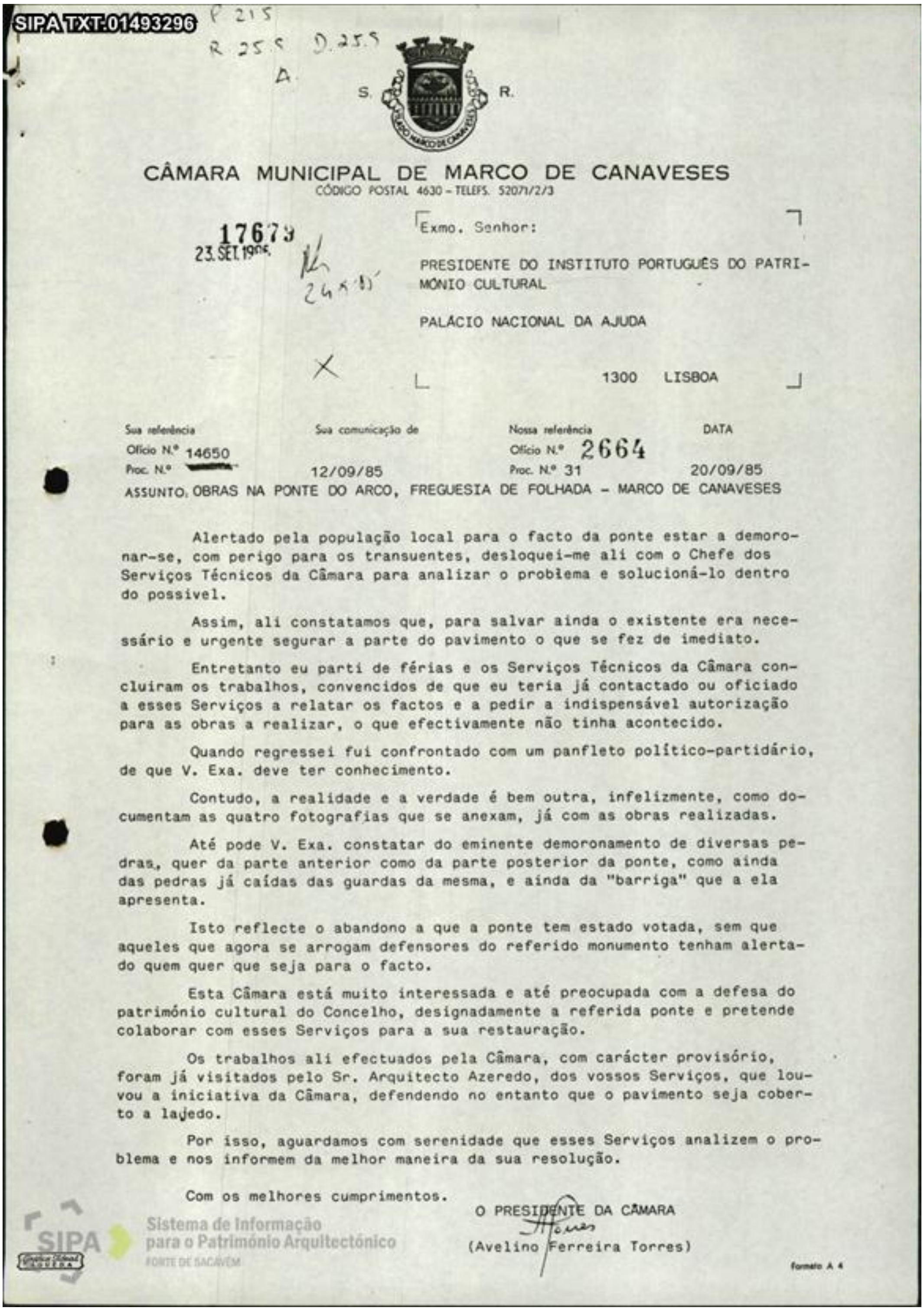
Due of the conservative state of pavement slabs that frightened the population, the Major approved an act to start an immediate intervention with the audition gotten by the Istituto Portugûese do Património Cultural: once taken, the paved restoration was made above the original layer of stones (Fig. 3. 6).



a) b)

Fig. 3. 6 - Pavement intervention: a) before; b) after (SIPA)

The following document shows the debate that occurred by a decision of the municipality to approve another intervention without respecting the procedure; however, despite the promise of a new layer stones, the pavement was left and today with can still see the same concrete.



S. R.

CÂMARA MUNICIPAL DE MARCO DE CANAVESES
CÓDIGO POSTAL 4630 – TELEFS. 52071/2/3

17673
23.SET.1985

Exmo. Senhor:

PRESIDENTE DO INSTITUTO PORTUGUÊS DO PATRIMÓNIO CULTURAL

PALÁCIO NACIONAL DA AJUDA

1300 LISBOA

Sua referência	Sua comunicação de	Nossa referência	DATA
Ofício N.º 14650		Ofício N.º 2664	
Proc. N.º [redacted]	12/09/85	Proc. N.º 31	20/09/85

ASSUNTO: OBRAS NA PONTE DO ARCO, FREGUESIA DE FOLHADA – MARCO DE CANAVESES

Alertado pela população local para o facto da ponte estar a demoronar-se, com perigo para os transuentes, desloquei-me ali com o Chefe dos Serviços Técnicos da Câmara para analizar o problema e solucioná-lo dentro do possível.

Assim, ali constatamos que, para salvar ainda o existente era necessário e urgente segurar a parte do pavimento o que se fez de imediato.

Entretanto eu parti de férias e os Serviços Técnicos da Câmara concluiram os trabalhos, convencidos de que eu teria já contactado ou oficiado a esses Serviços a relatar os factos e a pedir a indispensável autorização para as obras a realizar, o que efectivamente não tinha acontecido.

Quando regressei fui confrontado com um panfleto político-partidário, de que V. Exa. deve ter conhecimento.

Contudo, a realidade e a verdade é bem outra, infelizmente, como documentam as quatro fotografias que se anexam, já com as obras realizadas.

Até pode V. Exa. constatar do eminente demoronamento de diversas pedras, quer da parte anterior como da parte posterior da ponte, como ainda das pedras já caídas das guardas da mesma, e ainda da "barriga" que a ela apresenta.

Isto reflecte o abandono a que a ponte tem estado votada, sem que aqueles que agora se arrogam defensores do referido monumento tenham alertado quem quer que seja para o facto.

Esta Câmara está muito interessada e até preocupada com a defesa do património cultural do Concelho, designadamente a referida ponte e pretende colaborar com esses Serviços para a sua restauração.

Os trabalhos ali efectuados pela Câmara, com carácter provisório, foram já visitados pelo Sr. Arquitecto Azeredo, dos vossos Serviços, que louvou a iniciativa da Câmara, defendendo no entanto que o pavimento seja coberto a lajedo.

Por isso, aguardamos com serenidade que esses Serviços analizem o problema e nos informem da melhor maneira da sua resolução.

Com os melhores cumprimentos.

O PRESIDENTE DA CÂMARA

(Aveline Ferreira Torres)

Formato A 4

Fig. 3. 7 - Decision of *Câmara Municipal* adopted after intervention

Fig. 3. 8 - Photos late 80s (SIPA)

2007 - Traffic closing

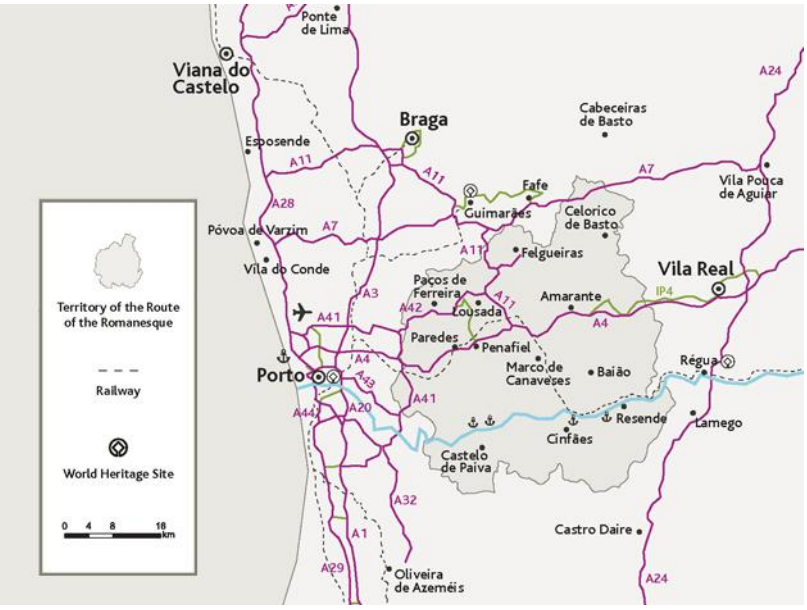
In an article published on *Jornal de Notícias* (Orlando, 2007) the *Câmara do Marco de Canaveses* forbidden to the traffic to cross “*Ponte do Arco”*. This act was adopted as preventive method to save the ruin condition of the bridge because there was not a sufficient level of safety. Despite of the prohibition, people has always been crossed the bridge that still continue; to stop definitely the traffic without a law act, the *Câmara* would have needed a sponsor (Fig. 3.9)

.

Fig. 3. 9 - 2006 (SIPA)

2010 - <u>The *"Ponte do Arco"* becomes part of the *"Rota do Romanico"*</u>

"Rota do Romanico" is a Route which linkes many buildings like churches, towers, monastery and bridges built during the Romanesque Period. The *Route* is located in the land of the valleys of Sousa, Tâmega and Douro, the heart of the North of Portugal (Fig. 3. 10). This heritage is structured in the Route of the Romanesque, germinated, in 1998, within the municipalities that comprise the VALSOUSA - *Associação de Municípios do Vale do Sousa* [Association of Municipalities of Vale do Sousa] and extended, in 2010, to the remaining municipalities of the NUT III - Tâmega, thus bringing together in a supramunicipal project a common historical and cultural legacy, with the purpose of conserving and appreciating the local cultural heritage (www.rotadoromanico.com).



a) b)

Fig. 3. 10 - *Rota do Romanico*: a) map; b) information tab on the site (www.rotadoromanico.com)

2015 - <u>Intervention for the conservation and protection of the *“Ponte of Arco”*</u>

Thank to the scope of the Route of the Romanesque, the need to schedule a maintenance (routine intervention to preserve appropriate structural performance according to ISO 13822) is particularly felt. For example cleaning operation of surfaces and vegetation removing has already accomplished and parapet addition with stones where lost (Fig. 3. 11).

Fig. 3. 11 - Upstream side, 2015

3.2 GEOMETRIC SURVEY AND TESTING

According to the document search, as done in precedent paragraph, we have not reached original draws or any other kind of surveys referred to the bridge geometry; due of this it had to been necessary to proceed with an accurate geometric survey of the work on the field. To accomplish this operation and get as more data as possible it had been decided to combine the use of photogrammetry with classical surveying instruments like rules, tapes, laser distance meter and plumb and with the Ground Penetrating Radar (GPR) too. Ones done, it was proceeded with the bridge’s drawings (plan, sections and elevations) and the following geometry and structural description.

3.2.1 PHOTOGRAMMETRY

Photogrammetry is the science of making measurements from photographs. The output of photogrammetry is typically a map, drawing, measurement, or a 3D model of some real-world object or scene. There are two types of photogrammetry: Aerial Photogrammetry when the camera is mounted in an aircraft and is usually pointed vertically towards the ground; and Close-range Photogrammetry (or Image-Based Modeling) when the camera is close to the subject and is typically hand-held or on a tripod (http://www.photogrammetry.com).

Because they are more balanced in terms of cost and accuracy, photogrammetric techniques and in particular the Image-based measurements techniques play an important role in engineering disciplines since they can provide amount of qualitative and quantitative information and knowledge about observed objects in a global, non-contact way with high spatial resolution (Riveiro et al., 2010). The final goal of a photogrammetric process is to obtain a set of 3D coordinates of points on the surface of the object in order to build 3D digital models of the object that represent its geometry. This is made possible by the exploitation of the Structure from Motion concept (Fig. 3. 12) and the referred algorithms developed in Computer Vision (to a deeply study of photogrammetry theory, see Cooper, Robson, 2001 and Zhizhuo, 1990).

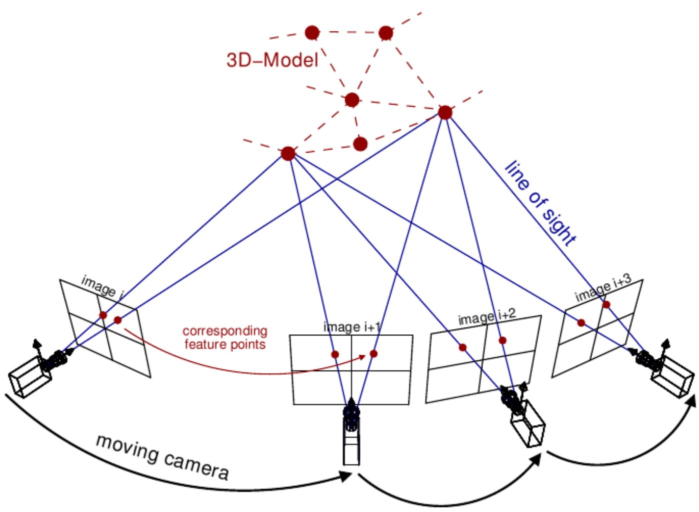


Fig. 3. 12 - Photogrammetry operations (http://www.theia-sfm.org/sfm.html)

The main role in this process is taken by the images acquisition phase because of the whole following job depend from the capturing. The following tab explains general principles of taking and selecting pictures that provide the most appropriate data for 3D model generation (Agisoft, 2014):

- EQUIPMENT
    a) use a digital camera with high resolution (5 MPix or more) and take images at maximal possible resolution;
    b) fixed focal length and avoid using flash;
- SCENE REQUIREMENTS
    a) avoid not textured, shiny, mirror or transparent objects;
    b) avoid absolutely flat objects or scenes;
- CAPTURING SCENARIOS
    a) to guarantee enough image overlap across the input dataset (60% of side overlap + 80% of forward overlap at least);
    b) using the correct acquirement methods based on the object that has to be analyzed been as distance as possible;
    c) good lighting e uniform is required to achieve better quality of the results;
    d) using at least two markers with a known distance between them on the object or alternatively, placing a ruler within the shooting area.

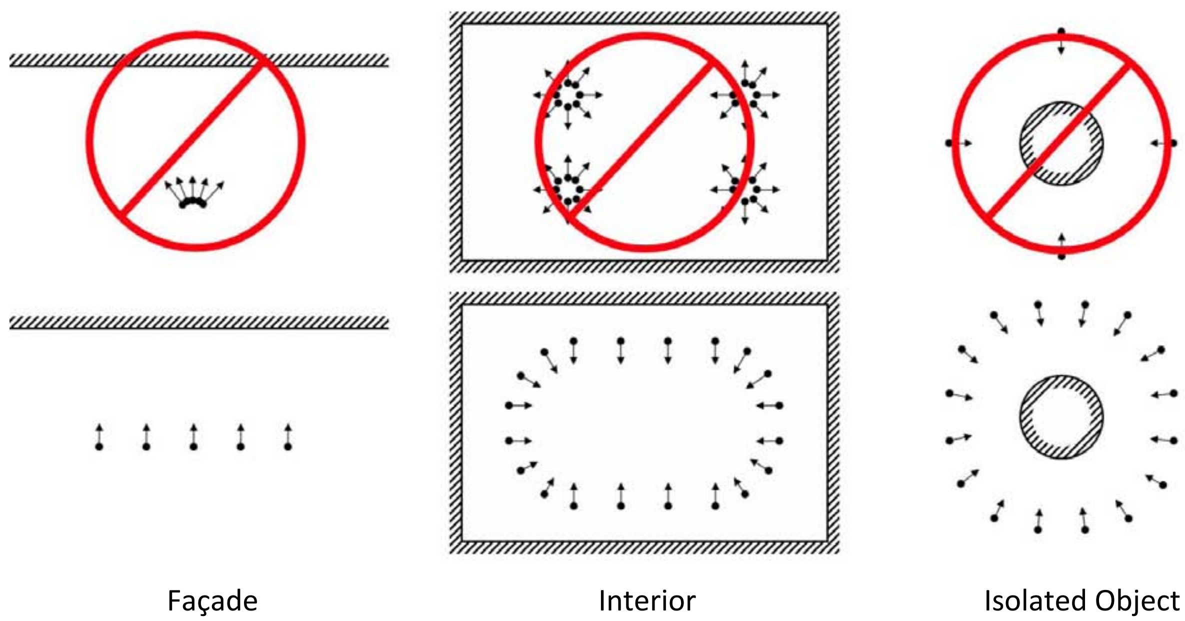


Fig. 3. 13 - Capturing scenarios (Agisoft, 2014)

Ones captured, images have to be evaluated to choose which must to build 3D digital models. The Image-Based Modeling process can be divided in three main phases according to De Luca (2011):

1- MAP REFERENCE POINTS

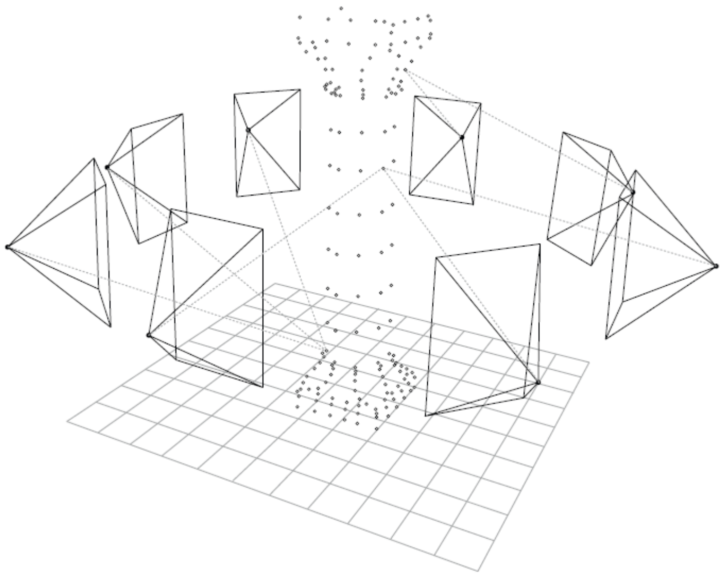


Fig. 3. 14

2- 3D MODEL RECONSTRUCTION

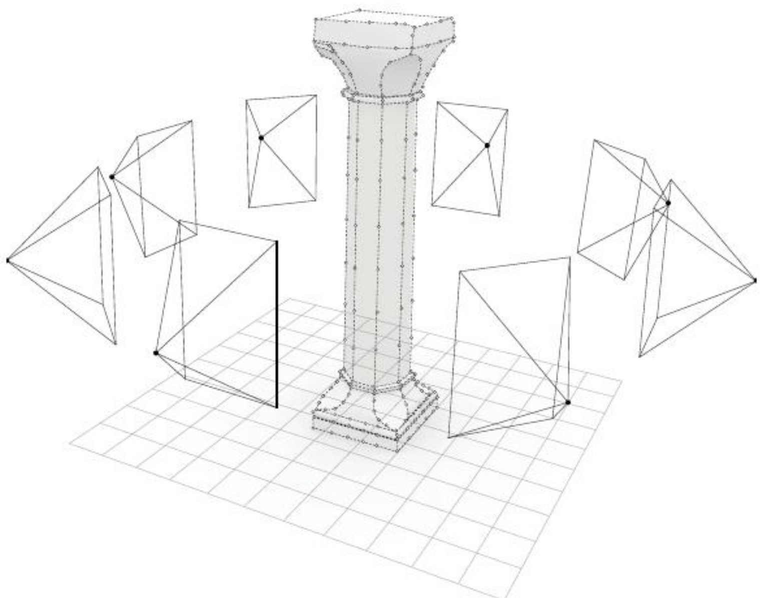


Fig. 3. 15

3- TEXTURE 3D MODEL

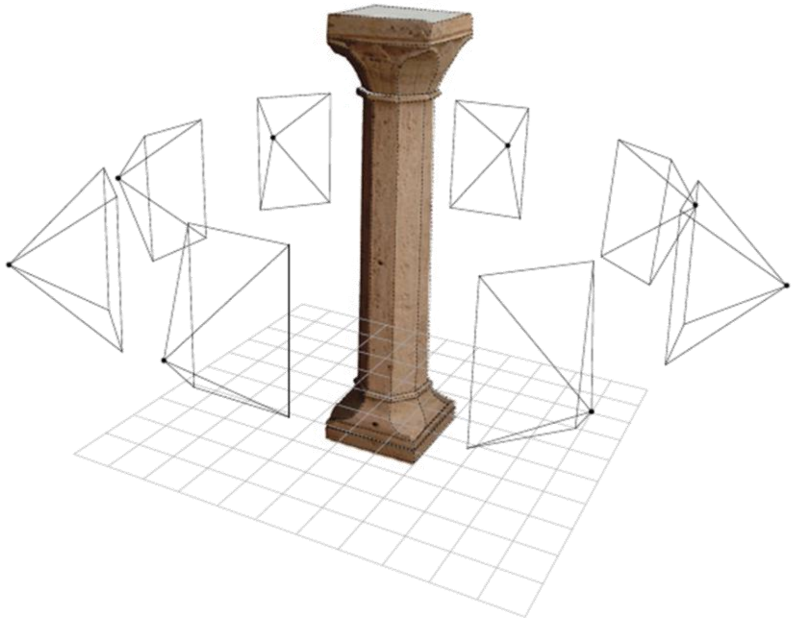


Fig. 3. 16

Ones got how photogrametry works, it had been decided to applies this technique to the study case. In fact the method was already used to analyze masonry bridges (see Jauregui et al.,2005; Riveiro et al., 2010). To accomplish this phase two surveys on field took place: Oct.16th and Nov.25th 2015.

The first survey had been necessary to get the as high number of information and datas as possible inherent the global geometry of the bridge.

The second, furthermore, had been needed to verify the quality model gotten with the photogrammetry to achieve the conservation level of the bridge and to make the GPR test.

Following the picture taken and acquired during the survey:

Fig. 3. 17 - Downstream side photos, Oct.16th 2015

Fig. 3. 18 - Upstream side photos, Oct.16th 2015

Fig. 3. 19 - Reference, Oct.16th 2015

Capturing the pictures during the survey with the criteria above described, was not so easy at all as thought because of environment’s conditions (trees, stones, river, gap of the field) that fought with the survey needs. Even thought the conditions were not so comfortable, we handled and got a wide number of information to use in process of modeling. Pictures were taken with a no-flashed reflex camera (Canon EOS 1100D, 10 MPix), with focal length of 18 mm.

The software is named Agisoft PhotoScan. It is an advanced image-based 3D modeling solution aimed at creating professional quality 3D content from still images.
Il software works with four main stages to get the final model (Agisoft, 2014):

1- ALIGNING PHOTOS

   Once photos are loaded into PhotoScan, they need to be aligned. At this stage PhotoScan searches for common points on photographs and matches them, as well as it finds the position of the camera for each picture and refines camera. As a result a sparse point cloud and a set of camera positions are formed.

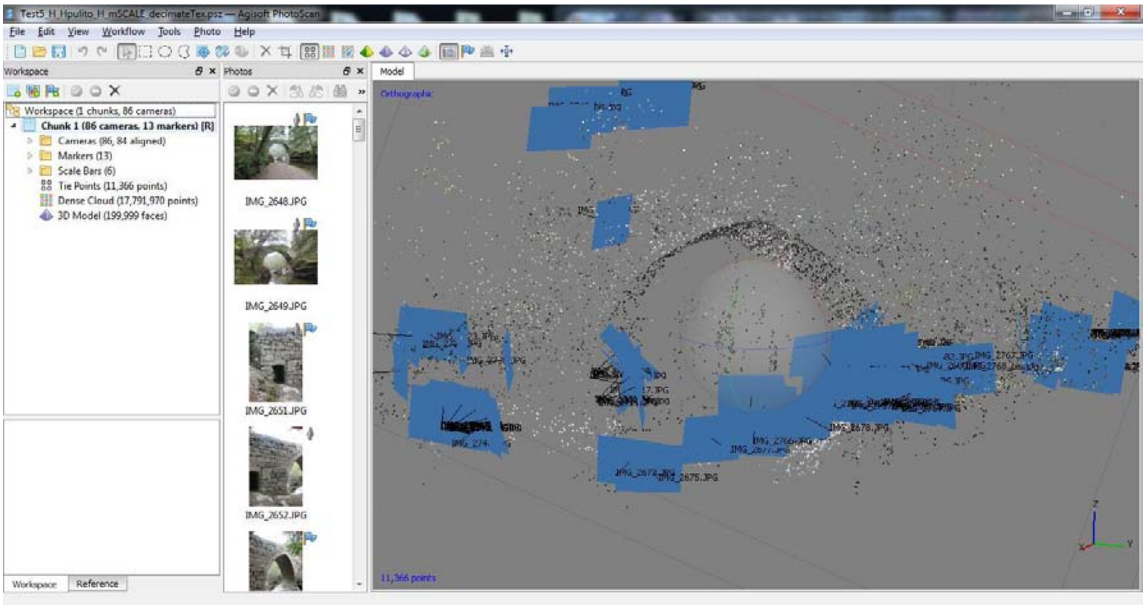


Fig. 3. 20

2- BUILDING DENSE POINT CLOUD

Based on the estimated camera positions the program calculates depth information for each camera to be combined into a single dense point cloud.

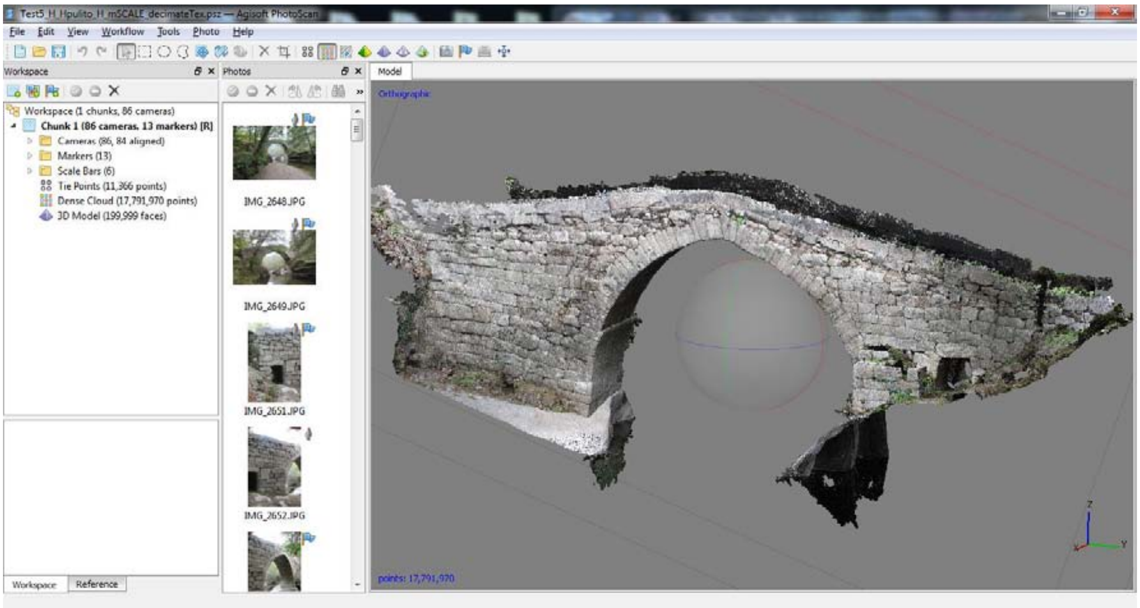


Fig. 3. 21

3- BUILDING MESH

PhotoScan reconstructs a 3D polygonal mesh representing the object surface based on the dense point cloud.

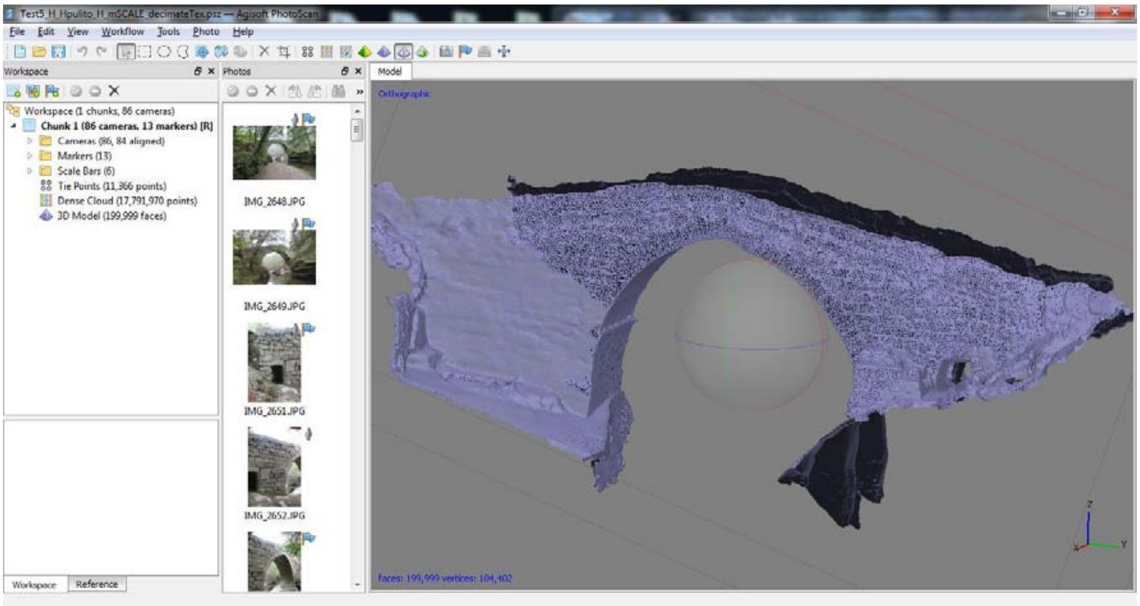


Fig. 3. 22

4- BUILDING MODEL TEXTURE

After geometry (i.e. mesh) is reconstructed, it can be textured and/or used for orthophoto generation.

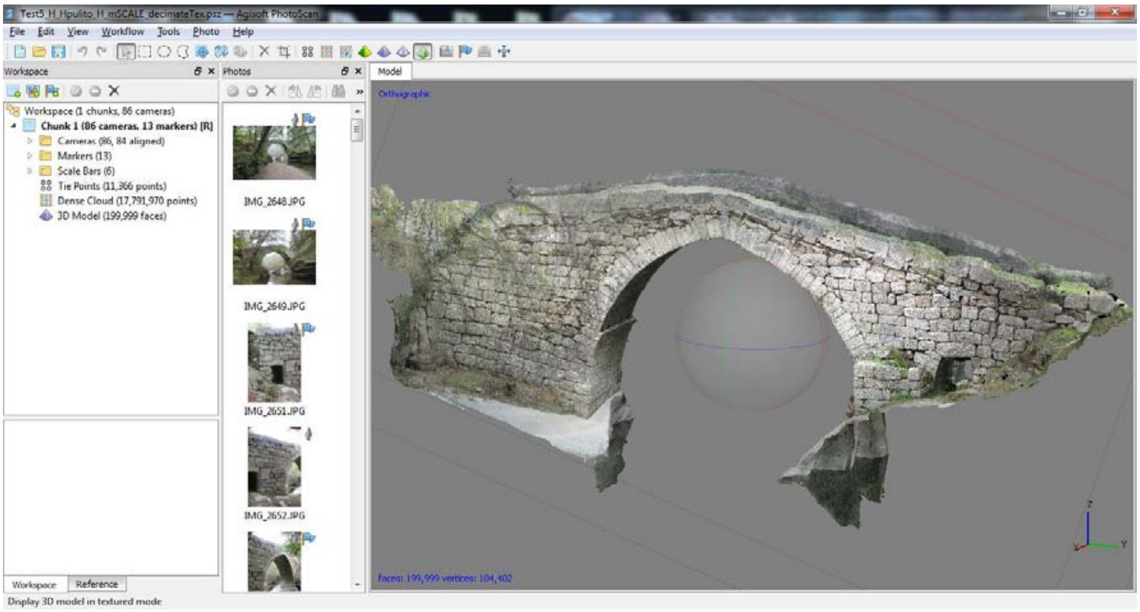


Fig. 3. 23

5- ORIENTING AND SCALING DOWN THE MODEL

Ones got the model it was necessary scale down and orient in the space through markers and scale bars using.

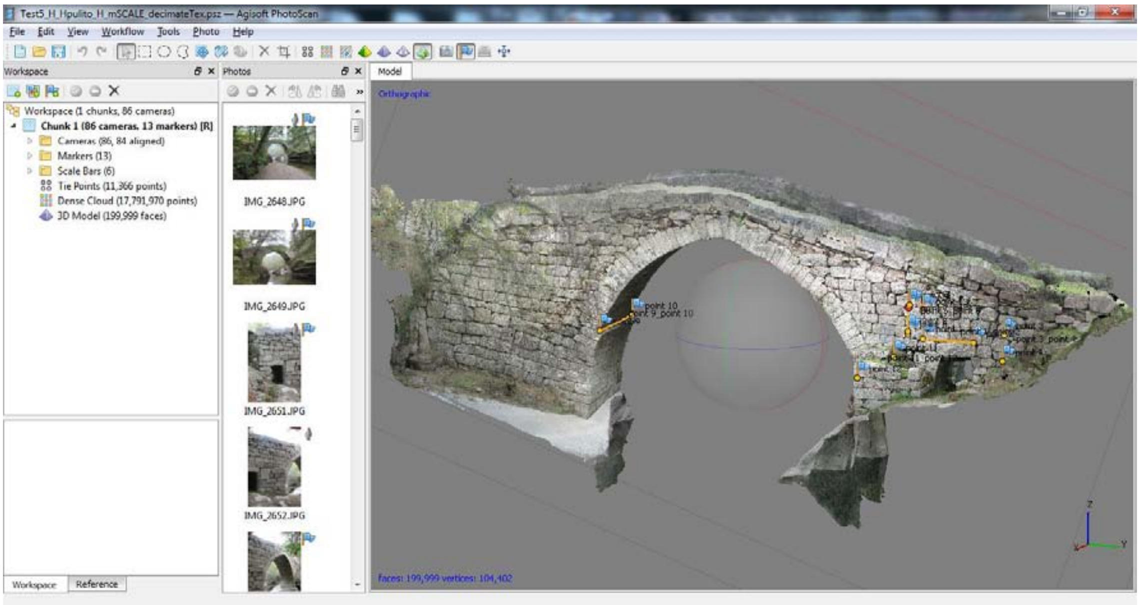


Fig. 3. 24

At the end the 3D final model was gotten (Fig. 3. 25), from which were pulled out the orthophotos lately used to redrawn the bridge (Fig. 3. 26).

Fig. 3. 25 - 3D final model by PhotoScan

a)

b)

c)

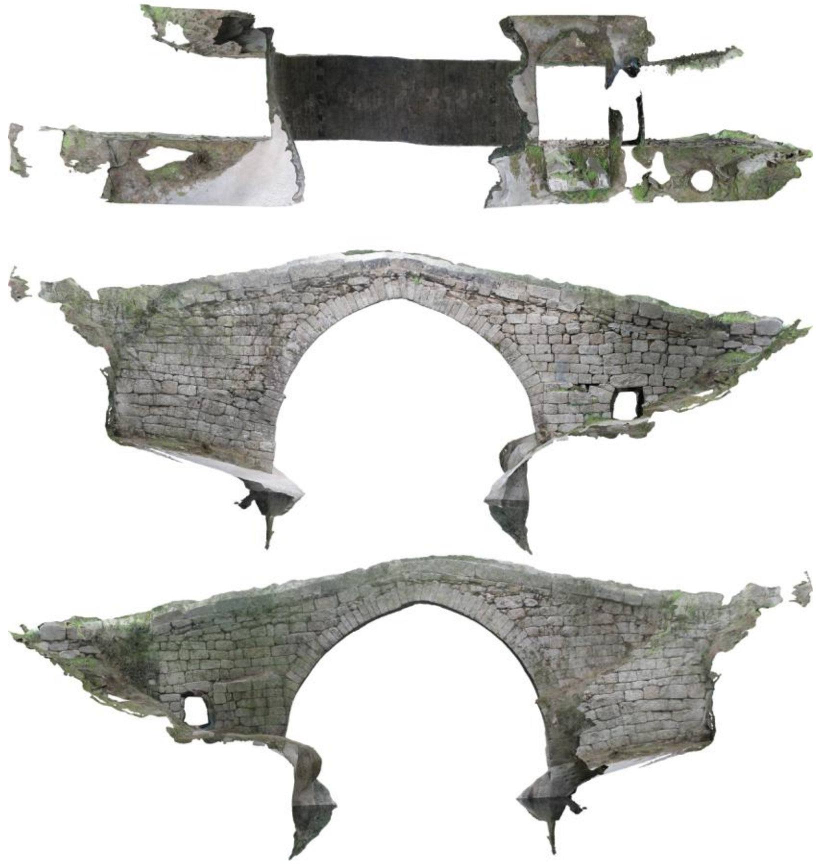


Fig. 3. 26 - Ortophotos by Photoscan: a) bottom, b) downstream and c) upstream view

3.2.2 GROUND PENETRATING RADAR INVESTIGATION

The Ground Penetrating Radar (GPR) is a non-destructive testing (NDT) based on the propagation of electromagnetic radiation, also designated by electromagnetic waves or radiowaves, through the ground or other dielectric media to detect subsurface and underground features. The radar application is based on the fact that the velocity of propagation of the electromagnetic energy and its reflection in the interfaces between different materials are affected by the electric and magnetic properties of these materials (Fernandes, 2006). In fact, if the waves hit irregularities such as separated surfaces, change in the salt content and humidity, hollow cavities, or metal elements inside the element, then the waves are reflected (Proske, van Gelder, 2006). typical modern radar system is generally constituted by the following four components: control unit, radar antenna(s), visualization unit and data storage device (Fig. 3. 27).

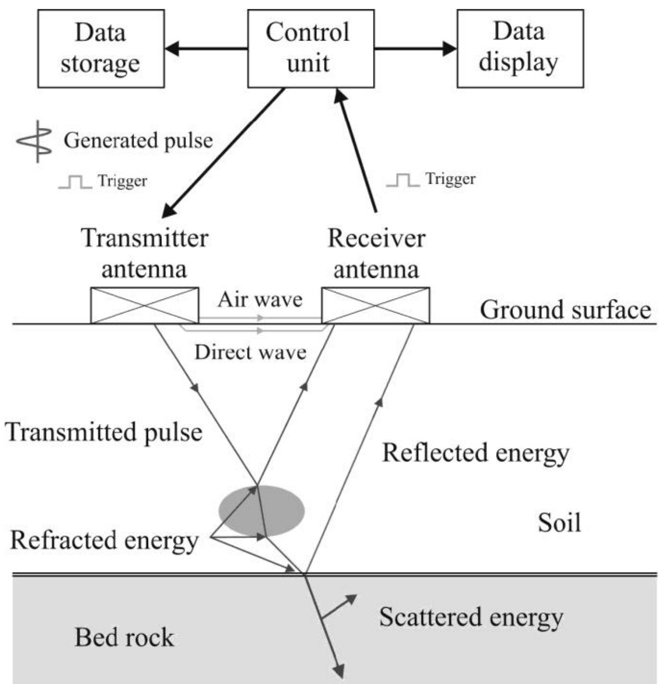


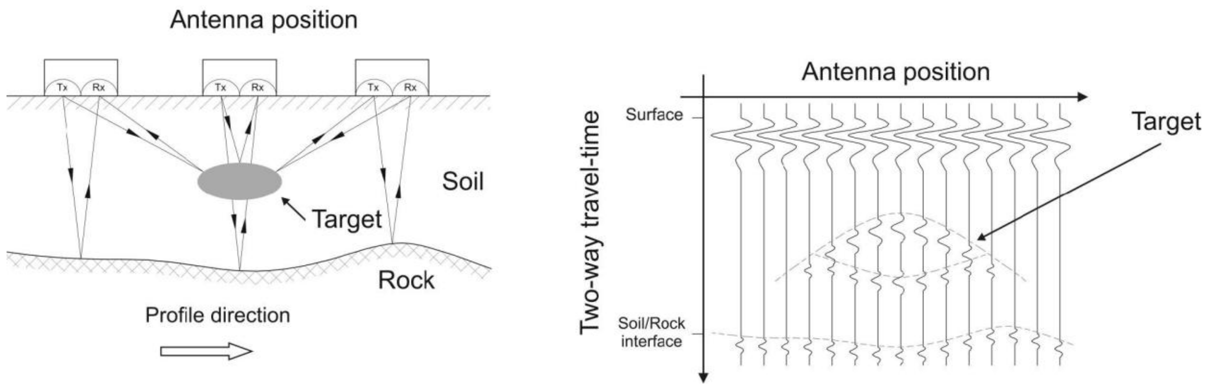
Fig. 3. 27 - Description of the components and operative mode of a modern GPR system (Fernandes, 2006)

An important parameter controlling the depth range of GPR is the transmitting antenna frequency. The antenna frequency employed at a GPR survey should be carefully chosen because there is a balance to be kept between a low frequency antenna, which gives deeper signal penetration but poorer resolution, and a higher frequency antenna, which gives better resolution but shallower penetration. Antennas with a 200–1500 MHz (Fig. 3. 28) centre frequency are best suited for civil engineering applications (Solla et al., 2012).

Central frequency	Depth of penetration	Resolution	Typical applications
10 MHz	50 m	Low resolution	Geotechnical, geological
25 MHz	30 m		
50 MHz	10 m		
100 MHz	5 to 20 m		Geotechnical, environmental, mining
200 MHz	2 to 7 m	Low to medium resolution	Geotechnical, engineering and environmental
500 MHz	1 to 4 m	Medium to high resolution	Engineering
1000 MHz	0.5 to 1.5 m	High resolution	
1500 MHz	0.5 m	Very high resolution	

Fig. 3. 28 - Depth of penetration, resolution and typical applications for usual frequencies (Fernandes, 2006)

The measurements consist of the phases indicated in Fig. 3. 29. Firstly, the control unit generates an electromagnetic pulse and sends it to the transmitter antenna that irradiates the investigation media with a broad beam of electromagnetic energy. That electromagnetic wave is then reflected by each interface between adjacent dielectric materials encountered during its propagation in the investigation medium and the reflected echoes are collected by the receiver antenna. Finally, the data is stored in the memory, where sampling, filtering and reconstruction occur being then displayed on a monitor (Fernandes, 2006).



a) b)

Fig. 3. 29 - Radar reflection survey over a target: a) Methodology and b) resultant radargram displayed as wiggle traces (Fernandes, 2006)

More details on the basic principles of GPR can be found in Annan (2003), Daniels (2004) and Fernandes (2006).

This technique is not so invasive at all and find a large application in the cultural heritage analysis in the masonry bridges too.

Solla et al. (2012), Fernandes (2006), and Diamanti et al. (2008) have been deeply working on the application of geophysical methods towards the internal characterization of masonry arch bridges. Ground Penetrating Radar (GPR) resulted to be very useful for the characterization of backfill materials in terms of homogeneity, structural configuration, and also detection of structural fault and for arch thickness estimation too (Riveiro et al. 2013).

For all these reasons it had decided to use this technique also for “Ponte do Arco” in which the measurements were taken during the second survey in Nov. 25th 2015 (Fig. 3. 30) by Francisco Fernandes engineer and professor at Universidade do Minho who draw up the following report that shows the evaluation from the taken results.

a) b)

Fig. 3. 30 - GPR survey: a) components of GPR, b) measurement’s phases, nov. 25th 2015

The bridge was surveyed with GPR in several locations in order to detect the thickness and shape of the stone arch. Two measurements were carried out in the walls for calibration purposes, and it was found that the average electromagnetic wave speed is around 11 cm/ns (between 10.25 and 11.65 cm/ns). It was found that the stones from the upstream oriented side exhibits a slightly higher speed, which mean that the stones have less moisture than the ones from downstream side.

The analysis of the bridge along its longitudinal axis revealed that the structure is not as a typical masonry arch bridge. Looking at the radargram illustrated in Fig. 3. 31 , it seems that there is in fact no real arch but, instead, all the structure seems to correspond to a structural wall with a round opening and a shallow cover in the bridge deck (the original pavement seems to be at a depth of 14-15 cm from the current bride deck surface). There are quite a few signals that are scattered from inside the bridge and they probably represent areas with a higher proportion of voids, or some kind of material heterogeneity.

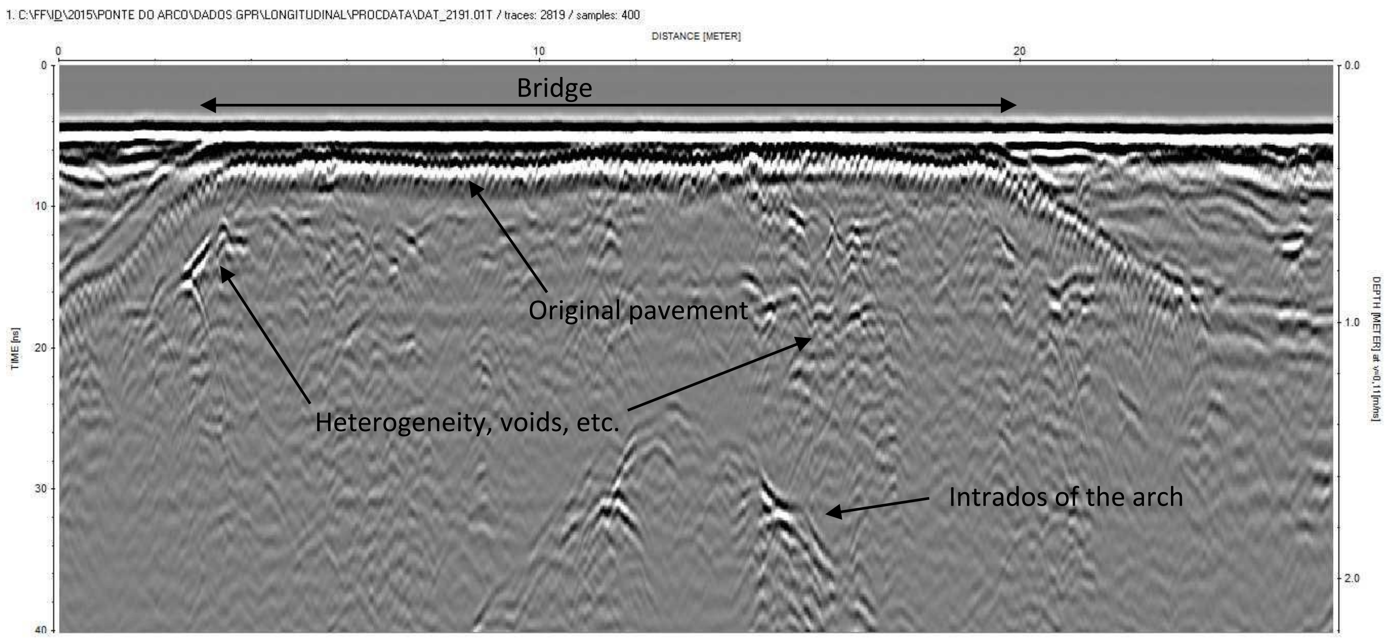


Fig. 3. 31 - Radargram carried out along the longitudinal axis and the center of the bridge (800 MHz)

Regarding the transversal profiles, they clearly confirm the anterior assumptions. The radargrams illustrated in Fig. 3. 32 show the aspect of the cross-section of the bridge from the center (the thinner cross-section) towards the left side (right side show similar results). They show a shallow covering over stone pavement. Additionally, it is possible to discern right below the signals from the stone original pavement a thickness that slightly increases from the center towards the sides and which exhibits a thickness from 35 cm (at the center of the bridge) and go up to 55 cm, near the beginning of the arch.

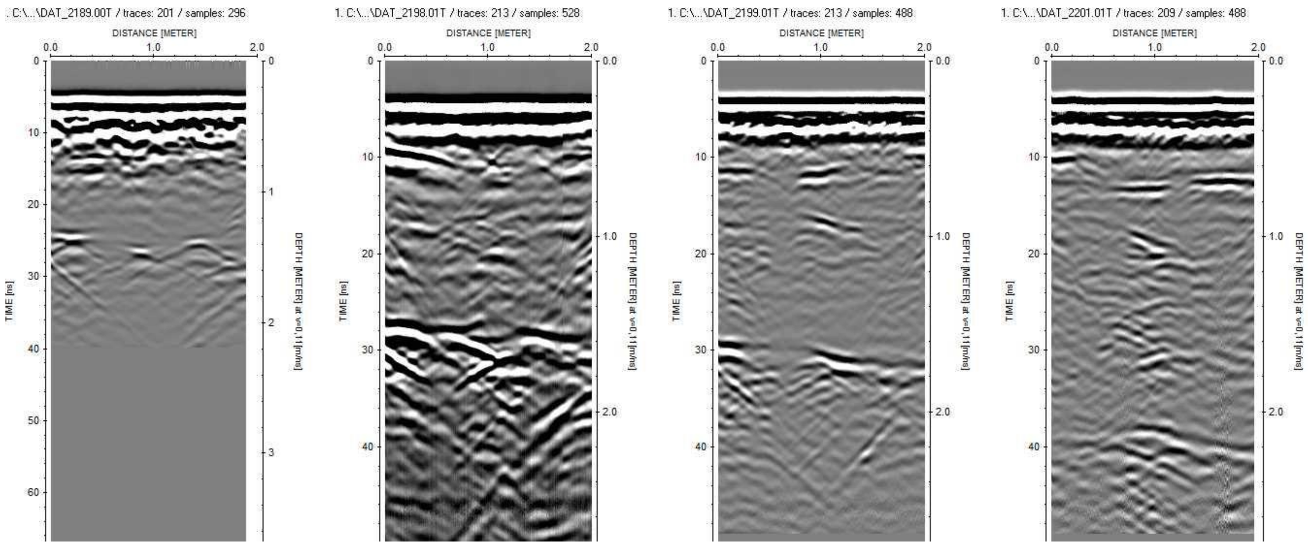


Fig. 3. 32 - Radargrams carried out along the transversal axis and from center of the bridge towards the left side (800 MHz)

3.2.3 GEOMETRICAL DESCRIPTION

Based on the information gotten during the survey and with the test on the field, we are now ready to show and present the geometrical description of the bridge, referring to the drawings made with Autocad in 3D model coming from the photogrammetry process (Fig. 3. 34).

The *"Ponte do Arco"* is a single and slightly pointed arch with a trestle-shaped structure. The bridge is 30 m length on the downstream and 25 m on the upstream and 9 m high from the lower level of springing line to the parapet. The impost basement is not horizontal but follows the altitude profile. Beside the main arch on the left bank we can note two external corps which work as cutwater (triangular on the upstream and trapezoidal on the downstream) and, thus, an rectangular opening that allows the formation of a drainage channel in case of overflow.

The bridge is a barrel vault with a square span arch and a 3.5 m width and a variable arch thickness; it presents 62 voussoirs on the profile.

About the shape of arch, usually it is described as a function of the span "s" and rise "r" or, normally, of the rise to span ratio r/s. In this case is 11 m span and rise depends by where is measurement with a maximum of 7.2 m and a minimun 5.6 m; the average ratio is 0.6 m and, consequently, is possible to classify as deep arch according to Oliveira et al. (2010).

Referring the arch is so difficult to feature it as a traditional pointed arch construction. In fact on the left side is possible to notice a settlement of the structure with a consequent modification of the geometry (Fig. 3. 33).

Fig. 3. 33 - Downstream side photo

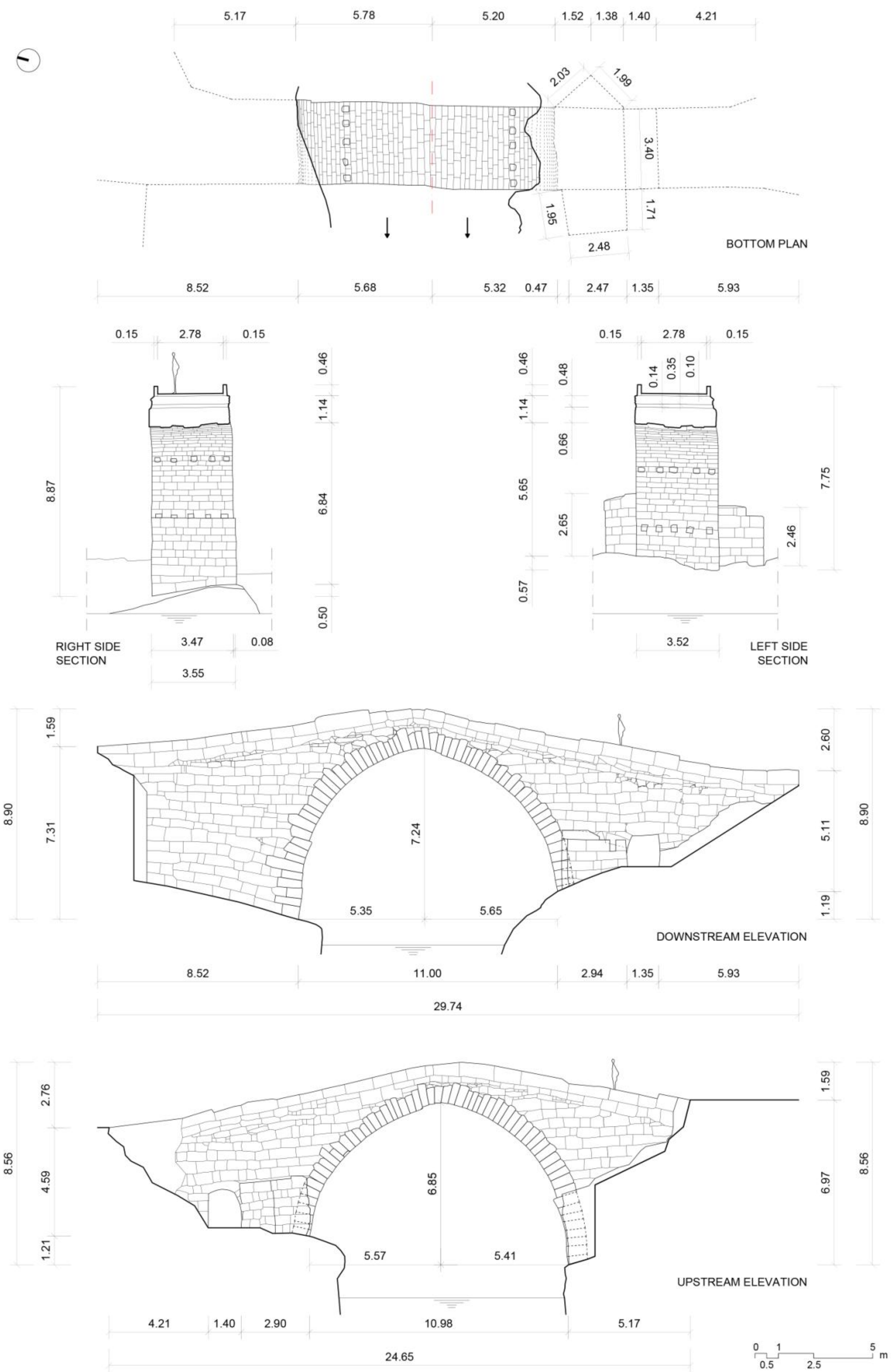


Fig. 3. 34 - Geometrical views

On the right side, otherwise, appears a misalignment of voussoirs which come from construction needs. This situation determines a no continue profile making hard to understand the geometric genesis (Fig. 3. 35). One more aspect that make the situation so complex come from the difference of the springing line which follow the foundation’s trend.

The bridge presents differences between upstream and downstream side: the analysis of this aspect can gives some information of the voussoirs’ size and the possible difference along the width of the vault. Overlaying the two draws we noticed a mismatching that highlight some original geometric flaws and general settlement of the structure (Fig. 3. 36).

About the voussoir’s size the length tend to be smaller approaching to the crown from 0.4 m to 0.2 m; the thickness is from 1.1 m to 0.5 m; width from 1.1 m to 0.2 m. Particularly checking the intradoses and extradoses there are not too many profile difference; so we can assume that the voussoirs’ thickness still be uniform along the barrel.

As we can get from Fig. 3. 35 (b) the bridge presents one more misalignment along the barrel and, specifically, on the crown.

a) b)

Fig. 3. 35 - Misalignments

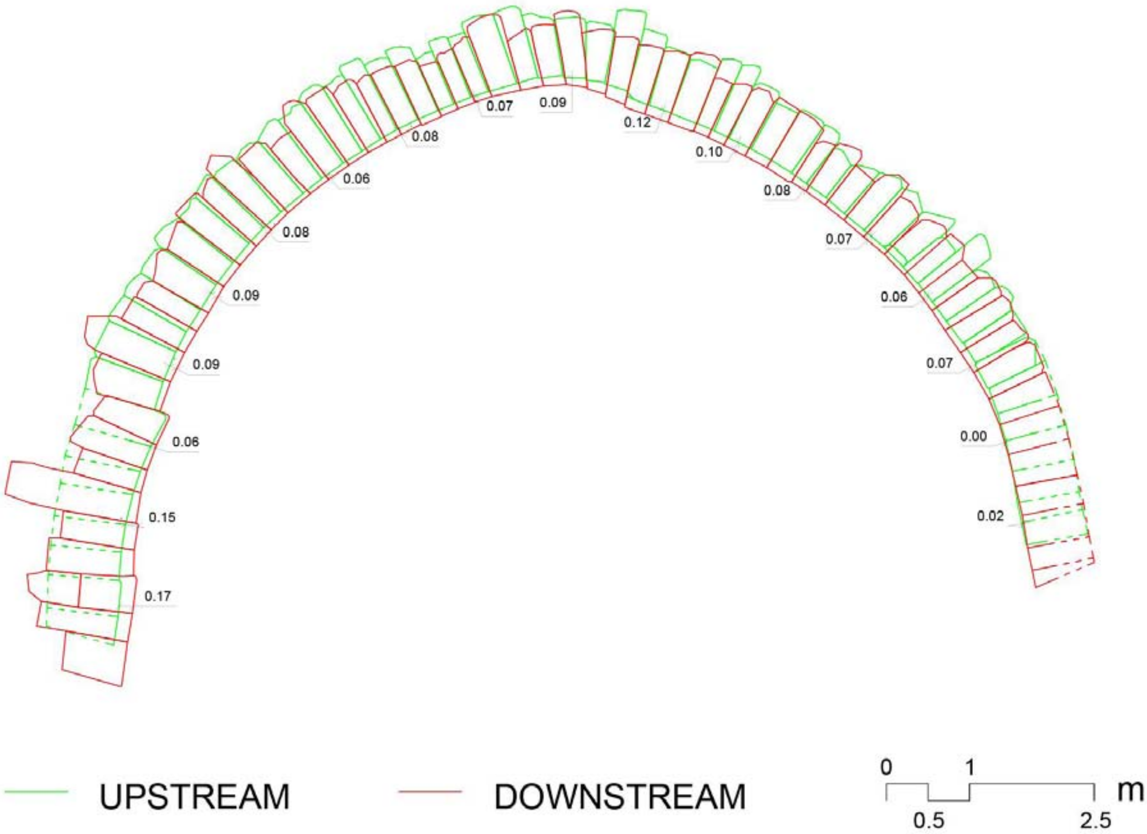


Fig. 3. 36 - Overlay of upstream and downstream profiles

3.2.4 STRUCTURAL DESCRIPTION

“*Ponte do Arco*” presents a structure which is compound for the most part by dry masonry stones of granite without mortar. Granite is the dominant rock in the northern part of Portugal and it belongs to the group of igneous rocks; it is coarse grained granular plutonic rock.

The high mechanical performance (maximum compression strength, modulus of elasticity, etc.) together with the relative ease of obtaining ashlars with a plain morphology, and its durability due to the lack of porosity and its mineral nature, makes granite an ideal stone to be employed in construction (Ozaeta García-Catalán, Martín-Caro Álamo, 2006).

Follows the description of each part of the bridge:

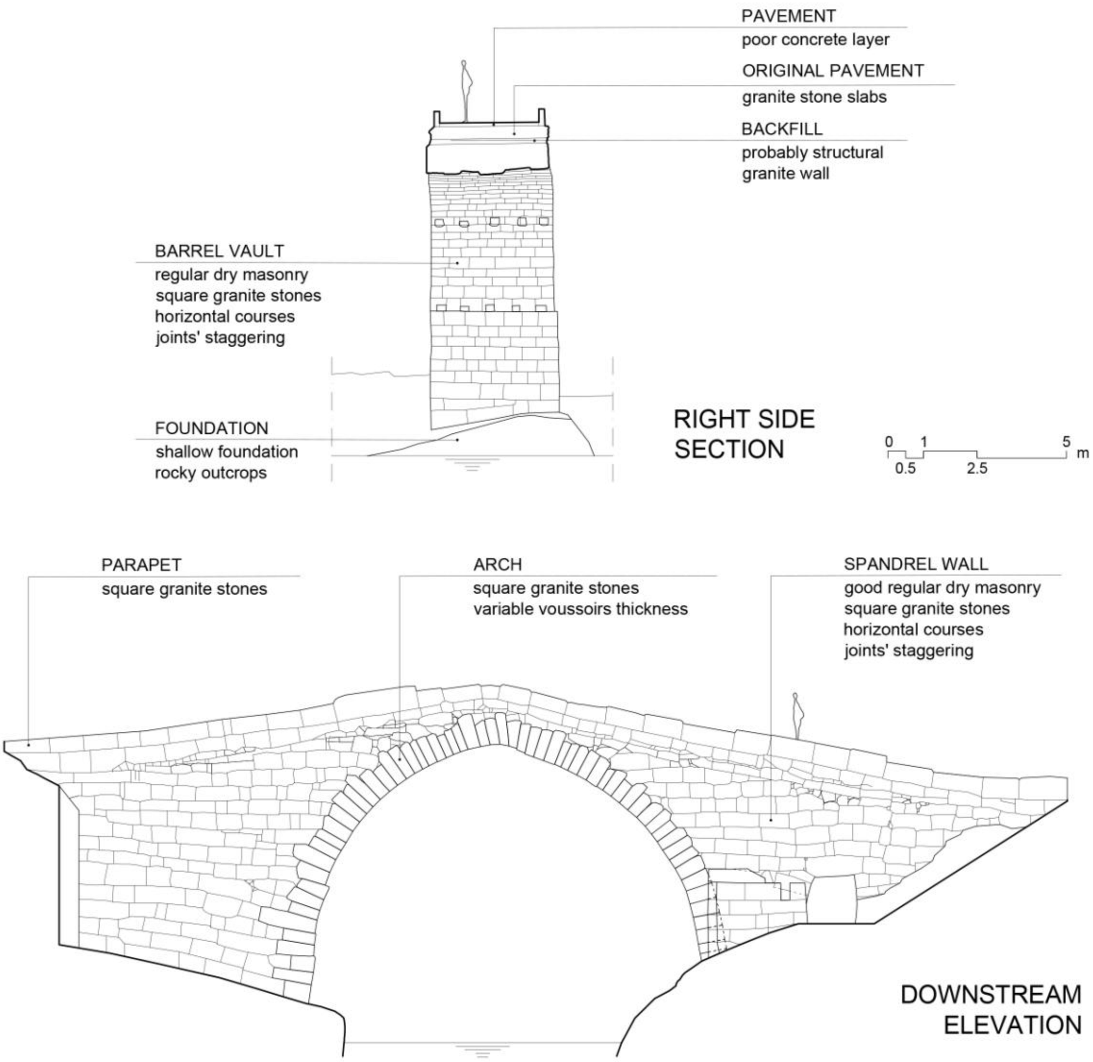


Fig. 3. 37 - Structural description

Analyzing the structure we noticed the presence of some discontinuities such as holes along the barrel to insert the wood centering (Fig. 3. 38); these specific points present masonry’s local weaknesses which can cause relevant consequences on the structure’s behaviour.

a)

b)

Fig. 3. 38 - Discontinuities

3.3 DAMAGE SURVEY

Many studies have analyzed the mechanical behaviour of masonry and its petrological properties that affect the deterioration processes. Thus, there is no classification system that clearly differentiates the nature and origin of the most common damages on these structures which identifies and explains the different processes that occur. On one hand, damages are caused by material degradation and chemical changes in the material, whilst on the other hand, damages affect structural resistance (Ozaeta García-Catalán, Martín-Caro Álamo, 2006). Starting from the survey and photos taken it has been possible to get a map of conservative state of the bridge and the referred damages. Thus, this damages was classified in two main category: damages affecting structural resistance and damages affecting durability, according to Ozaeta García-Catalán, Martín-Caro Álamo (2006) and Rodrigues (2008), and take back into draws.

ISO 13822 states as damage is unfavourable change in the condition of a structure that can affect structural performance.

3.3.1 DAMAGES AFFECTING STRUCTURAL RESISTANCE

These kind of damages are mechanical and come as direct consequences both on the global behaviour of the structure and on the single parts. The phenomena might be caused by the advanced deterioration of the materials and from external forces and seismic assessments too.

Follow the damages found on *“Ponte do Arco”* with a short description with images as example and related drawings.

CRACK

mechanical failure of masonry where the predominant force is compression and the bond is orthogonal to the direction of thrust. The failure is characterized by cracks of the parallel to the direction of compression.

LOSS AND DISLOCATION OF ARCH MATERIAL

This damage can be either due to load actions or to durability, or both. If is due to insufficient load carrying capacity, it is usually a symptom of movements of the supports at the springing of the arch barrel; if loses axial force in the arch barrel, or due to heavy local impact loading near the crown of the arch barrel when the depth of fill over the crown is small.

SLIDING

The formation of a shear mechanism is a usually caused by foundation settlement.

JOINT OPENING

This damage can have several origins. movements of the supports at the springing of the arch barrel; it may also come from the transverse bending and axial tension forces present in the arch barrel; or the presence of water, vegetation, seismic action or uncommon loads activity.

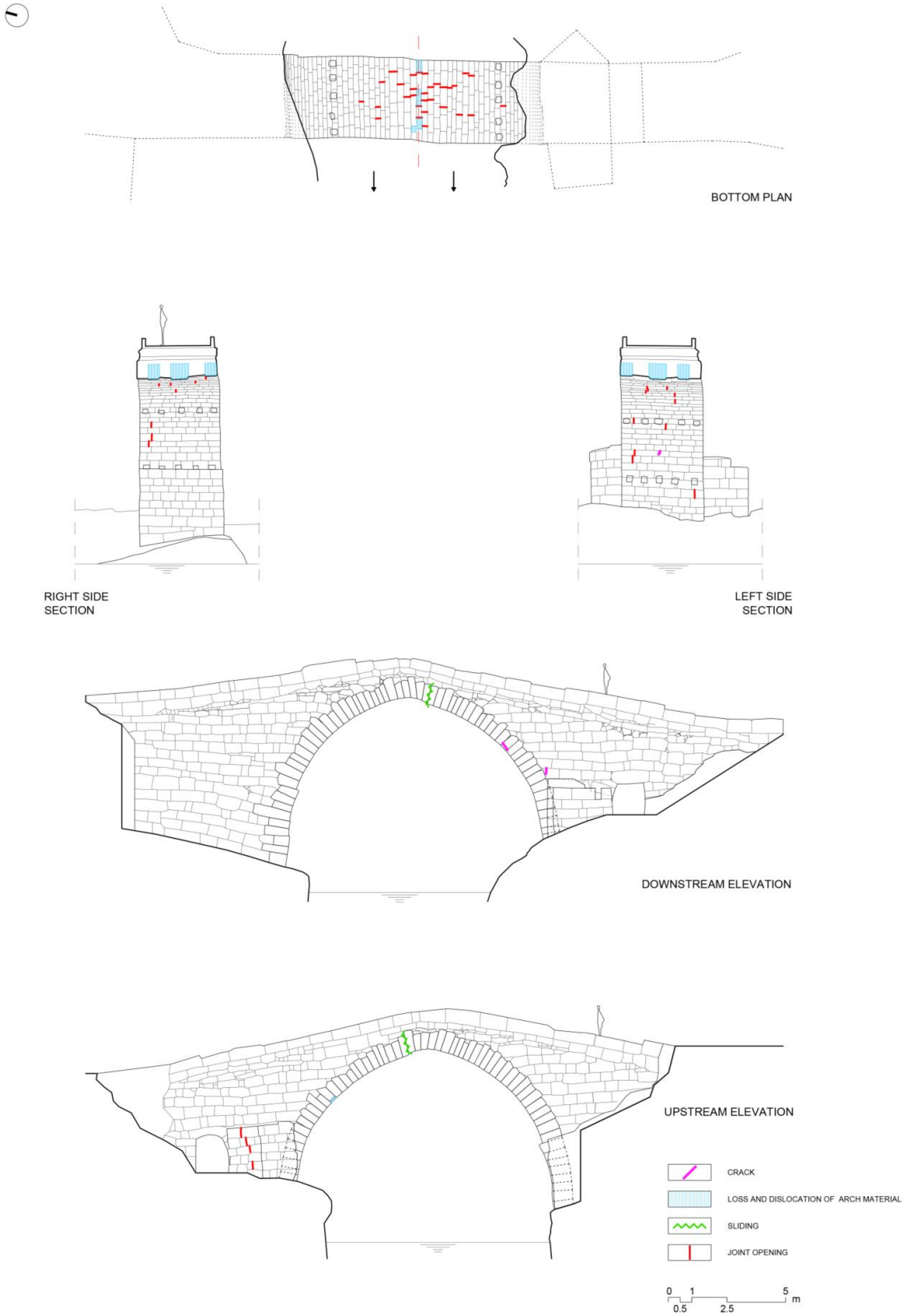


Fig. 3. 39 - Draws of mechanical damages

3.3.2 DAMAGES AFFECTING DURABILITY

There is a wide range and a large number of damages on masonry arch bridges that are caused by the insufficient durability of the materials used in their construction, and thus, these damages are caused by different processes of deterioration and progressive changes to the materials employed in their construction. it is important to point out that masonry structures have an excellent durability because are chemically inert but, if it is not controlled and stabilized, deterioration due to weathering and severe environment changes in material chemicals can be the cause of serious damages or lead to damage that puts the strength capacity of the structure into doubt. Further, deterioration can occur because of wearing due to the continuously use or for maintenance.

BIOLOGICAL COLONIZATION

This classification includes the actions of organisms such as bacteria, fungi, lichen, moss.
Associated damages are generally related to superficial degradation. The life cycle of some bacteria leads to the formation of acids with the development of chemical attacks and black crusts.

VEGETATION

The roots of vegetation exerts pressure on masonry by opening cracks and splitting the stones or bricks. Further, this damage which is associated with mechanical phenomena, can also result in chemical deterioration processes

WET SPOT

The presence of water and humidity are mainly the cause of masonry deterioration. Their effects include the washing out of joint material, hydration of salts, transportation of harmful agents, mineral carbonation of stone, etc. Moreover is the best condition for the grow of biological colonization with the inferred consequences.

GRAFFITI

Graffiti only spoils the appearance of the structure caused by the human’s action.

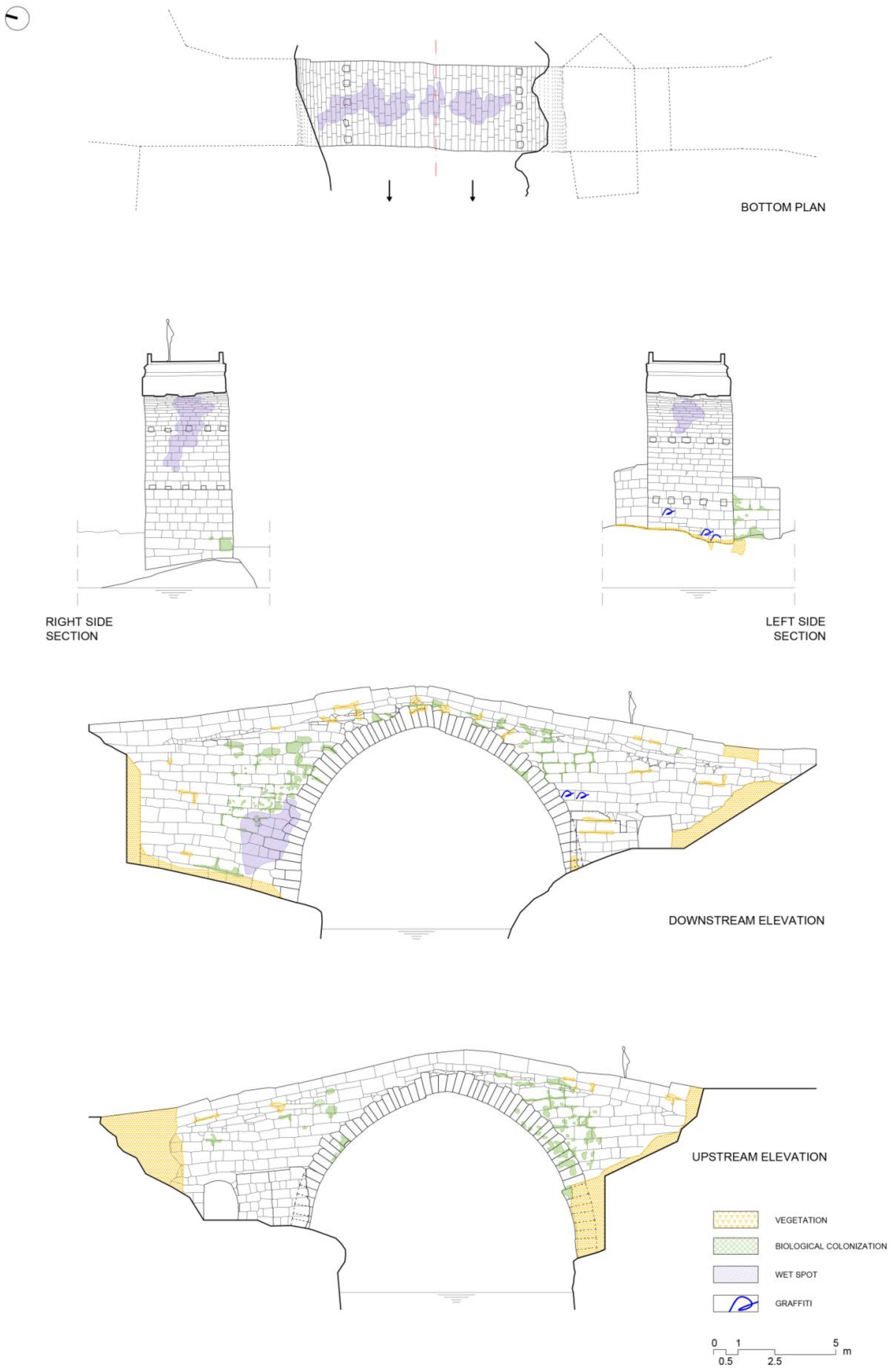


Fig. 3. 40 - Draws of durability damages

3.4 ADOPTED PROPERTIES

Basing on data achieved from the analysis, considering properties’ average values of literature (see Tab. 2. 1) and evaluating the state of preservation (see chap. 3, par. 3), the following properties have been adopted:

Tab. 3. 1 - Mechanical and physiscal adopted properties

	Property	Symbol	Unit	Value adopted
Masonry	Unit weight	γ	kg/m^3	2500
	Friction coefficient	μ	-	0.7
	Friction angle	φ	deg	50
	Cohesion	c	MPa	0.5
	Young’s modulus	E	MPa	5000
	Poisson ratio	ν	-	0.2
	Compressive strength	f_c	MPa	8
Backfill	Unit weight	γ	kg/m^3	2200
	Friction angle	φ	deg	40
	Cohesion	c	MPa	0.05
	Young’s modulus	E	MPa	1000
	Poisson ratio	ν	-	0.2

The values have adopted with the numerical assessment (that will be explained soon) based on the needs of the properties’ model.

4. NUMERICAL LOAD-CARRYING CAPACITY ASSESSMENT

To evaluate the behaviour and the load bearing capacity of the bring under vertical forces we have decided to use the kinematic approach of limit analysis of equilibrium. As explained above (see chap. 2, par. 2.3) this analysis is based on the theory developed by Heyman on the general behaviour of masonry and, specifically, masonry arch’s behaviour too where not too much data as necessary to describe the structural response. The arch develops hinges when the line of thrusts is tangent to the edge of the profile; with the developments of mechanism the structure collapses.

The “*Ponte do Arco*” analysis, conducted with this method, has been done with the *LimiteState Ring 3.1* software (see chap.2, par. 2.1). The software got a development by Sheffield University during the 1990s and many authors use it such as Oliveira et al. (2010) Riveiro et al. (2010), Solla et al. (2012), Sarhosis et al. (2014).

Ones defined the geometrical model in which enter the materials properties, we have proceeded with the study of the load-carrying capacity of the bridge when the load position changes and when main parameters that modify the structural response changes too; finally with the safety verification according to the Italian Code (NTC 2008).

4.1 GEOMETRIC MODEL, MATERIALS AND LOADS

The structural response of masonry arch bridge essentially depends on arch and fill properties.
The arch is the structural element responsible for transferring the load to foundations, while the soil adds dead weight, disperses the applied load at the surface and provides a horizontal restraint to movements of the arch.
The numerical structural model which forecasts and evaluates the work behaviour, has to consider the interaction between arch and fill in the response and has to guarantee the possibility to define the geometry of the arch and materials properties too.
Due of these reasons, the Ring 3.1 software has been employed for those evaluations.
Based on the draws coming from survey phase, it has been possible to determine the geometry of the model considering: an effective bridge width of 3.50 m; a trestle-shaped fill profile with a surface fill depth of 0.35 m; foundation quotes are different; a number of 100 units of voussoirs (this number is the resulted dividing the arch length with the minimum width of each one.
Referring to the arch, because of it geometrical complexity, it has decided to design three models which present a common intrados profile coming from the interpolation of different points, and a different extrados profile. Those models are showed as follows:

MODEL 1: constant thickness of arch established 0.50 m as minimum length.

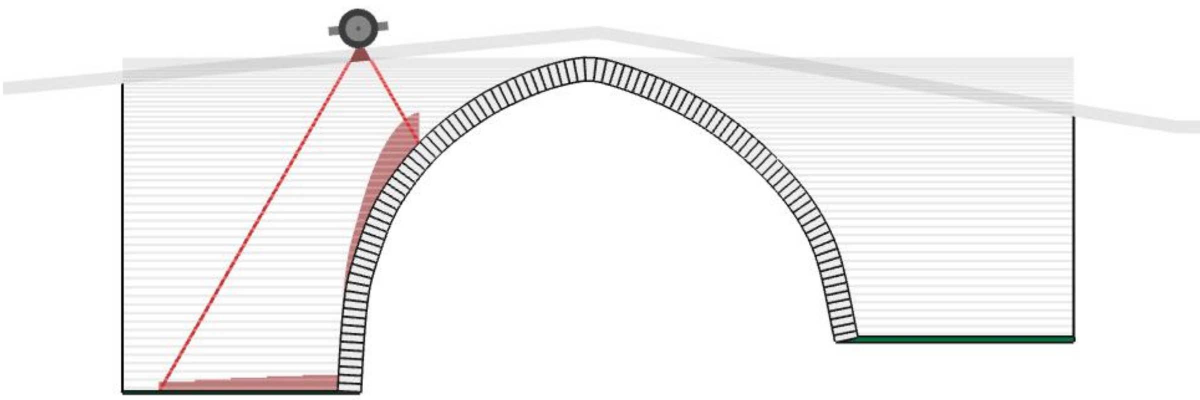


Fig. 4. 1

MODEL 2: variable thickness of arch, coming from the interpolation of different points with not uniform extrados profile.

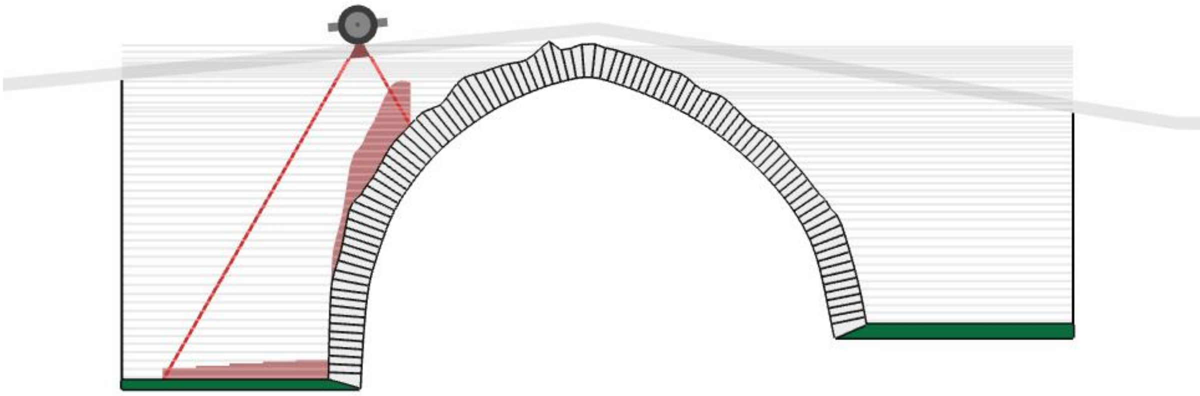


Fig. 4. 2

MODEL 3: variable thickness of arch, where joint depth’s is found with the mortar loss tool present in the Ring 3.1; this edit allows to reduce the contact length both to the extrados an intrados starting from a constant arch thickness (in this case 0.85 m).

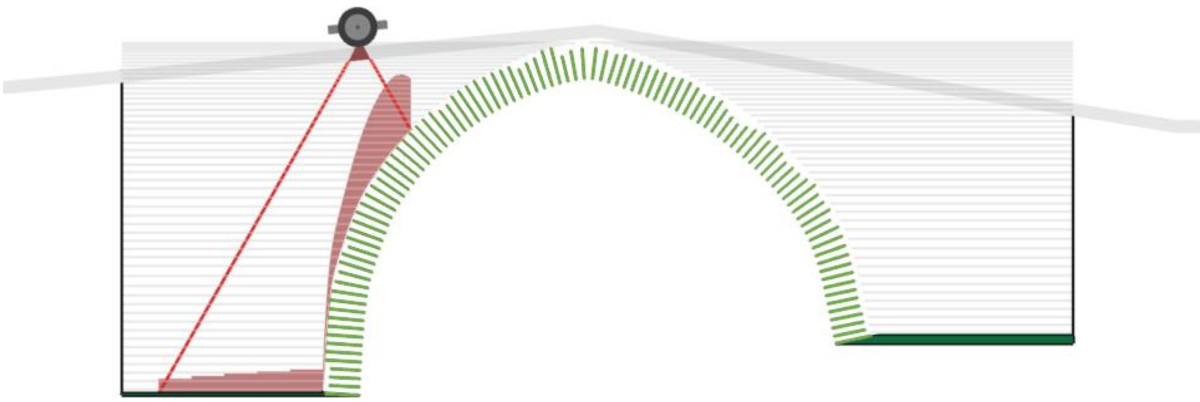


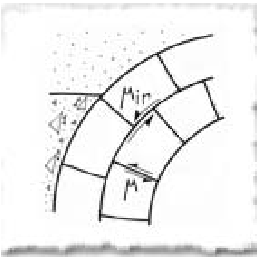
Fig. 4. 3

Referring to material properties, have been used the data in Tab. 3. 1. resumed in Tab. 4. 1 as follows:

Tab. 4. 1 - Ring adopted material properties

	Property	Symbol	Unit	Value adopted
Masonry	Unit weight	γ	kg/m^3	2500
	Friction coefficient	μ	-	0.7
	Compressive strength	f_c	MPa	8
Fill material	Unit weight	γ	kg/m^3	2200
	Friction angle	φ	deg	40
	Cohesion	c	MPa	0.05

Il Friction coefficient μ (Fig. 4. 4 a) is 0.7 considerig the voussoirs surface roughness and keeping the average value between what Lourenço et al. say (2004) (Fig. 4. 4 b).



Series	$\tan\phi_i$	$\tan\phi_f$	$\tan\phi_i / \tan\phi_f$
P (Polished)	0.18	0.43	2.4
S (Sawn)	0.62	0.63	1.0
R (Rough)	0.56	0.74	1.3

a) b)

Fig. 4. 4 - Friction coefficient: a) Scheme (LimitState, 2014), b) Reference values (Lourenço et al., 2004)

Moreover the load dispersion through the fill was modelled according to the classical Boussinesq distribution with a dispersion angle of 30°, while an earth pressure coefficient *kp*, based on the Rankine theory and equal to half of the value adopted for arches, was also used by Oliveira et al. (2010), Smith et al. (2004).

The used load is defaulted at 1 kN Single Axle by the program, with width 1.80 m and loaded length 0.30 m. In the present study, non-linear geometric effects were not considered of relevance as no shallow arch have been studied.

4.2 LOAD CAPACITY

The ultimate load-carrying capacity is expressed in terms of a load factor, which is the ratio between the collapse load and the live load, comprised of a standard vehicle. Obviously, a different load factor is associated with each possible location of the moving vehicle.
The software allows to move the position of the load along the bridge calculating the load factor and the collapse mechanism associated for each load cases.
Adopting live load of 1 kN, the multiplier, gotten by Ring 3.1, represents the collapse load.
La Fig. 4. 5 represents the relation between the collapse load with position along the arch; the graph shows the collapse load changes according the applying position.

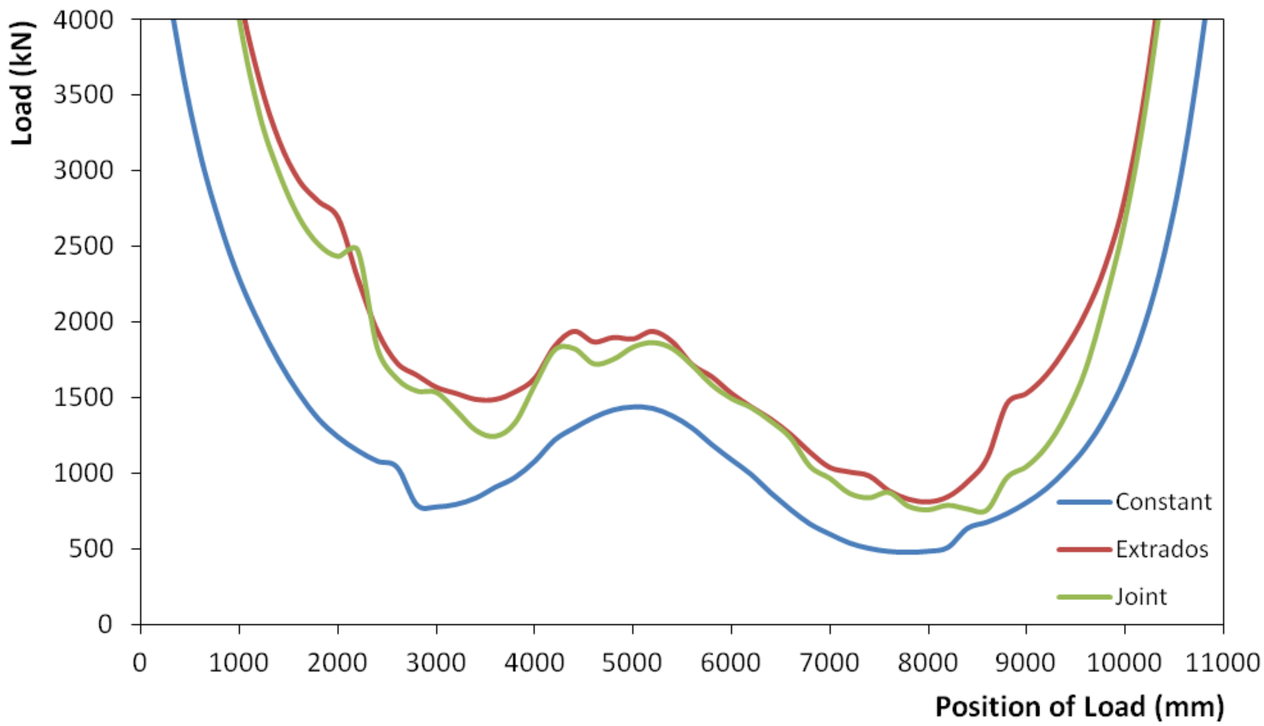


Fig. 4. 5 - Collapse load changes graph

According to the graph, profiles demonstrate that trends are similar with lower multiplier when the load is at ¼ of arch span. The ultimate load-carrying capacity is 482 kN form Model 1 (constant thickness), 812 kN for Model 2 (variable extrados profile), 763 kN for Model 3 (variable joint depth). Model 1 ensures a higher level of safety, in Model 2 lever ensured is lower and, in the

Model 3 presents the most reliable behaviour because of the geometrical characteristics that are closer to real arch. Due of these reasons, it has decided to use this one for the following analysis.

The Fig. 4. 6 shows the rotational mechanism that takes place when the load is charged in the most unfavorable position.

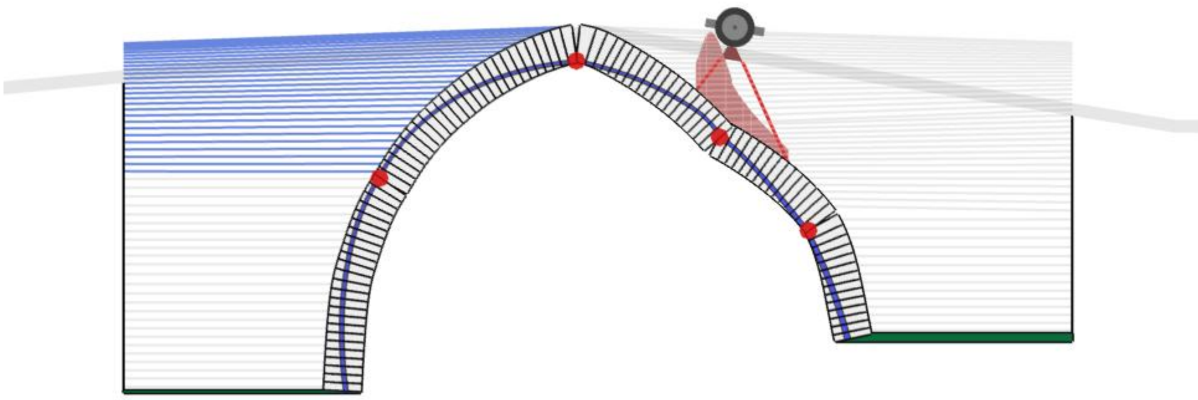


Fig. 4. 6 - Rotational mechanism of failure with hinges

4.3 PARAMETRIC ANALYSIS

As above mentioned, the structural response of masonry arch bridge essentially depends on arch and fill properties.

In order to obtain a deep insight into the most important parameters controlling their load-carrying capacity, a parametric analysis was performed on the bridge. The relevant variables considered here are geometrical and mechanical parameters of the arch, mechanical and physical parameters of the fill:

- fill properties (γ);
- compressive strength (*f*c) of masonry arch;
- joint depth (j) of masonry arch.

The physical properties of the soil placed above the arch, encompass its self-weight and internal friction angle. The variation in these properties directly implies the simultaneous and coherent variation of both parameters, as well as indirectly the variation of the earth pressure coefficient and fill-barrel friction angle (Oliveira et al., 2010).

From bold values that represent which were adopted for the numerical analysis of “*Ponte do Arco*”, we have moved to results that grow or decrease the parameter’s characteristics.

The values adopted for parametric analysis are provided in Tab. 4. 2

Tab. 4. 2 - Values adopted for parametric analysis (reference values marked in bold).

	Parameter	**Unit**	**Parametric variation**					
Fill	**Fill properties (*γ*)**	**(°; kN/m^3)**	(20; 18)	(25; 19)	(30; 20)	(35; 21)	**(40; 22)**	(45, 23)
Arch	**Compressive strength (f_c)**	**(MPa)**			4	6	**8**	12	16
	Joint depth (j)	**(α)**			0.80	0.90	**1.00**	1.10	

Fill properties (γ) (°; kN/m^3)	Load factor (Fc) (kN)
(20; 18)	486
(25; 19)	560
(30; 20)	633
(35; 21)	690
(40; 22)	**763**
(45; 23)	814

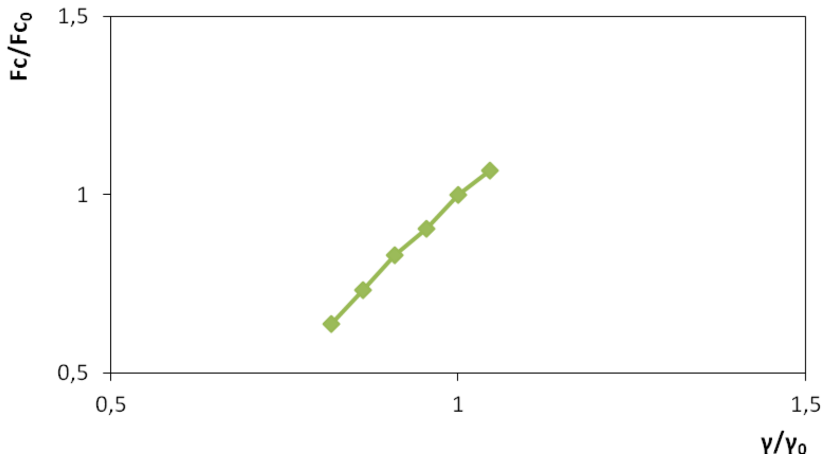


Fig. 4. 7 - Parametric analysis: fill properties

Compressive strength (f_c) (MPa)	Load factor (Fc) (kN)
4	640
6	720
8	**763**
12	797
16	814

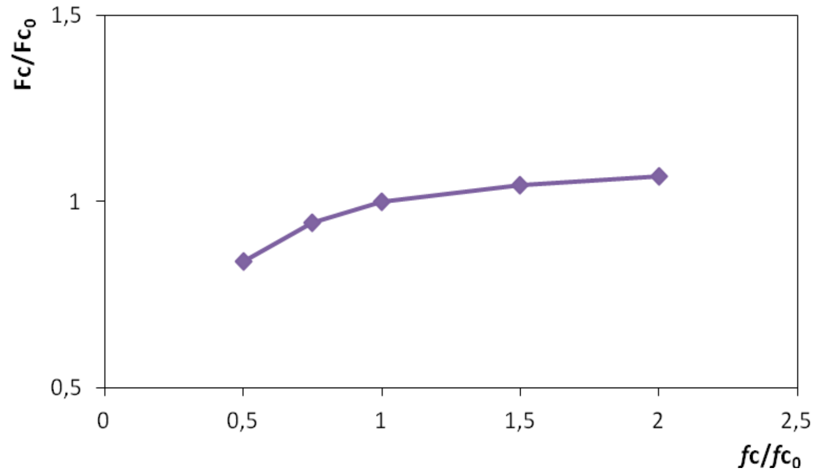


Fig. 4. 8 - Parametric analysis: compressive strength

Joint depth (j) (α)	Load factor (Fc) (kN)
0.80	640
0.90	720
1.00	**763**
1.10	797

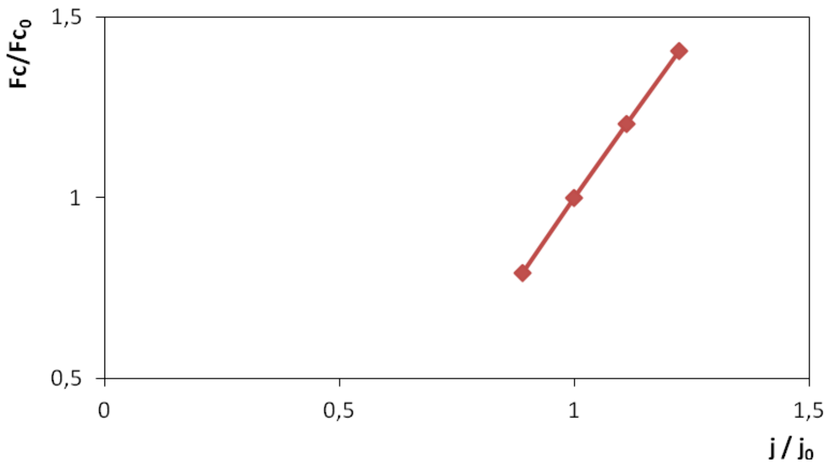


Fig. 4. 9 - Parametric analysis: joint depth

The results from the parametric analyses are summarized in Fig. 4. 10, where load factor (denoted as FC) is correlated with referred parameters.

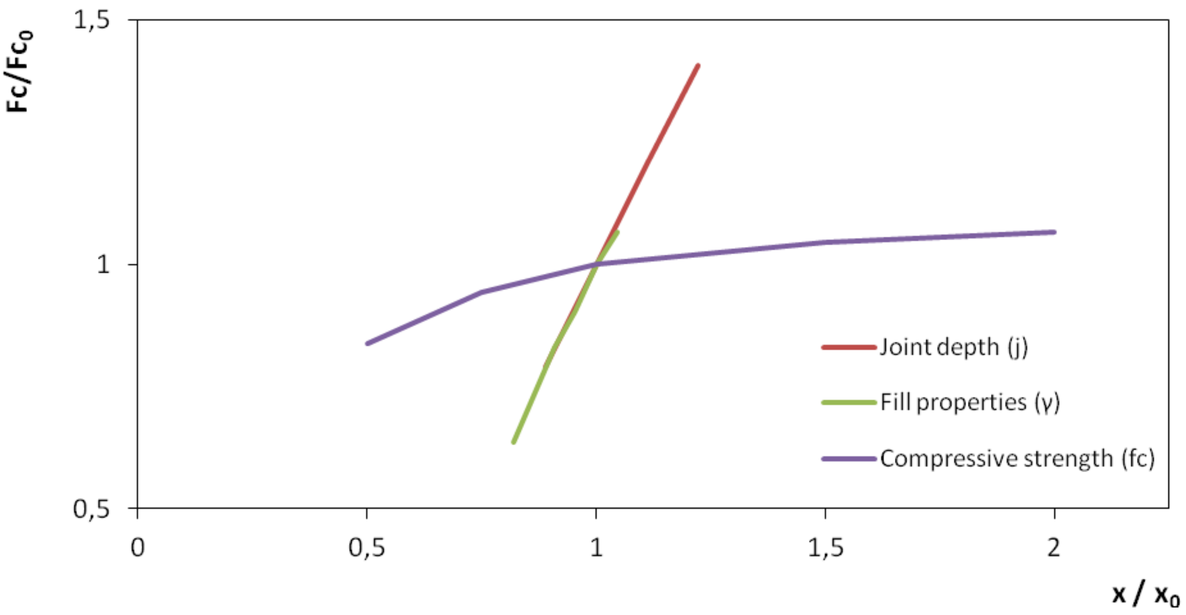


Fig. 4. 10 - Comparison between investigated variables

As we can get by the curves’ slope, Joint depth, which represents arch thickness, and Fill properties mark crucially on the load bearing capacity. The main consequence is that parameter values have to be well thought and based on a deep knowledge, before using those for. In fact, from this analysis it might be chosen which kind of test and investigation for the bridge behaviour main data are relevant.

4.4 SAFETY VERIFICATION

The topic inherent the "*Ponte do Arco*" safety, it has decided to use the Italian law as governing due of the deep analysis in this work and according to the specific application to the existing buildings.

Particularly, the Circolare n. 617 (2009) established in the paragraph dedicated to the safety (C8.3), that the chance to find the action value that the structure can support with the safety limit imposed by the NTC (2008), using the partial factors of safety.

Based on those providing it has decided to determine the traffic action, with the geometric characteristics forecasted by NTC (chap. 5, par. 1.3.3) (Fig. 4. 11) that the bridge can support in safety conditions.

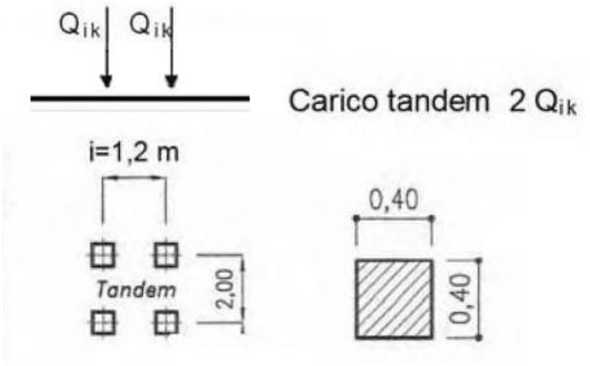


Fig. 4. 11 - Characteristic of traffic load according to NTC 2008

The Italian code points out the hypothesis of test safety evaluation of existing masonry buildings minding only to USL (Ultimate Limit States), analyzing failure's kinematisms and providing the confidence factors use. The confidence factor reduces the average value of resistance of the existing construction materials and it depends from the "knowledge level" of the structure (geometry, details, materials) (C8.2).

The knowledge level gotten in the present evaluation of "*Ponte do* Arco" is limited (LC1) because neither tests and samples referred to material properties had been taken.

To find out the characteristic value of variable actions Q_{iK} (Fig. 4. 11), has considered this equation:

$$R_d = E_d$$

R_d , design value of the resistance.

$$R_d = \frac{R_k}{\gamma_m}$$

R_k , characteristic value of the resistance.

γ_m , partial factor for a material property.

$$\text{con } R_k = \frac{R_{average}}{FC}$$

$R_{average}$, average value of the resistance.

FC , confidence factor.

E_d ,design value of effect of actions: which can be evacuate based on design values of actions (F_{dj})

$$F_{dj} = F_{kj} \times \gamma_{Fj}$$

F_{kj} , characteristic value of an action j.

γ_{Fj} , partial factor for action j.

Ring 3.1 software allows to apply partial safety factors both on material properties and actions.

For a material property of masonry we provided the following values:

- $\gamma_m = 3$, for historical masonry (Tab. 4.5.II. of NTC);
- FC = 1.35, for a Limited knowledge level (LC1 of Tab. C8A.1.1 Circolare)

For actions, adopted values come from Tab. 5.1.V of NTC that provides Fundamental Combinations:

- $\gamma_G = 1$, favourable value for permanent actions (self-weigh of structure and backfill); because dead load are distributed, they increase the load carrying capacity;
- $\gamma_Q = 1.35$, unfavourable value for traffic variable action; because bridges are weaker with concentrated action.

In the present evaluation other variable actions such as snow and wind action are not considered (chap 3 par. 7.5 CNR-DT 213/2015); otherwise the dynamic load traffic factor is not considered too because the roadway plan does not allow an high run speed.

The characteristic value of traffic variable action resulted by the analysis and which come out from a mixed collapse mechanism, is equal to Q_{iK} = 62.7 kN with design value Q_d = 84.6 kN.

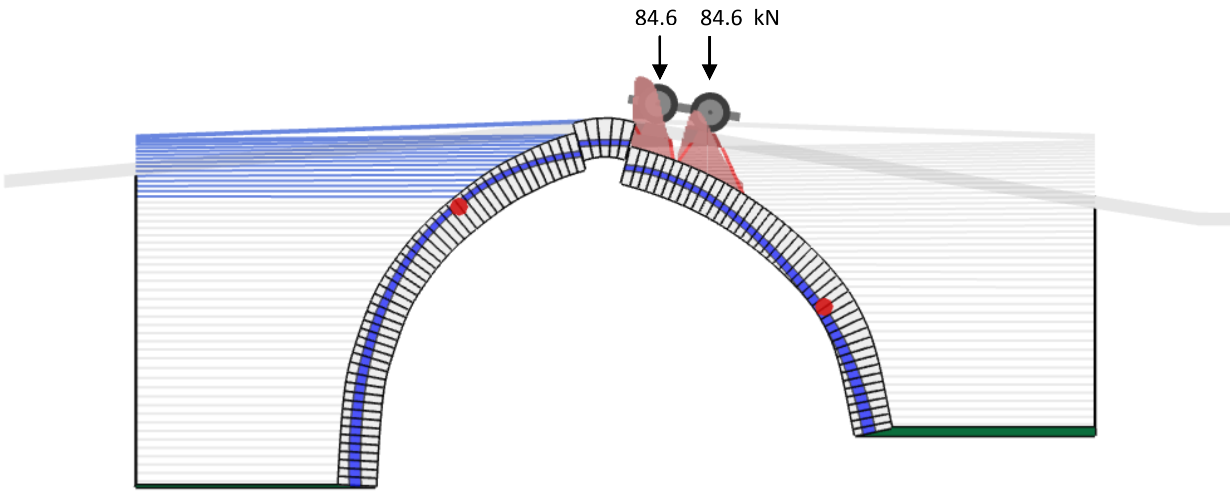


Fig. 4. 12 - Collapse mechanism of safety verification

If the Friction coefficient is left at 0.7, without factor γ_m e FC decreasing as show by Gilbert engineer in the annexes to Ring 3.1 manual, the Q_{iK} is 223 kN (Q_d = 301 kN) with a rotational mechanism (Fig. 4. 13). This has been considered to highlight how changes the bridge response because tiny modifications cause substantial alteration on the structure behaviour.

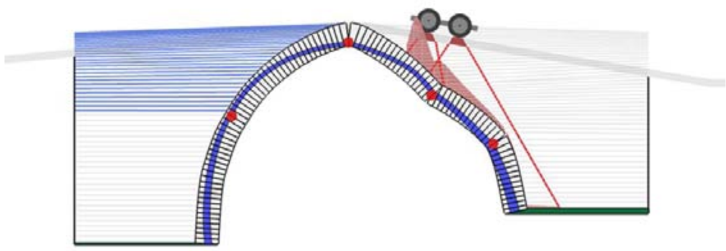


Fig. 4. 13 - idem

5. NUMERICAL DYNAMIC ASSESSMENT

The following chapter provides the complete evaluation of the behaviour compared both to dynamic and seismic action. To accomplish this analysis we have chosen the three-dimensional model with the finite-element method using the macro-modelling strategy made by Pelà et al. (2009). This operation has been conducted with the commercial software Straus7 use; this software is typically used by designers and professionals for structural analysis.

Ones designed the model with the referred properties, we defined the seismic action specifying elastic response spectrum established by *Norma Portuguesa* and, finally, the identification of the dynamic properties of the bridge. Thank to these elements we made a modal response spectrum analysis to have a face indication of bridge’s response and, after that, a simplified pushover analysis to study the behaviour evolution when the seismic action grows up. Finally, thank to the model’s exploitation, we made an incremental analysis, increasing self weight values to find a vertical safety factor.

5.1 GEOMETRIC MODEL, MATERIALS AND LOADS

Starting from draws taken during surveys, we defined a pattern-façade that had to be imported on Straus7 and divided through bridge's part: vault, spandrel wall and backfill.

From this phase we proceed with the parts triangular meshes creation, with 6 nodes plates elements and with the extrusion of those to make 16 nodes bricks.

Had followed the edge's condition with fixed restraints at the basement and lateral support on the upstream faces which have the task to reply the soil opposition present on that side (coefficient of subgrade reaction of linear elastic springs 160 MPa/m, referred to Winkler model on soil behaviour as highlighted by Terzaghi (1955) for dense sands).

Properties valued adopted are provided in Tab. 5. 1; as said the strategy adopted is the macro-modelling. An elastic-plastic constitutive law has been applied to materials, considering the post-peak behaviour with Mohr-Coulomb failure criterion (see chap. 2 par. 2.1).

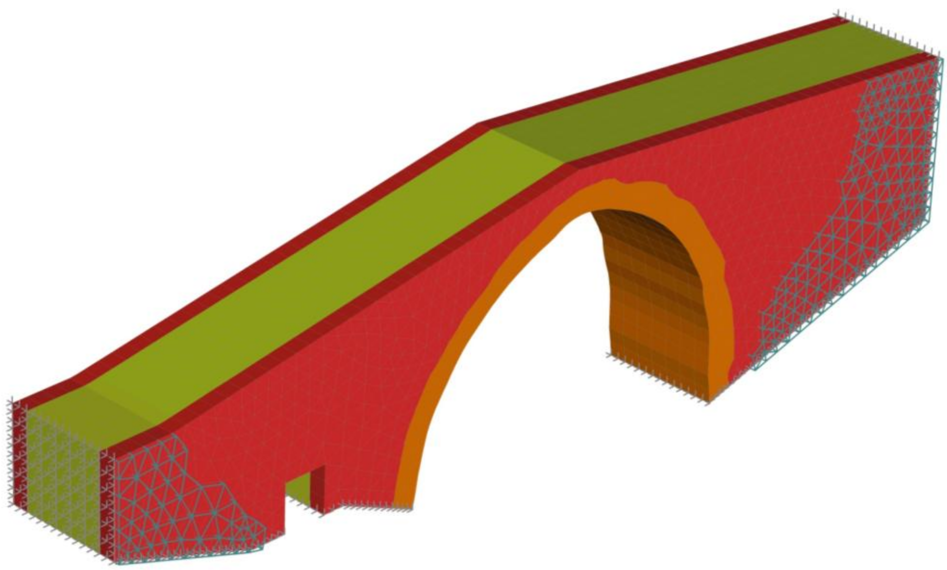


Fig. 5. 1 FEM Model: upstream side 3D view.

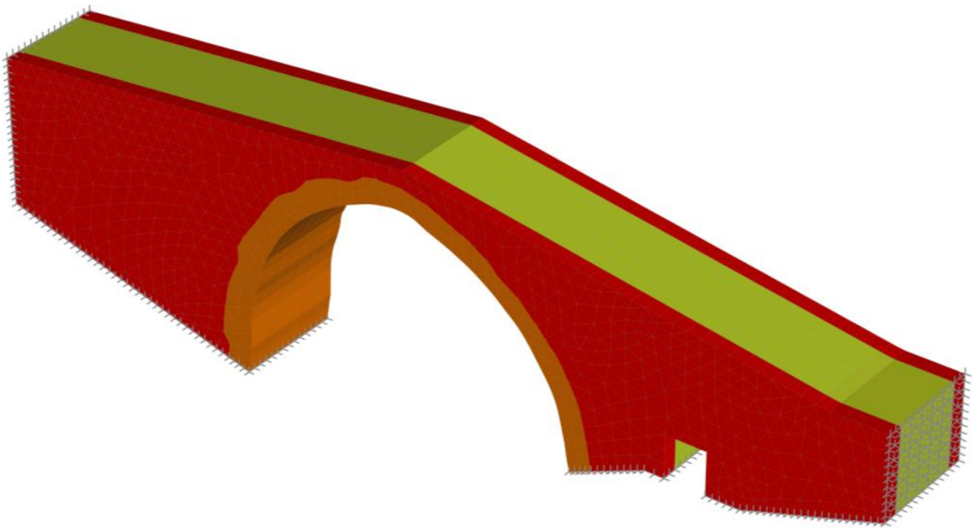


Fig. 5. 2 - FEM Model: downstream side 3D view.

Tab. 5. 1 - FEM adopted material properties

	Property	Symbol	Unit	Value adopted
Vault	Unit weight	γ	kg/m^3	2500
	Friction angle	φ	deg	50
	Cohesion	c	MPa	0.5
	Young's modulus	E	MPa	5000
	Poisson ratio	ν	-	0.2
Spandrel wall	Unit weight	γ	kg/m^3	2400
	Friction angle	φ	deg	50
	Cohesion	c	MPa	0.3
	Young's modulus	E	MPa	3000
	Poisson ratio	ν	-	0.2
Backfill	Unit weight	γ	kg/m^3	2200
	Friction angle	φ	deg	40
	Cohesion	c	MPa	0.05
	Young's modulus	E	MPa	1000
	Poisson ratio	ν	-	0.2

At this point we proceeded with the determination of seismic action that have to been considered for the following analysis. The *Norma Portuguesa* (2009) is the Portuguese version of Eurocode 8 that divides the national territory through different zones with a different seismic hazard; moreover, it provides the Type 1 (far-field) and the Type 2(near-field) as seismic action. In Tab. 5. 4 are shoed the reference peak ground accelerations zones:

"Ponte do Arco", localized by Marco de Canaveses, is::
- in zone 1.6 for the Type 1 seismic action with PGA =0.35 (m/s^2);
- in zone 2.5 for the Type 2 seismic action with PGA=0.80 (m/s^2);

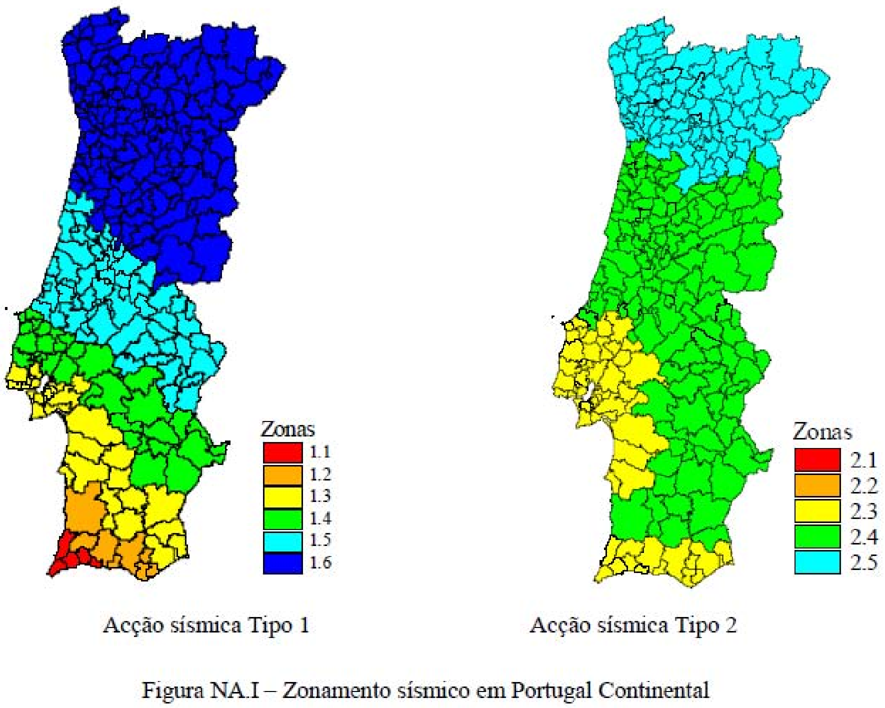


Fig. 5. 3 - Seismic hazard map of Portugal (*Norma Portuguesa*, 2009).

Quadro NA.I – Aceleração máxima de referência a_{gR} (m/s^2) nas várias zonas sísmicas

Acção sísmica Tipo 1		Acção sísmica Tipo 2	
Zona Sísmica	a_{gR} (m/s^2)	Zona Sísmica	a_{gR} (m/s^2)
1.1	2,5	2.1	2,5
1.2	2,0	2.2	2,0
1.3	1,5	2.3	1,7
1.4	1,0	2.4	1,1
1.5	0,6	2.5	0,8
1.6	0,35	–	–

Fig. 5. 4 - Type1 and Type2 reference peak ground accelerations (*Norma Portuguesa*, 2009).

To find the two elastic response spectrum we have to point out the soil type in which the work is seated; in this case are rocks and, consequently, A ground type.

Em Portugal, para a definição dos espectros de resposta elásticos, o valor do parâmetro S deve ser determinado através de:

para $a_g \leq 1$ m/s^2 $\quad S = S_{max}$

para 1 m/s$^2 < a_g < 4$ m/s^2 $\quad S = S_{max} - \frac{S_{max} - 1}{3}(a_g - 1)$

para $a_g \geq 4$ m/s^2 $\quad S = 1{,}0$

em que:

a_g valor de cálculo da aceleração à superfície de um terreno do tipo A, em m/s^2;

S_{max} parâmetro cujo valor é indicado nos Quadros NA−3.2 e NA−3.3.

Em Portugal, para a definição dos espectros de resposta elásticos para a **Acção sísmica Tipo 1** devem adoptar-se os valores do Quadro NA−3.2 em vez dos do Quadro 3.2.

Quadro NA−3.2 – Valores dos parâmetros definidores do espectro de resposta elástico para a Acção sísmica Tipo 1

Tipo de terreno	S_{max}	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,1	0,6	2,0
B	1,35	0,1	0,6	2,0
C	1,6	0,1	0,6	2,0
D	2,0	0,1	0,8	2,0
E	1,8	0,1	0,6	2,0

Quadro NA−3.3 – Valores dos parâmetros definidores do espectro de resposta elástico para a Acção sísmica Tipo 2

Tipo de terreno	S_{max}	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,1	0,25	2,0
B	1,35	0,1	0,25	2,0
C	1,6	0,1	0,25	2,0
D	2,0	0,1	0,3	2,0
E	1,8	0,1	0,25	2,0

Fig. 5. 5 - Seismic parameters (*Norma Portuguesa*, 2009).

Ones observed the rules prescription (Fig. 5. 5), seismic parameters and referred spectra are defined. Values showed are referred to a return period of 475 year considering a damping of 5%.

Tab. 5. 2 - "Ponte do Arco" seismic parameters

Type	**PGA**	**Elastic Response Spectrum Parameters**			
	(m/s^2)	S_{max}	T_B (s)	T_C (s)	T_D (s)
1	0.35	1	0.10	0.60	2.00
2	0.80	1	0.10	0.25	2.00

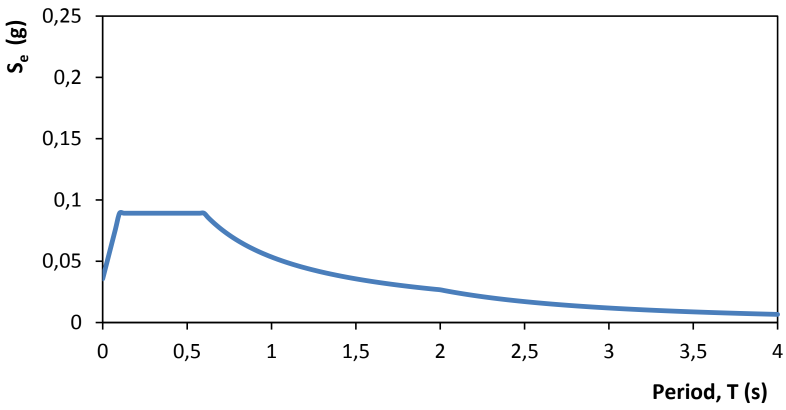


Fig. 5. 6 - Type 1 (far-field) elastic response spectrum (*Norma Portuguesa*, 2009).

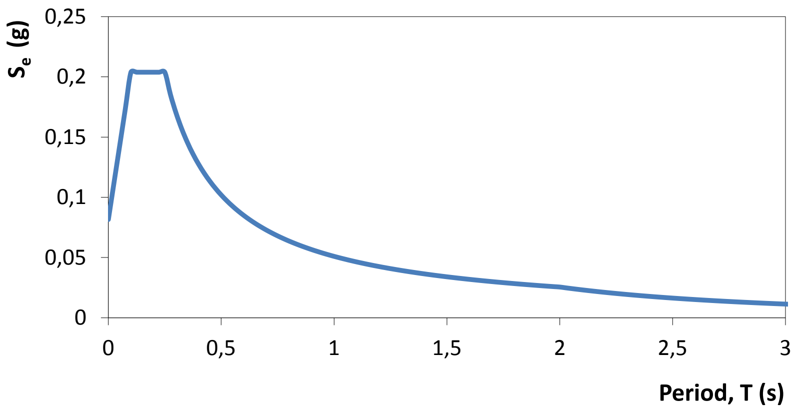


Fig. 5. 7 - Type 2 (near-field) elastic response spectrum (*Norma Portuguesa*, 2009).

5.2 DYNAMIC PROPERTIES

Dynamic properties identification has been made with Natural frequencies del solver of Straus7. The modal analysis corresponds to the study of the dynamic properties of structures under vibration excitation. Each structure, depending on its physical and mechanical characteristics (boundary conditions, stiffness, mass) possesses “individual” ways of vibrating (as many as the nr. Of degrees of freedom of the system), called natural modes of vibration, corresponding to harmonic motions. A mode of vibration is characterized by a modal frequency and a mode shape.
Tab. 5. 3 shows the firsts 15 vibration modes with the referred frequencies and participating mass; in Fig. 5. 8 Mode 1 and Mode 3 are showed.

Tab. 5. 3 - Modal frequencies and participating mass for the first fifteen modes of numerical model.

Mode	Frequency (Hz)	X (%)	Y (%)	Z (%)
1	**8.609**	0.000	0.000	**41.258**
2	14.650	0.004	0.000	0.024
3	**17.990**	**44.799**	0.038	0.002
4	**20.600**	0.003	0.002	**22.275**
5	**21.620**	0.172	**23.512**	0.001
6	25.770	0.188	10.711	0.000
7	27.900	0.004	0.009	0.074
8	**30.380**	**22.775**	1.038	0.000
9	30.780	0.007	0.000	1.691
10	34.820	4.294	3.307	0.021
11	35.460	0.010	0.001	2.325
12	36.860	0.000	0.000	8.076
13	**38.960**	0.056	**34.459**	0.008
14	43.300	0.000	0.009	1.143
15	43.940	0.008	0.143	0.000
		72.320	**73.229**	**76.898**

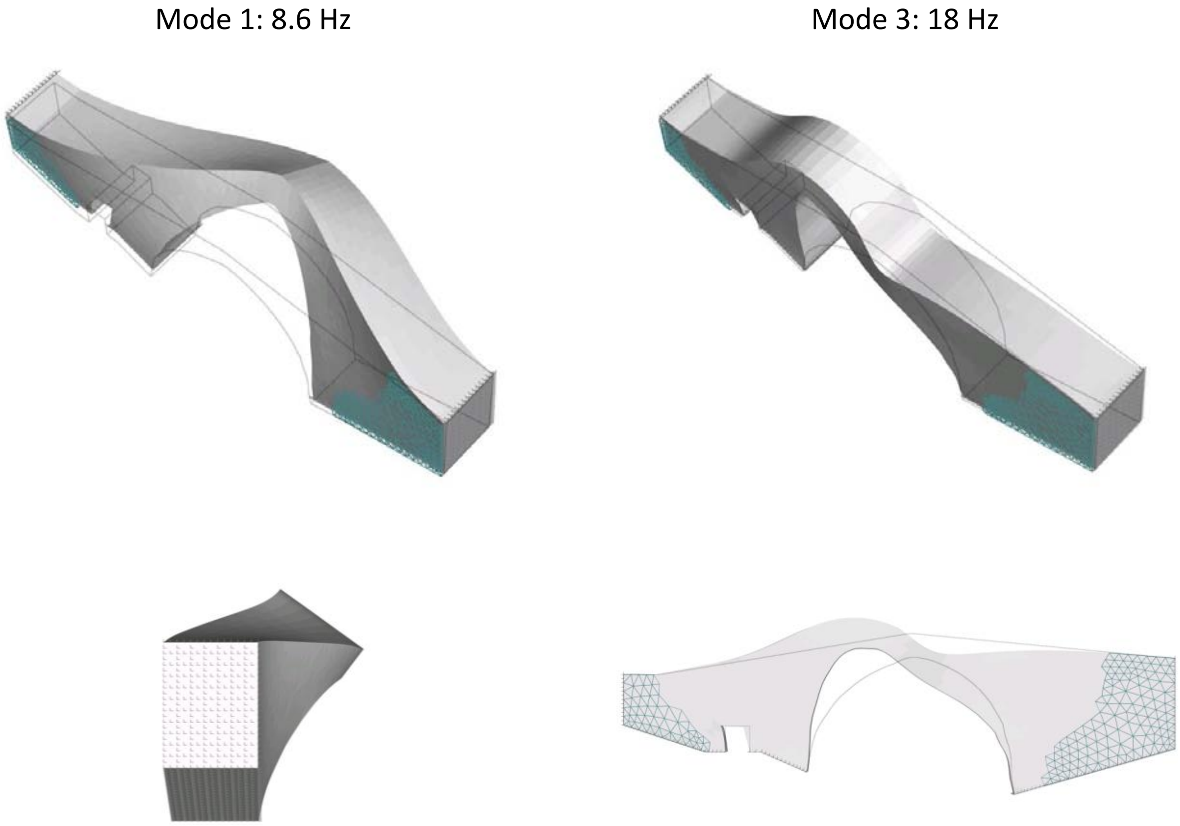


Fig. 5. 8 - Numerical mode shapes and frequencies for the global model

5.3 MODAL RESPONSE SPECTRUM ANALYSIS

In Response-Spectrum Analysis the seismic action is represented by a response spectrum (see Fig. 5. 7) which serves to excite the structure to be analyzed. The advantage is that this method requires very little input data and low computational effort.

From this analysis results that under the seismic action, the response, applied onto cross direction, of the bridge is elastic with a lower stress values (Fig. 5. 9); for those reason it has decided to make a pushover analysis with the goal of evaluate the bridge behaviour when the seismic action grows.

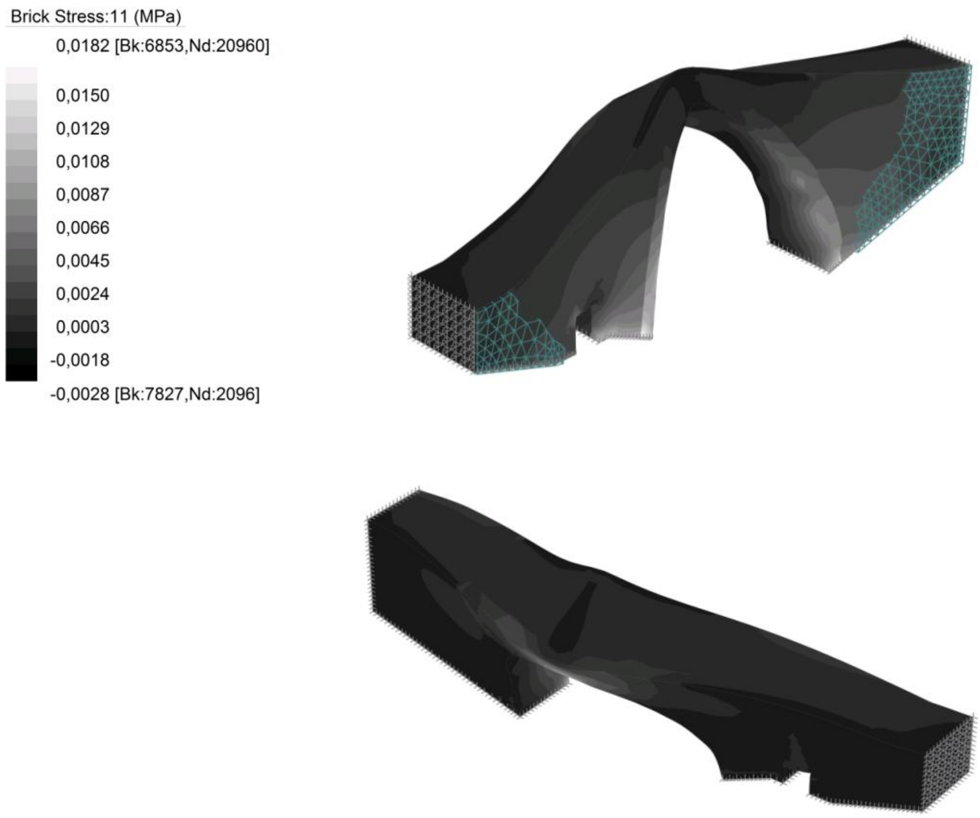


Fig. 5. 9 - Stress graphs by modal response spectrum analysis

5.4 PUSHOVER ANALYSIS

Preliminary natural frequency analysis was carried out in order to determine the bridges' resonant frequencies and the mode shapes. These analyses showed the prevailing influence of the first mode of vibration on the dynamic behaviour of the structures. Pushover analyses were performed by applying to each bridge a monotonically increasing pattern of transversal forces, representing the inertial forces which would be experienced by the structure during the ground shaking. The loading is imposed in a two-step sequence making use of a numerical model characterized by material and geometric nonlinearity. In the first step, the vertical (permanent) load is applied and in the subsequent steps the lateral loads are added in an incremental way. The maximum capacity of the structure corresponds to the situation in which a further lateral load increment is impossible. The selection of an appropriate lateral load distribution is a key factor of the pushover analysis, since the loads should represent the inertial forces acting on the structure during the earthquake. In the present work, lateral forces proportional to the mass distribution are used (Pelà, 2009).

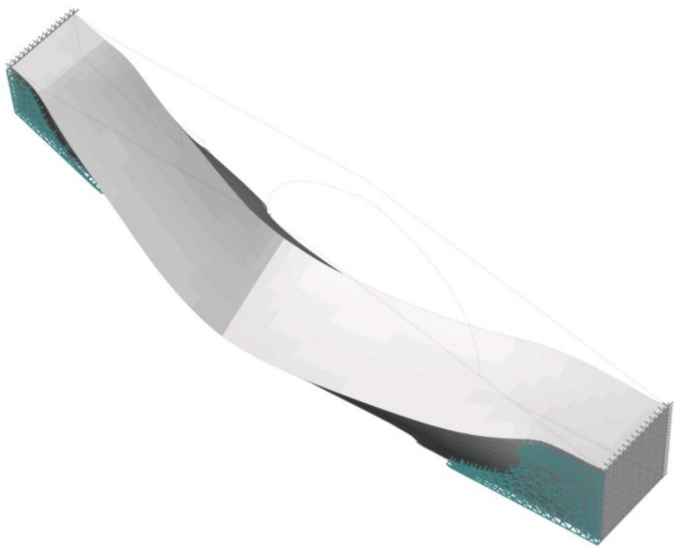


Fig. 5. 10 - Pushover analysis

The Fig. 5. 11 shows the relation between the displacement of control point (on the crown’s top) and α multiplier, that represents a ratio between the horizontal forces and a quantity depending of the correspondent weights of the present masses.

As we can see the structure starts to has a non linear behaviour when α is 0.7g with a displacement of 0.006 m.

α is more than three times S_e (T_1 = 0.116 s) = 0.2g as show at Fig. 5. 7 that represents the demand of seismic action. So the structure can supports this action in total safety even in this model.

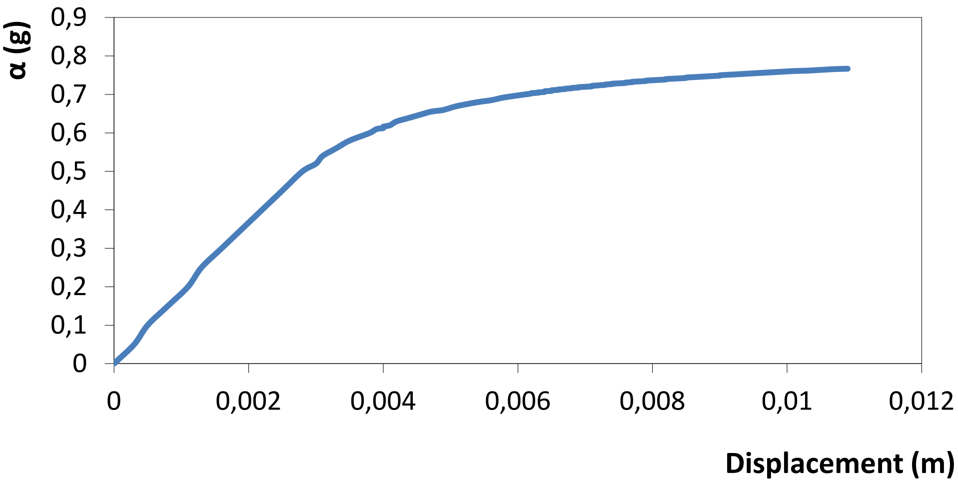


Fig. 5. 11 - Pushover analysis graph

5.5 ANALYSIS UNDER SELF-WEIGHT

Finally, thank to the model’s exploitation, we made an incremental analysis, increasing self weight values to find a vertical safety factor.

Applying a six-timed self weight, the model shows a linear behavior too; this confirms that the bridge is in a good safety state. The result confirms the masonry arch bridge present a good performance with distributed loads; these actions, if led until defined values, develop axial forces on the arch.

6. CONCLUSIONS

The present job has allowed to achieve a global view of the masonry bridge problem; particularly, thank to the analysis of “*Ponte do Arco*” it has been possible to define the problem through multiple points of view, historical, geometrical, structural and engineering that it presents.

The study, has highlighted a series of questions starting from the age: referring to this point, would be suitable to search documents and papers that proof the real dating and, would be possible to compare other construction with, showing the differences and similarities between feature, material and building methods too.

The properties material analysis allowed to pay attention at case’s needs: specifically the opportunity to lead inspections on the backfill would be better got if it was compared with samples to understand the nature and the material resistance. This is the natural consequence of the parametrical analysis of these values on the load carrying capacity of the bridge.

From the verification of the load carrying capacity, is issued the incidence of material’s properties, the friction coefficient, on the general behaviour of the structure; its importance, allow to speculate about its crucially.

Starting from the analysis of existing structural damages and considering which are major, it is necessary a continuous monitoring strategy both on voussoirs location and specifically on the crow and on joints’ opening along the barrel to understand them evolution.

The dynamic characteristics started the game when we analyzed the evaluation on seismic response of the structure: these properties can be defined with an analysis applied to a numerical model as done; however the praxis follows dynamic tests on the building to evaluate how good is the model adopted.

When we consider the vertical action on the “*Ponte do Arco*”, according to the model adopted, it shows a good resistance to the vehicles pass and it does not present specific safety critics.

About the structural behaviour, the FEM analysis simplified (macro-modelling) has allow to obtain an estimation referred to the seismic actions: we deduced that the response analysis is higher compared to the demand.

To get a more honest feedback on the acquired vales, would be necessary to build an accurate model, using the micro-modelling with the definition of interfaces between the different part of the building. This is achievable only when a deep knowledge on the material properties is acquired. On other hands is possible to use the DEM (discrete element model), in which the nature of masonry, and specifically the dry masonry, is likely to the reality.

In each case a specific element to keep in mind is the lower seismic risk of the zone.

Taking a look to the future perspective, the masonry bridge analysis is open to a great deal of developments: surely, the definition of an universal method adopted from the scientific community would be desirable; however, due of the peculiarity of each real case is so hard to consider that this hypothesis comes alive. Referring to a model analysis on the bridge’s behaviour under seismic action, we are not in the condition to purpose one which provides a convincing answer.

Concluding, the study of masonry bridges showed is proficiency: the discovery of this specific field, with the annexed topics, has allowed to develop a deep interest on the “*shape*”, compound by beauty, reason and symbolism.

CONCLUSIONI

Il lavoro condotto ha permesso di portare a termine e di definire una visione globale della problematica relativa ai ponti muratura; in particolare, grazie all’analisi condotta sul “*Ponte do Arco”* è stato possibile affrontare compiutamente il problema attraverso le molteplici sfaccettature di natura storica, geometrica, strutturale e ingegneristica che esso presenta.
Nel dettaglio, lo studio ha svelato una serie di problemi che partono, innanzitutto dalla condivisione, tra gli studiosi, circa il problema della datazione dell’opera: in riferimento a questa vertenza, infatti, sarebbe opportuno, approfondire la ricerca dei documenti comprovanti l’effettiva datazione e, al contempo, sarebbe possibile procedere ad un confronto con altre costruzioni della zona, rammostrando le differenze e similitudini tra forma, materiali e tecniche costruttive adottate.
Il vaglio inerente le proprietà dei materiali ha consentito di prendere coscienza circa le necessità che si manifestano: nello specifico, l’opportunità di condurre attività di ispezione nel riempimento si corroborerebbe con la realizzazione di un pozzo di ispezione finalizzato a capire la natura e la resistenza dei materiali impiegati. Tutto ciò si pone come la naturale conseguenza derivante dall’analisi parametrica dell’incidenza di tali valori sulla capacità di carico del ponte.
Dalla verifica sulla capacità di carico è emersa l’incidenza della proprietà del materiale, cioè l’angolo di attrito, sul comportamento generale della struttura; la sua importanza ci permette di ipotizzare che sia in ogni caso necessario determinarne il relativo valore.
Partendo dall’analisi dei danni strutturali presenti e preso atto di quelli aventi un’incidenza maggiore, si dimostra necessario un continuo e pedissequo monitoraggio sulla dislocazione dei conci, in particolare quelli presenti in chiave nonché sull’ingenza delle aperture dei giunti all’interno della volta per poterne comprendere l’eventuale evoluzione.
Le caratteristiche dinamiche del ponte entrano in gioco quando si effettuano valutazioni sulla risposta alle azioni sismiche: tali proprietà possono essere determinate tramite un’analisi applicata ad un modello numerico come è stato effettuato; tuttavia, la prassi esegue test dinamici direttamente sull’opera anche per valutare la bontà del modello ipotizzato.

In relazione alle azioni verticali agenti sul "*Ponte do Arco*" risulta che il medesimo, in base alle ipotesi considerate ed ai modelli adottati, sia in una buona condizione di resistenza e il passaggio degli autoveicoli entro determinati parametri, non presenti particolari problemi per la sicurezza.

Per quanto riguarda il comportamento della struttura, l'analisi FEM semplificata (macro-modelling) ha permesso di ottenere una stima rispetto alle azioni sismiche: se ne è dedotto che la capacità di risposta è nettamente superiore alla domanda richiesta.

Per ottenere un riscontro maggiormente veritiero sui valori così acquisiti, sarebbe altresì necessario costruire un modello maggiormente accurato, utilizzando una micro-modellazione con la definizione di interfacce specifiche tra le varie parti dell'opera. Tutto ciò è realizzabile solo nel caso in cui sia stata acquisita una conoscenza adeguata delle proprietà dei materiali. Altra via percorribile è quella definita dai DEM (discrete element model), nella quale la natura della muratura e, in particolare nel caso della muratura a secco, risulta essere maggiormente verosimile.

Un dato che, in ogni caso per la particolarità del contesto in cui il ponte si trova, deve essere tenuto in debita considerazione, è il fatto che la zona presenta un basso rischio sismico.

Volendo volgere lo sguardo ad una prospettiva futura, l'analisi dei ponti in muratura è aperto a numerosi sviluppi: sicuramente, la definizione di un metodo univocamente condiviso dalla comunità scientifica sarebbe auspicabile; tuttavia, date le peculiarità che i singoli casi concreti di volta in volta presentano è arduo ipotizzare che ciò sia effettivamente perseguibile. In relazione allo sviluppo di un modello di analisi comportamento dei ponti in muratura sotto l'azione sismica non si è ancora in grado di formularne uno in grado di riproporre una risposta che sia verosimile.

In ultima analisi, l'indagine relativa al dominio dei ponti in muratura si è dimostrata vincente e proficua: l'addentrarsi in tale ambito imbattendosi nelle problematiche connesse, ha permesso sviluppare un profondo interesse per la "*forma*", fatta di bellezza, ragione e simbolo.

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